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PAPER TITLE	Evaluation of Long Term Performance of Warm Mix Asphalt Using Advanced 3-D Imaging Technique and Indirect Tensile Test		
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Warm mix asphalt, indirect tensile test, image analysis, adhesion failure, stripping

ABSTRACT:

Warm Mix Asphalt (WMA) is produced at lower temperature than Hot Mix Asphalt (HMA) and is more susceptible to adhesion failure at the binder-aggregate interface. To evaluate the potential mechanisms, the extent of failure is computed using the indirect tensile test. Two binder types and different compaction temperatures were used for sample fabrications. Three levels of conditioning were used namely; unconditioned, Long-Term Aged (LTA)+1 Freeze-Thaw (F-T) cycle and LTA+3 F-T cycles. Effects of conditioning on indirect tensile strength (ITS), fracture energy and percent adhesion failure of mixtures were investigated. Percent adhesion failure was quantified using the advanced 3-D imaging technique. Test results showed that polymer modified binders have greater ITS and fracture energy than unmodified binders regardless of compaction temperatures and conditioning levels. Most modified mixtures passed the test even after 3 F-T cycles. Percent adhesion failures of modified binders were lower than unmodified binders. The extent of failure also increased with the number of F-T cycles.

Evaluation of Long Term Performance of Warm Mix Asphalt Using Advanced 3-D Imaging Technique and Indirect Tensile Test

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1 INTRODUCTION

Over the years, extensive research works have been conducted to evaluate the ability of warm asphalt technology to reduce asphalt production and laying temperatures. Warm mix asphalt (WMA) being produced at lower temperatures than HMA is more susceptible to adhesion failure at the aggregate-binder interface due to moisture damage (Khodaii et al. 2012). Various test procedures have been developed to examine the moisture susceptibility of asphalt mixtures. Moisture sensitivity tests are divided into two main groups. The first group is based on qualitative measurement on loose mixtures that involves observing changes in the percentage aggregate coating after immersion in water or in boiling water. Most of the current qualitative measurements are based upon visual inspections using the naked eye. The second group of tests is based on quantitative measurement. They are conducted on compacted mixtures to determine the mechanical strength. The main drawback associated with these two groups of test is that standard laboratory procedures are only capable of computing them separately.

Hamzah et al. (2014) combined these two groups of measurements using the strength properties of asphalt mixture via direct tension test and also image analysis of the fractured surface of asphalt mixture specimens subjected to direct tensile test. However, the surface of the fractured specimen is not always parallel to the plane of the 2-D captured image. This may induce a certain degree of inaccuracies when quantifying the amount of binder stripping from the aggregate surface. To overcome the issue, this study presents an innovative approach utilizing 3-D imaging techniques to compute adhesion failure in relation to its mechanical strength using the indirect tensile test. Investigations of the adhesive failures of asphalt mixture in most previous studies have been focused on specimens subjected to only moisture conditionings. However, the effect of aging that stiffens the binder with time may also contribute to additional adhesive failure apart from moisture damage. The contribution of this paper which includes the combined effects of long-term aging and moisture conditioning may provide some preliminary information on the long-term adhesive strength of both HMA and WMA mixtures. The resistance of each mixture type in terms of adhesive strength can then be better evaluated by subjecting them to the worst environmental conditions.

2 RESEARCH OBJECTIVES AND APPROACH

This paper investigated the adhesion strength of various asphalt mixtures in relation to its mechanical strength using indirect tension. All mixtures were subjected to different levels of conditioning. This paper adopted the approach used by Hamzah et al. (2014) in which the compacted mixture was first subjected to the performance test, followed by examination of the fractured surface using the 2-D image analysis technique. The 2-D imaging technique used was strictly monoscopic with images of specimens taken from merely one viewpoint. This paper attempts to overcome this shortcoming by instead employing a 3-D imaging technique. This gives more precise quantification of the adhesion failure. **This study aims to introduce and explain the basic concepts related to the use of image analysis rather than generalizing on the laboratory results. The approach adopted in this study was universal and can be applied in various fields to further quantify surface failures. In Malaysia, qualitative measurement is still carried out based on visual inspection using the naked eye. Such quantification is very subjective in nature and varies with different observers. With the introduction of 3-D imaging technique, it may be incorporated into the Malaysian Public Works Department specifications to evaluate stripping if it is made mandatory in the future.** Adhesion failures attributed to the combined effects of aging and moisture conditioning were investigated. The fracture

energy needed to break the unconditioned and conditioned specimens was also computed from the load-deformation curve of ITS.

3 EXPERIMENTAL WORKS

3.1 Materials

Two types of binders were used in this investigation, namely; an unmodified PG-64 and a styrene-butadiene-styrene (SBS) modified PG-76 binder. The granite aggregates used in the preparation of all mixtures were supplied by Kuad Sdn. Bhd. in Penang. All mixtures incorporated Pavement Modifier (PMD) as the filler. PMD is a grayish-black powder mineral filler that has been used for anti stripping purposes. The mix gradation is shown in Table 1. WMA mixtures were prepared using Evotherm[®] 3G (M1) as the warm mix additive, added at a dosage rate of 0.5 % by mass of binder. According to [Hurley and Prowell \(2006\)](#), this was the optimum dosage rate. The properties of Evotherm[®] 3G and its blending parameters with base binder are presented in Table 2.

Table 1. Aggregate gradation for mixture type AC-14 ([PWD 2008](#)).

Sieve sizes (mm)	20	14	10	5	3.35	1.18	0.425	0.15	0.075
Lower-upper limits	100	90-100	76-86	50-62	40-54	18-34	12-24	6-14	4-8
Passing (%)	100	95	81	56	47	26	18	10	5

Table 2. Physical and chemical properties of WMA additive ([MeadWestvaco 2012](#)).

Properties	Physical state	Color	Odor	Flash point	Solubility in water	pH	Boiling/condensation point
Evotherm [®] 3G (M1)	Liquid	Amber (Dark)	Amine -like	Closed cup: 204.4 °C	Insoluble	10-12	> 200 °C

3.2 Specimen preparation

A total of 60 specimens was prepared for this investigation, including three conditioning types, four different mixing and compaction temperatures and one anti stripping agent as the filler. The dimensions of the specimens prepared and tested in this study were 100 mm in diameter and 63.5 mm in height. The Servopac Gyrotory Compactor was used to compact the specimens to 7±1% air voids. The Marshall Mix design procedures [ASTM D1559 \(ASTM 2006\)](#) conforming to the Malaysian Public Works Department (JKR) specification ([PWD 2008](#)) for mix type AC-14, was applied to determine the optimum binder content of all mixtures. The optimum binder contents of the base PG-64 binder were 5.0% and 5.2% for HMA and WMA mixtures, respectively. The resultant optimum binder contents for the PG-76 mixtures were 5.3% and 5.6%, respectively. The mixing and compaction temperatures of all mixtures are listed in Table 3.

Table 3. Mixing and compaction temperatures of asphalt mixtures.

Binder	Mixture	Mixing temperature (°C)	Compaction temperature (°C)
PG-64	HMA	160	150
	WMA Control (0 % additive)	140	130
	WMA (0.5 % additive)	120-140	110-130
PG-76	HMA	180	170
	WMA Control(0 % additive)	160	150
	WMA (0.5 % additive)	140-160	130-150

3.3 Laboratory moisture conditioning

Specimens were first aged using the long-term aging process described in [AASHTO R30 \(AASHTO 2008\)](#) followed by moisture conditioning. Moisture conditioning of the specimens was carried out in

accordance with [ASTM D4867 \(ASTM 2006\)](#) procedures. The only modification made was the addition of Sodium Carbonate (Na_2CO_3) to the distilled water at a concentration of 6.62 g per liter to accelerate stripping within the asphalt specimens ([Hicks et al. 2003](#)). The specimens were then immersed in the solution and vacuumed for 15 min. to attain saturation levels in the range of 55% and 80%. Subsequently, these specimens were subjected to freezing at -18 ± 3 °C for 16h and thawing at 60 °C for 24 h as one cycle. This investigation was carried out on three sets of specimens; unconditioned, conditioned 1 freeze thaw (1 F-T) cycle and 3 freeze thaw (3 F-T) cycles.

4 TEST METHODS

The indirect tensile strength (ITS) test was used to characterize the tensile behavior of asphalt mixtures. The test temperature for all mixtures was set at 15 °C. Such a low temperature stiffened the specimens and easily disintegrated when loaded until failure ([Xie et al. 2012](#)). Hence, this low temperature condition will ensure that the asphalt mixtures attain close to elastic properties ([Huang et al. 2003](#)). The loading rate of 50.8 mm/min was selected for use in this investigation. The fractured cross-section was photographed after the test using a high-resolution 16 megapixels digital camera. The percentages of adhesion failures and broken aggregates were determined by an image analysis software. The Indirect Tensile Strength (ITS) was computed using Equation (1).

$$ITS = \frac{2000 P_{\max}}{\pi D t} \quad (1)$$

Where ITS is in MPa; P_{\max} refers to the maximum force of each stress-elongation curve; D and t are the diameter and thickness of the specimen, respectively in mm.



Figure 1. Experimental setup.

5 IMAGING TECHNIQUE

Images of the fractured surfaces of all specimens were taken after the test for image analysis. An imaging technique in 3-D was used to compute the percentages of adhesion failures and broken aggregates of each specimen.

5.1 Imaging technique in 3-D

In this paper, imaging technique in 3-D was introduced to more precisely quantify the amount of binder stripping from the aggregate surfaces. Prior to 3-D image analysis, ordinary 2-D photos had to be first merged and converted to a 3-D model using the 123-D Catch software.

5.2 Merging 2-D images into a 3-D model

Autodesk 123-D Catch software was utilized for this purpose. In this paper, a minimum of 20 photos were taken for each specimen at different angles from the horizontal axis. Figure 2 shows the conversion of 2-D photos, captured at various angles, into a 3-D model. These models were then saved and exported into Cloud Compare for 3-D image analysis.

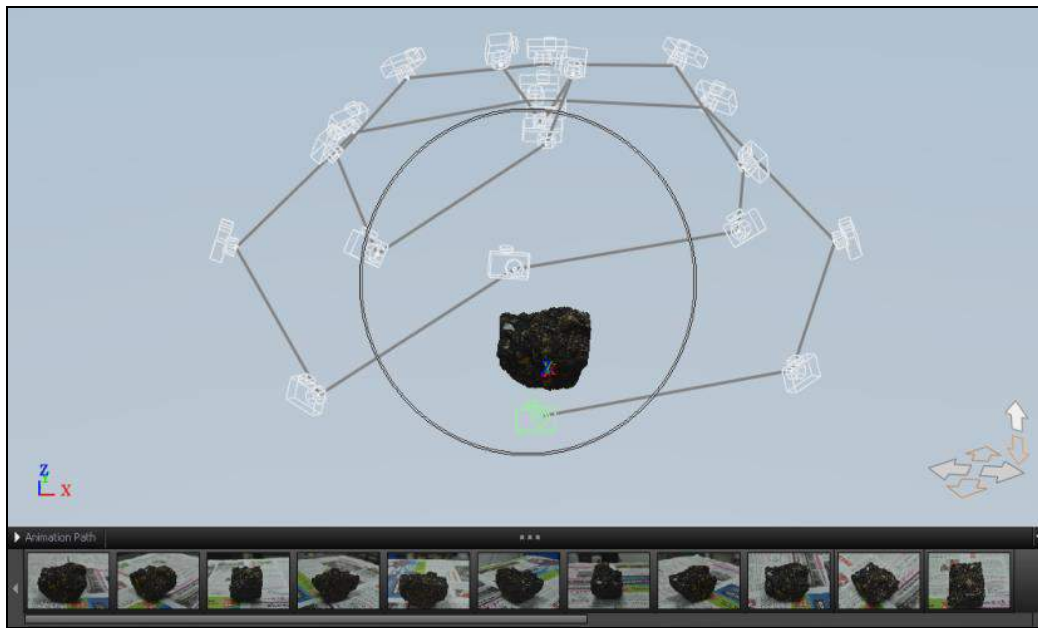


Figure 2. Conversion of ordinary 2-D images into a 3-D model.

5.3 3-D Image processing

Subsequently, after conversion, these 3-D models were exported into Cloud Compare for image processing. Figure 3(a) shows one of the 3-D models imported from 123-D Catch. Prior to processing, this model had to be first cropped, leaving only the fractured surface of the specimen as depicted in Figure 3(b). This model was then converted to scalar fields for statistical computations. The multidimensional domain of the model was reduced to lower dimensions, consisting of only one or two component axis. Conversion to grayscales yielded models with merely one band or intensity. Lower values were associated with binder covered areas and higher intensities corresponded to strip and broken aggregate surfaces. Figures 4(a) and (b) show the conversion of the segmented fractured surface of the original 3-D model into grayscales.

Thresholding was used for classification of the 3-D models. The default number of classes used in this software was 255. Determination of the threshold range values for stripped and broken aggregate surfaces were executed via the trial and error method by narrowing the number of classes between 0 and 255. Figure 4(c) shows the classification of the model into stripped and broken aggregate areas, respectively. The statistical tool included in this software conveniently displayed the amount of pixel point clouds that corresponded to each of the threshold range values selected. These values were then used to compute the percentages of adhesion failure and broken aggregate.

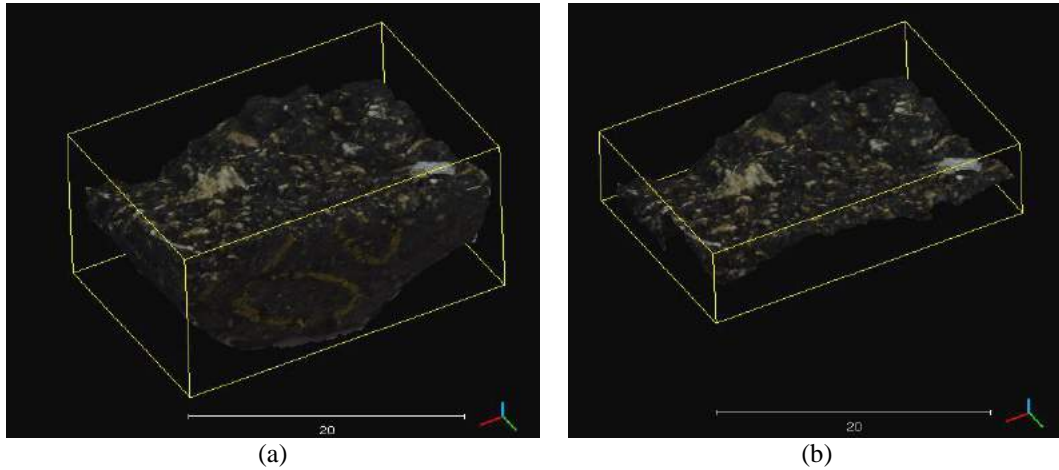


Figure 3. (a) A 3-D model of WMA mixture mixed at 130 °C, imported from 123-D Catch, (b) Segmented fractured surface of the 3-D model.

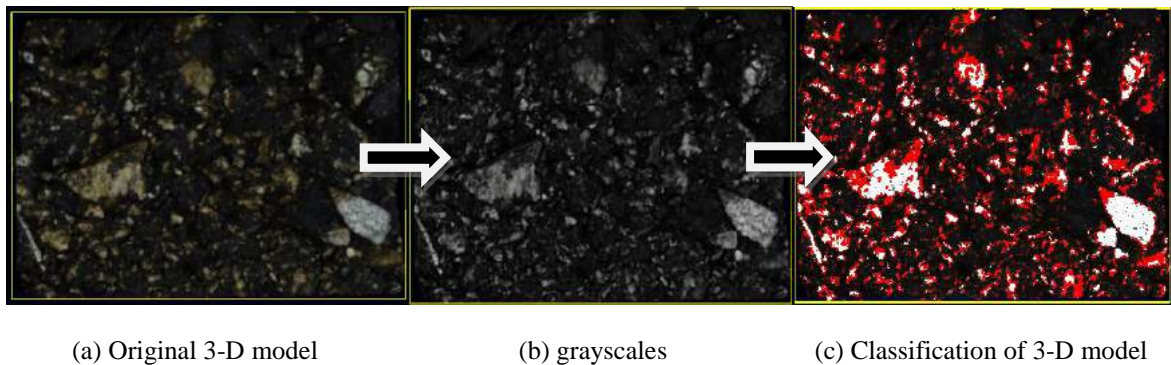


Figure 4. Conversion of the segmented fractured surface of the original 3-D model into grayscales.

6 RESULTS AND DISCUSSION

The results of the image analysis are used to supply information on the amount of pixels that correspond to the total fractured surface, stripped and broken aggregate areas. For computation of the results, failures attributed to the adhesion and broken aggregates are expressed in percentages, while the indirect tensile strength is expressed in MPa.

6.1 Influence of combined aging and moisture conditioning on adhesion failure

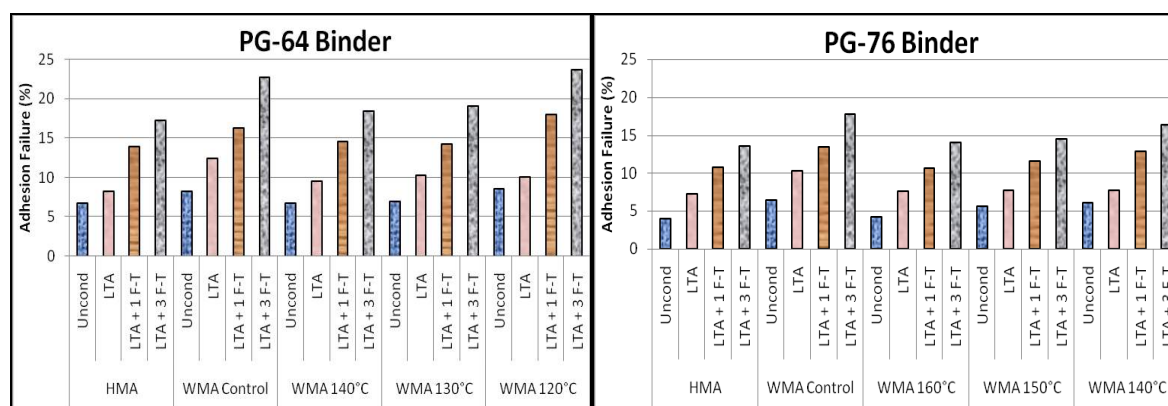
Figure 5 shows the percentage of adhesion failure for PG-64 and PG-76 binders, computed using the 3-D imaging technique. Based on these results, it can be observed that combined aging and moisture conditioning has a notable influence on the adhesive property of the asphalt mixtures, irrespective of binder type. The percentage of adhesion failure of both HMA and WMAs for PG-64 mixture is relatively higher than PG-76 mixture, regardless of conditioning levels. Aging of the specimens stiffens the binder contributing to higher adhesive failures after testing. All mixtures show an increment in adhesion failure after long-term aging. The percentage of adhesion failure further escalates with the number of F-T cycles.

The percentage of adhesion failure of dry specimens for all PG-64 mixtures, produced at different mixing temperatures, is relatively similar. However, WMA mixtures prepared at 140°C with 0% additive and 120°C, exhibited the highest failure with respect to adhesion after 1 F-T cycle. This may be attributed to the effect of lower production temperature, resulting in WMA mixtures more susceptible to adhesion failures due to moisture infiltration. As for the WMA mixture with 0% additive, this may be caused by the absence of warm mix additive to further enhance the adhesive properties of the mixture. WMA mixture (with additive) produced at 140°C and 130°C show comparable performance to HMA even after exposure to 1 F-T cycle.

After the combination of aging and 3 F-T cycles, the percent adhesion failure for HMA using PG-64 binder is still lower than the corresponding WMA mixtures at different mixing temperatures. This is an indication of the superior performance of HMA as compared to WMA for mixtures prepared with PG-64 binder, after being subjected to aging and higher F-T cycles. As for WMA mixtures, percentages of adhesion failure for WMA produced at 140°C and 130°C are relatively low and equivalent to one another after 3 F-T cycles.

According to Figures 5(a) and (b), all mixtures made using PG-76 binder exhibit lower adhesion failures compared to PG-64 mixtures, regardless of mixing temperatures and levels of conditioning. Xie et al. (2012) also reported similar findings. In contrast to PG-64 mixtures in dry condition, WMA mixtures produced at different temperatures using PG-76 binder have comparable adhesion failures as HMA. This may be due to the addition of warm mix additive into the mixture that enhances the adhesive properties of these mixtures. The benefit of using this additive is therefore more pronounced when used in conjunction with PG-76 binder. Lower percentage of adhesion failures signify that PG-76 mixtures perform better than PG-64 mixtures and therefore, more resistant to stripping. This may be explained by the rheological properties of the binder which make PG-76 mixtures more resistant to stripping at low temperature of 15 °C as compared to PG-64 mixtures.

Polymer modified binder exhibits higher strain capacity and hence, more ductile at failure compared with unmodified binder. This may be attributed to the lower freezing temperatures of PG-76 binders than PG-64 binders (Hamzah et al. 2014). Based on Khattak et al. (2007), the general glass transition temperatures of PG-76 binder ranged from -60 to -127°C, while the equivalent values for unmodified binder remained around -15 to -10°C. The addition of polymers directly contributes to the significant reduction in the glass transition temperature of asphalts. As a result, unmodified PG-64 binders are more brittle than PG-76 binders. PG-76 mixtures are expected to be more resistant to binder stripping and exhibit lower adhesion failures (Khattak et al. 2007). In this paper, the results have shown that most WMA mixtures in dry condition, particularly those prepared using PG-76 binders, are more resistant to adhesion failures than HMA. In addition, the percentage differences between HMA and WMA in terms of adhesion failure are lesser in PG-76 mixtures compared to PG-64 mixtures.



(a) PG-64

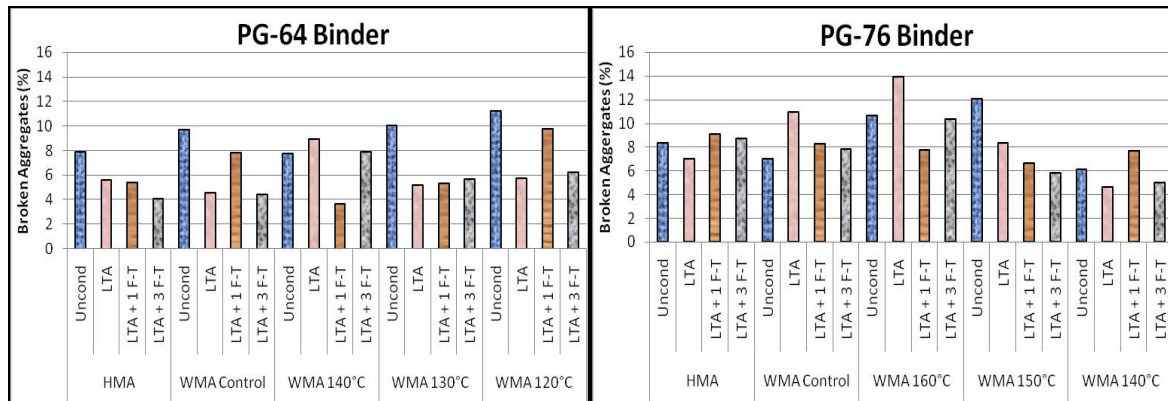
(b) PG-76.

Figure 5. Percentage of adhesion failure results of image analysis on fractured surfaces (Qualitative measurement).

6.2 Failures due to broken aggregates

Figure 6 shows the percentage of broken aggregates quantified using the 3-D imaging technique. The percentages of broken aggregate for PG-76 mixtures are generally higher than PG-64 mixtures. Hamzah et al. (2014) also reported similar trends and attributed this to aggregate gradation and orientation during compaction. Failures may be caused by the aggregates that are already fractured during the compaction process. There is also no specific trend or relationship between the percentages of broken aggregate and mixing temperatures as well as levels of moisture conditioning. Therefore, the trend of these failures is independent of mixing temperature and F-T cycle.

Based on the results shown in Figure 5(b), PG-76 binders exhibit lower percentages of adhesion failure compared to unmodified binders. This may lead to the propagation of failures along the cracks within the aggregates that have already developed during compaction. PG-64 mixtures which are more brittle at low temperature exhibit higher adhesion failures. As a result, the tendency of failures to occur in PG-64 mixtures is higher at the binder-aggregate interface rather than at the cracks developed during compaction. This explains the lower percentages of broken aggregates in PG-64 mixtures as depicted in Figure 6(a). More research is still required to investigate the nature of aggregate fracture during compaction, which may include factors such as aggregate gradation, shape, and orientation as well as its physical and chemical properties.



(a) PG-64

(b) PG-76.

Figure 6. Percentage of broken aggregate results of image analysis on fractured surfaces.

6.3 Influence of combined aging and moisture conditioning on indirect tensile strength and fracture energy

Figures 7(a) and (b) show the indirect tensile strength (ITS) results of PG-64 and PG-76 mixtures, respectively. All mixtures show an increment in ITS after long-term aging. The ITS of all mixtures when combined with moisture conditioning decreases with F-T cycles. The reduction in ITS is more significant after 3 F-T cycles compared to after 1 F-T cycle and in dry condition. The ITS of all WMA mixtures is found to be lower than HMA mixture, regardless of binder type. This indicates that WMA mixtures are more susceptible to moisture damage, attributed to their lower production temperatures. When comparing all WMA mixtures, WMA mixtures with 0% additive (WMA Control), prepared using PG-64 and PG-76 binders, generally show the lowest ITS after combined aging and 3 F-T cycles. This finding can again be substantiated by the absence of warm mix additive to help improve its adhesive strength. The overall results have shown that the ITS of PG-76 mixtures remain higher than PG-64 mixtures. Long-term aging increases the ITS of the asphalt mixtures, while combination with moisture conditioning decreases the ITS of the specimens. Based on the figures, a combination of long-term aging and 1 F-T cycle produces ITS of mixtures almost similar to those of unconditioned specimens.

Figures 8(a) and (b) demonstrate that both binder types have an indirect tensile strength ratio (ITSR) values above the 80% acceptable limit after aging and 1 F-T cycle. Aging process increases the ITS while, moisture conditioning lowers the ITS values. Subjecting the specimens to 1 F-T cycle might not be severe enough to lower the ITSR values below the permissible limit. However, all of PG-64 mixtures show ITSR values below the limit after 3 F-T cycles. The effect of adding warm additive is more effective in PG-76 mixtures when all mixtures except WMA control, are still able to exhibit ITSR values above the allowable 80% limit even after the severe 3 F-T cycles.

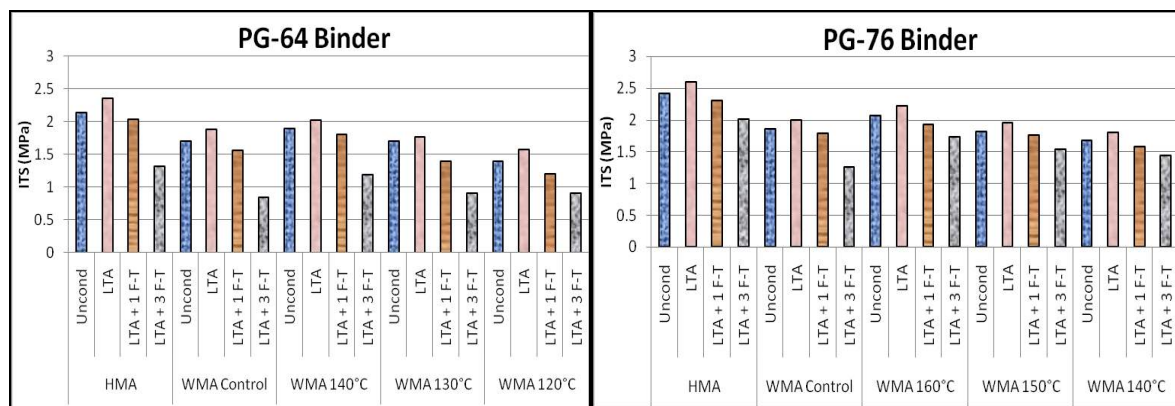
According to Figures 9(a) and (b), fracture energy of both PG-64 and PG-76 mixtures have shown a similar trend as the ITS of the mixtures. The energy needed to fracture HMA specimens of both binder types is always higher as compared to WMA mixtures, irrespective of mixing temperatures and conditioning levels. The aging process helps to stiffen the binder resulting in higher fracture energy required to break the specimens. However, with the inclusion of moisture conditioning, the fracture energy of all mixtures decreases correspondingly. Lowest fracture energy can be observed especially after 3 F-T cycles. This can be

explained by the infiltration of moisture into the binder-aggregate interface leading to adhesive failures within the bond.

Figure 10 demonstrates the effects of aging and moisture on the overall ITS test results. Aging increases the ITS of asphalt mixtures, while moisture conditioning has the opposite effect. The results show that binder type and moisture conditioning have notable effects on the ITS. Overall ITS for PG-76 mixtures is always higher than the corresponding PG-64 mixtures, regardless of conditioning level. Modified mixtures also exhibit lower reduction in ITS even after the severe 3 F-T cycles. The difference in ITS between unconditioned and 1 F-T cycle is very minimal for PG-76 mixtures as compared to PG-64 mixtures.

According to [Romeo et al. \(2010\)](#) and [Xie et al. \(2012\)](#), the tensile strength of SBS modified mixtures were slightly higher than unmodified mixtures. [Kanitpong and Bahia \(2005\)](#) also found that the performance of polymer modified binders was superior compared to unmodified binders. The addition of polymers enhanced the adhesion, cohesion as well as rutting properties of asphalt binders. Adhesion and cohesion properties of asphalt binders can be reliable indicators of the performance of asphalt mixtures conditioned in the laboratory.

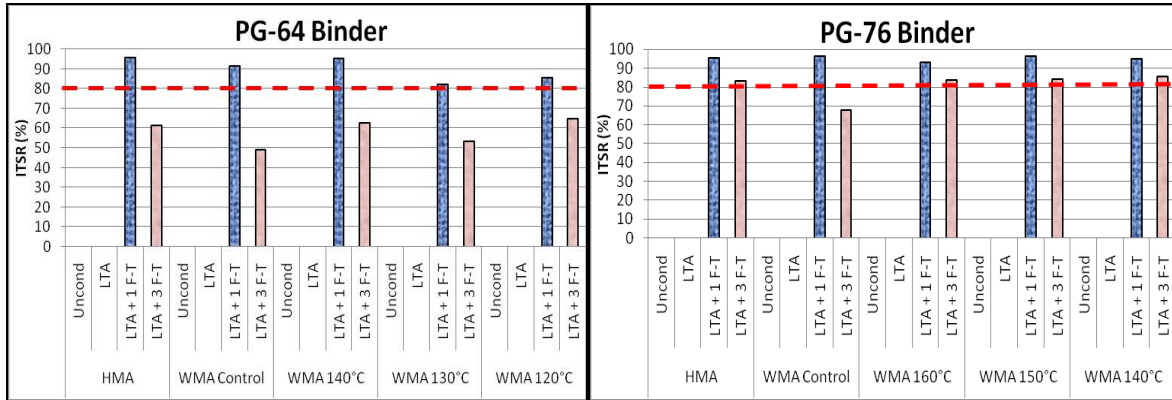
Based on the laboratory results, it can be observed that the overall ITS and fracture energy of PG-76 mixtures is higher than PG-64 mixtures, while the percentages of adhesion failure of PG-76 mixtures are lower than PG-64 mixtures. Hence, it can be concluded from these findings that ITS and fracture energy is inversely proportional to the percentage of adhesion failure. These parameters can, thus be very useful in predicting the long-term performance of asphalt mixtures in the laboratory.



(a) PG-64

(b) PG-76.

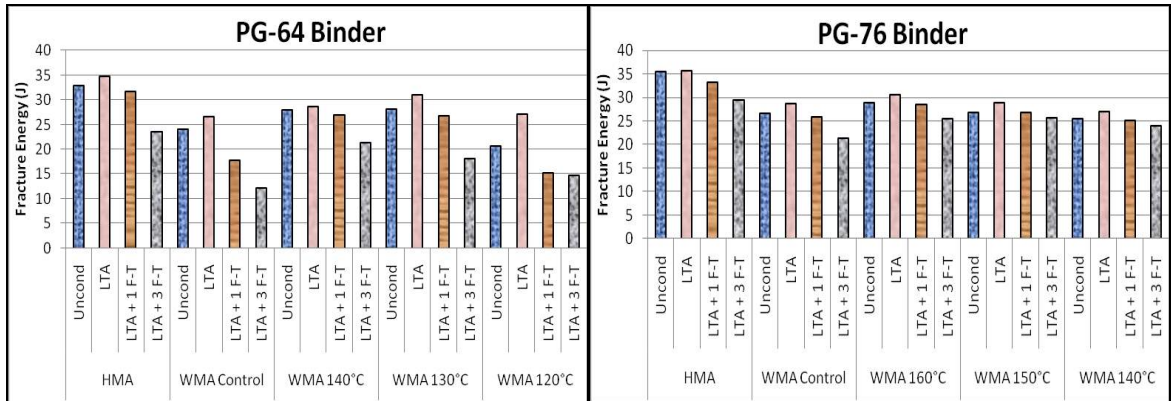
Figure 7. Indirect tensile strength results of fractured specimens (Quantitative measurement).



(a) PG-64

(b) PG-76.

Figure 8. Indirect tensile strength ratio results of fractured specimens.



(a) PG-64

(b) PG-76.

Figure 9. Fracture energy results of fractured specimens.

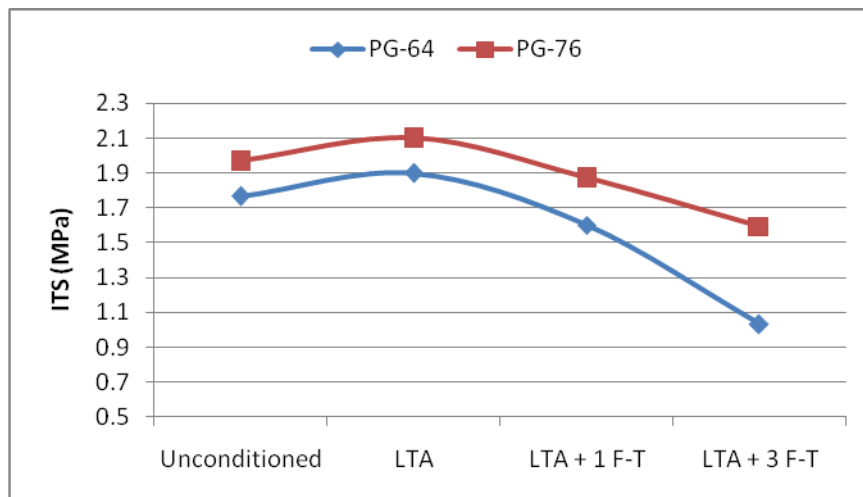


Figure 10. Effect of aging and moisture conditioning on the overall ITS of asphalt mixture.

7 CONCLUSIONS

The use of the 3-D imaging technique was necessary for more precise quantification of adhesion failure. Overall, the percentage failure due to adhesion indicated that PG-76 mixtures possessed better resistance to combined aging and moisture damage than PG-64 mixtures. The percentage failure increased with F-T cycles, irrespective of binder and mixture type. The percentage of adhesion failure for PG-76 mixtures was lower than that of the unmodified mixtures, while the percentage of broken aggregate showed an opposite trend to adhesion failure. Lower adhesion failures in the PG-76 binder have contributed to cracking in the aggregates already fractured during compaction. Thus, failures in broken aggregates for PG-76 mixtures were relatively higher than PG-64 mixtures. The ITS and fracture energy is reduced with the number of F-T cycles. These values were inversely proportional to the percentage of adhesion failure. Lower ITS and fracture energy corresponded to mixtures with higher percentages of adhesion failure.

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PAPER TITLE	Innovative Urban Signalized Junctions		
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Urban Signalized Junctions- Compact Underpass- Safety- ITS- Sustainability

ABSTRACT:

Urban signalized junctions normally suffer delays, accidents, pollution from long stopping times at signals. Typically, grade separations are not considered at these junctions for cost, ROW and aesthetics concerns resulting from long structures required to achieve minimum clearance and gentle grades for high/heavy vehicles.

In most cases high/heavy vehicles constitute 5%-20% of the traffic mix at urban signalized junctions which means grade separations are not allowed for less than 20% of the traffic. Cutting clearance requirements to nearly half and increasing grades to nearly double would take significant amount of traffic off the signal. A through underpass would take the heavier of through movements off the signal while innovative designs can allow taking both through movements and the heavier of left turn movements to level-1 yet allowing a full movement at grade signal. Underpasses with modified standards to accommodate small vehicles only should result in significant cost reductions, would require considerably less ROW than conventional underpasses. Additionally, having the grade separation designed fully underground would eliminate any negative aesthetic impacts of bridges. A full movement at-grade signal will allow high/heavy vehicles movement in all directions in addition to small vehicles in directions not provided in the underpasses. Priority could be provided to emergency vehicles and buses at signals through ITS.

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Introduction

Traffic signals at urban Junctions despite being a relatively successful means of regulating conflicting traffic on intersecting roads, they have major adverse impacts not only on road users being drivers, commuters on public transport or pedestrians but on the neighborhood residents in addition to other species like birds, trees, etc., While grade separations provide the solution for eliminating conflicts of intersecting traffic on expressways and freeways, transport agencies are always reluctant to use them on urban collectors and arterials due to their high costs, massive footprint requiring wide ROW for long lengths of the intersecting roads and maybe aesthetically undesirable if bridges are to be constructed facing and overlooking adjacent buildings. This paper will investigate the value of introducing underground grade separations designed to standards that allow only small vehicles to go through them accordingly, the high costs and large footprint of interchanges can be significantly reduced. The author argues that small vehicles constitute 80-95 % of the traffic mix at most urban junctions (for ease of reference will be referred to as 90% of the traffic mix throughout this paper). The remaining high/heavy vehicles which are mainly trucks and buses (for ease reference will be referred to as 10% of the traffic mix throughout this paper) will be allowed an at-grade signal at (level-Zero). Only grade separations at one level below ground (level -1) will be proposed throughout this paper to have high/heavy vehicles move safely at level-Zero and to eliminate the negative aesthetic impact of bridges. Innovative ideas will be proposed to allow the maximum number of movements a free flow operation. The following sections will discuss the disadvantages of traffic signals, limitations for providing grade separations on urban roads, percentage of heavy/ high vehicles on urban roads, providing grade separations for the dominant vehicle types in the traffic mix, ideas for innovative urban grade separations, evaluation of the proposed designs against sustainability standards, potential disadvantages of the proposed grade separations and finally a conclusion.

Disadvantages of traffic Signals

Although traffic signals have proven to provide an efficient solution for managing conflicting traffic whenever two or more roads intersect, they have several disadvantages. It is prudent for the goals of this paper to identify the disadvantages of traffic signals.

Delays:

Delays are inevitable at traffic signals. The Highway Capacity Manual (HCM 2010) defines delay as “the additional travel time experienced by a driver, passenger, or pedestrian” and chapter 18 of HCM provides equations for calculating the delay a motorist experiences due to the presence of a traffic signal. This includes time spent decelerating, in queue, and accelerating. Additionally the HCM attributes the Level Of Service (LOS) Criteria at signalized junctions to delay time experienced by the motorists. Table 1 provides LOS criteria of HCM based on seconds of delay per vehicle.

Table 1: LOS based on seconds of delay per vehicle

LOS	Control Delay per vehicle (seconds per vehicle)
A	≤ 10
B	> 10-20
C	> 20-35
D	> 35-55
E	> 55-80
F	> 80

Source HCM 2010

As delay increases, LOS gets poorer and congestion starts. In addition to the stress and frustration congestion causes to drivers, it has drastic effects on individual and national economies. A study by INRIX, a leading international provider of real-time traffic information, transportation analytics and connected driver services, in collaboration with the Centre for Economics and Business Research (CEBR), that was intended to quantify

the “cost of traffic congestion” on national economies in the U.S., U.K., France and Germany between 2013 and 2030 have concluded that over this period, the cumulative cost of congestion for these economies combined is estimated to be a staggering \$4.4 trillion. At the individual level, traffic congestion costed drivers \$1,740 on average across the four countries in 2013, and this number is expected to grow more than 60% to \$2,902 annually by 2030.

Safety:

Reports from the National Center for Statistics and Analysis (NCSA) office of National Highway Traffic Safety

Administration-USA show that between 2007 and 2011, 751 people died each year in red light running crashes and \$ 378 million in cost are lost each month due to red light fatalities. According to another report by ABC news (2013), red light running crashes result in 800 fatalities and 200,000 injuries each year. In the other side of the globe, Dubai, a city having one of the top five growing economies in the world, and now a main business and tourism hub in the Middle East (Everington, 2015) has one of the most advanced roadway networks in the Middle East. Jumping the red light resulted in 111 crashes killing 3 people and injuring 159 people in the first 9 months only of 2015 (Agarib, 2015).

Important questions to ask are: are red light runners necessarily intentional violators? And is running a red light the only cause of accidents at traffic signals? The answer is no to both questions as accidents at traffic signals may take place as a result of drivers being trapped in the dilemma zone. According to the Federal Highway Administration (FHWA), the dilemma zone is the area in which it may be difficult for a driver to decide whether to stop or proceed through the intersection at the onset of the yellow signal indication. Although numerous studies by different agencies tackled the problem of dilemma zone and offered possible mitigations, according to the Institute of Transportation Engineers (ITE) the dilemma zone is still a main reason for accidents at traffic signals as some drivers trapped in the dilemma zone will decide to stop while others will decide to continue. An abrupt stop may result in a rear end crash and a wrong decision to continue may result in a right angle crash (Ceccarelli, 2012).

Pollution:

Air and noise pollution are two major adverse impacts from stopping at traffic signals. In America, one third of carbon dioxide emissions come from moving people or goods, 80 percent of which come from cars and trucks. As traffic congestion increases, so too do fuel consumption and CO2 emissions. Therefore, congestion mitigation programs should reduce CO2 emissions (Barth & Kanok, 2009). From human health perspective, motorized vehicles emit polluting nanoparticles which contribute to respiratory and heart diseases. Traffic signals has been identified as hot spots for concentration of such pollutants. With drivers decelerating and stopping at signals, then accelerating to move quickly when lights go green, peak particle concentration was found to be 29 times higher than that during free-flowing traffic conditions (Kumar, 2015). In the same context, noise level increases with traffic volumes in an exponential manner (Marathe, 2011). Unlike high speeds at which noise pollution is mostly attributed to tire and aerodynamics noise, noise pollution on congested roads is mainly attributed to acceleration and breaking, engine and vehicle horn noise which have serious impacts on human health including headaches, stress and heart diseases.

This paper argues that at any urban junction, taking 90% of the traffic using an urban signalized junction off the traffic signal will considerably mitigate such disadvantages hence, will improve the overall traffic operations, the environment and the quality of life of people.

Limitations for Providing Grade Separation on Urban Roads

Typically, grade separations are not considered on urban roadways (collectors or arterials) for cost, ROW and aesthetics concerns resulting from long structures required to achieve minimum clearance and gentle grades required by the design standards. This has been best described by Hernandez (2011) stating that although grade separations allows traffic to move with fewer interruptions, resulting in higher overall speeds, the main disadvantage of grade separations is the cost. The large physical structures that form this type of roadways are significant and expensive. The structures are so large that their heights are obtrusive, which make them unpopular with nearby landowners. Trying to pass a plan for a new grade separation can be difficult because of the obtrusiveness and the cost.

The internationally recognized design manual “A Policy on Geometric Design of Highways and Streets-2011”, also referred to as the “Green Book” or “AASHTO” referring to the agency that published it “the

American Association of Highways and Transportation Officials” is used by many transport agencies throughout the world as a complementary document to the local geometric design standards. AASHTO has set grade and clearance standards for the different classifications of roadways. For the intent of this paper, the standards for urban collectors and urban arterials from AASHTO will be referred to.

For urban collectors, AASHTO sets the minimum vertical clearance for underpasses at 4.3 meters with an additional allowance for future resurfacing. Table 2 identifies the maximum grade requirements for urban collectors.

Table 2: Maximum grades for urban collectors

Type of Terrain	Metric								U.S. Customary								
	Maximum Grade (%) for Specified Design Speed (km/h)								Maximum Grade (%) for Specified Design Speed (mph)								
	30	40	50	60	70	80	90	100	20	25	30	35	40	45	50	55	60
Level	9	9	9	9	8	7	7	6	9	9	9	9	9	8	7	7	6
Rolling	12	12	11	10	9	8	8	7	12	12	11	10	10	9	8	8	7
Mountainous	14	13	12	12	11	10	10	9	14	13	12	12	12	11	10	10	9

Note: Short lengths of grade in urban areas, such as grades less than 150 m [500 ft] in length, one-way down-grades, and grades on low-volume urban collectors may be up to 2% steeper than the grades shown above.

Source: AASHTO 2011

While, for urban arterials, AASHTO sets the minimum vertical clearance for designs of new underpasses at 4.9 meters. It allows clearances to be designed at 4.3 meters only if an alternate route can be provided with a minimum clearance of 4.9 meters. Table 3 also identifies the maximum grade requirements for urban arterials.

Table 3: Maximum grade for urban arterials

Type of Terrain	Metric						U.S. Customary						
	Maximum Grade (%) for Specified Design Speed (km/h)						Maximum Grade (%) for Specified Design Speed (mph)						
	50	60	70	80	90	100	30	35	40	45	50	55	60
Level	8	7	6	6	5	5	8	7	7	6	6	5	5
Rolling	9	8	7	7	6	6	9	8	8	7	7	6	6
Mountainous	11	10	9	9	8	8	11	10	10	9	9	8	8

Source: AASHTO 2011

Many road agencies set more stringent values of minimum vertical clearance and longitudinal grades on urban roads. Table 4 identifies grades on urban collector and arterial roads (or equivalent depending on each agency classification) and vertical clearance requirements collected from different standards in the Gulf Cooperation Council (GCC) countries where the author was involved in roads design of several projects over a span of more than 10 years.

Table 4: Grade and vertical clearance requirements from different GCC standards

Standard Name	Authority/ Country	Maximum Grade	Vertical Clearance
Highway Design Manual	Ministry of Communication- Kingdom of Saudi Arabia	Urban Arterials 70 KPH: 4.5% Desirable- 6% max	5.5 meters
		Urban Arterials 60 KPH: 6% Desirable- 8% max	
Qatar Highway Design Manual	State of Qatar	Secondary routes- 6%	5.5 meters (original manual-1997), 6.0 meters (Circular-2012)
Geometric Design Manual for Dubai Roads	Dubai- United Arab Emirates (UAE)	Urban Collectors- 6% Urban Arterials- 6%	5.5 meters
Road Geometric Design Manual	Department of Transport- Abu Dhabi- UAE	City Boulevard: A table for different speeds ranging from 7% to 8%	6.5 meters

Hands on design experience in these countries concluded that unless dictated by extremely stringent constraints, grades of newly designed grade separations are never to exceed 5% and vertical clearance is never to be designed for values that are less than those shown in table 4. This is mainly to cater for heavy and high vehicles (mainly trucks and busses) in the traffic mix which have overall heights that are significantly bigger than private cars and vans. Additionally, loads transported on these vehicles (mainly trucks) or the limitations of the power of their engines or both make it difficult for them to negotiate steep grades. Accordingly, the provision of a grade separation at a congested signalized junction is a decision driven by cost and ROW availability that is based on achieving vertical clearance and grade requirements for these vehicles.

Percentage of Heavy and High Vehicles on Urban Roads

It can be argued that most professionals working in the field of transportation engineering would agree that the percent heavy vehicles used in their work whether being Transport Master Plans, Traffic Impact Studies or Pavement Designs in urban areas are mostly in the range of 5% to 10%, sometimes less than 5% and sometimes between 10 and 20% with a very rare likelihood of exceeding 20% and this would be only if the project is a truck route, located in an industrial area, close to a port, or similar. Hallenbeck, et al (1997) provided the information shown in table 5 for National average truck percentage by functional classification on US Urban roadways.

Table 5: National average truck percentage by functional classification on US urban roadways

Functional Class of Roadway- FHWA	Cars	Single-Unit Trucks Including Buses	Combination Trucks	Multi-Trailer Trucks
11	91.5%	2.6%	5.2%	0.7%
12	88.9%	3.7%	5.9%	1.5%
14	90.5%	3.2%	5.8%	0.6%
16	94.4%	2.4%	2.9%	0.3%

Class 11 = Urban Interstate

Class 12 = Urban Other Freeways and Expressways

Class 14 = Urban Principal Arterial

Class 16 = Urban Minor Arterial

Compiled from Hallenbeck, et al (1997)

From a different part of the world, the State of Qatar is an ambitious country that won the 2022 FIFA world cup bid and have massive construction projects taking place. Qatar roads network currently lacks a freeway/expressway system, accordingly most of its truck traffic serving the mega construction projects taking place in the country, particularly in the capital “Doha” run on the urban roadway system mainly comprising urban arterials and urban collectors, the intersections of which are controlled by traffic signals.

“The Doha expressway program is a 5 year program that consists of 30 Expressway projects to improve traffic flows, reduce congestion, travel time and environmental impacts in Doha with a total cost of US \$20bn” (KBR website).

Although, as stated above, most of the truck traffic take place on a network that lacks a freeway/expressway system, the chart shown in figure 1 below provided by the Ministry of Municipality and Urban Planning (MMUP) limited heavy vehicle percent at peak hour on Urban roads to around 16% while for the road classes that are the subject of this paper (Collectors and Arterials) represented in the chart by “Secondary Urban” and “Tertiary Urban” heavy vehicle percentage is limited to around 8% and 5% respectively.

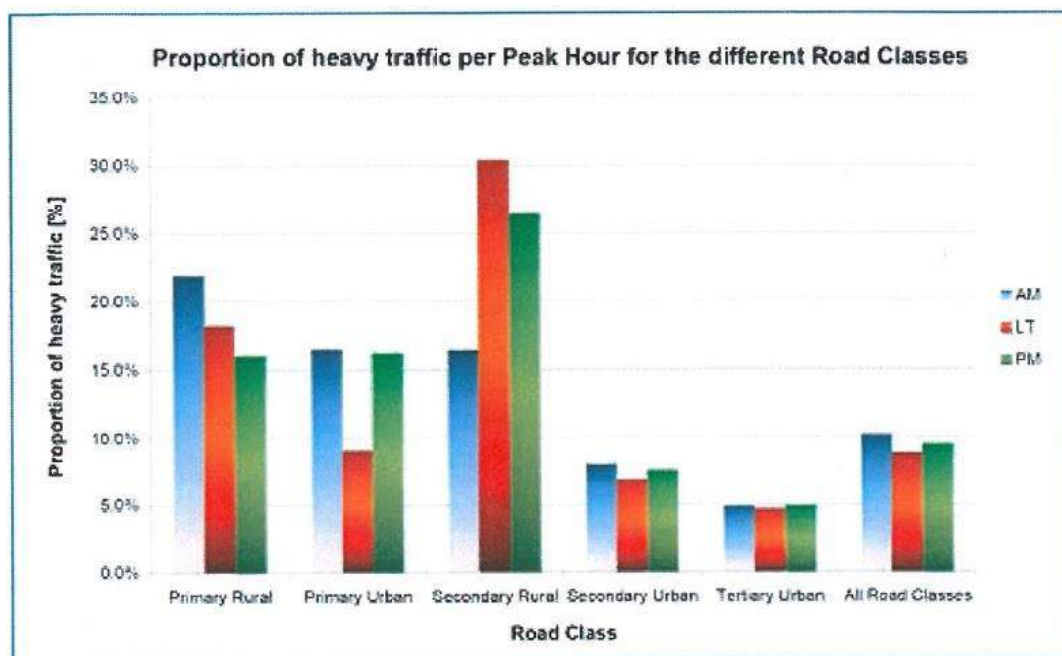


Figure 1: Proportion of heavy traffic in the peak hour on the State of Qatar roads

The author argues that further studies in different countries around the globe would result in similar or close to the same percentages of heavy vehicles on urban roads as in the USA or Qatar, accordingly and as mentioned in the introduction for ease of reference, on an urban signalized junctions assuming a traffic split of 90% small vehicles and 10 % high/heavy vehicles (trucks and busses) maybe considered a reasonable assumption.

Providing Grade Separations for the Dominant Vehicles in the Traffic mix

Restricting the decision to provide grade separations on meeting the design criteria for high/ heavy vehicles, in the author’s vision, is a big obstacle to mobility, safety and sustainability goals as it results in having the dominant types of vehicles in the traffic streams suffer stopping at signals and the entire community suffer the adverse impacts of such recurrent stopping of traffic for 10% only of the traffic travelling on urban roads.

This paper argues that grade separations applying vertical clearance and grade requirements to accommodate small vehicles only should be thoroughly explored as a solution if congestion is experienced at any urban junction. As a minimum, a conventional underpass with modified vertical clearance and grade requirements to accommodate small vehicles in the direction of travel having the heavier of the intersecting through movements should be considered. Such design should remove 90% of the traffic in this direction from the signal significantly enhancing signal timings and the overall cycle length for the remaining traffic using the at-grade signal. While the following section will provide a study on what could be considered a “small

vehicle” that maybe allowed in the underpass, accordingly what is the minimum design vertical clearance and maximum grade proposed for the grade separation, figure 2 shows how the design for the proposed “compact underpass” should look like. As normally expected, at major urban signalized junctions additional ROW in the vicinity of the junction is required anyway to accommodate additional left and right turning movement lanes for lengths of the intersecting roads. It is argued that widening to accommodate a “compact underpass” will not require additional ROW for lengths that exceed the same required to provide such turning movement lanes.

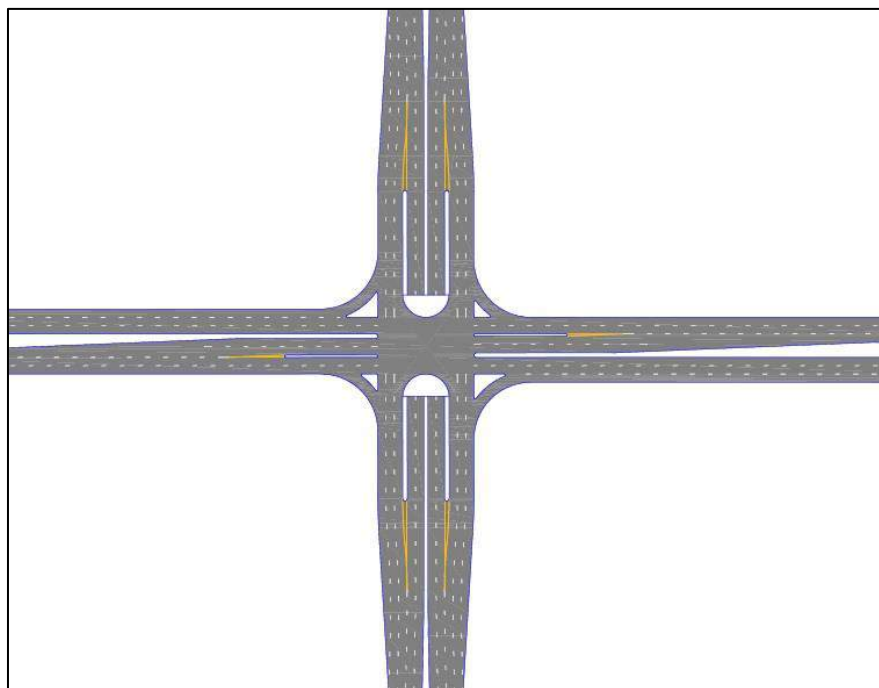


Figure 2: Compact underpass in the direction of the heavier of through movements with clearance and grade requirements to accommodate small vehicles

Vertical Clearance and Grade Requirements to Accommodate Small Vehicles

This section provides an attempt to identify the reasonable vertical clearance and grade requirements that will accommodate 90% of the traffic mix thus shortening as much as practical lengths of retaining structures for underpasses and limiting additional ROW requirements to the same lengths required to accommodate additional turning movement lanes at a conventional urban signalized junction.

Chrest (2011), researched several design codes for multistory car parking garages and concluded that most codes require 7' (2.14 meters) as the minimum height that will accommodate private cars, vans, etc., however added that NFPA, 101 requirements for means of egress systems would require 7' 6" (2.28 meters)

A web search on city bus dimensions used by several transport authorities worldwide identified that the minimum height of a one story bus dimension is 4.3 meters, making it impractical to use it for the aimed compact underpass design, while another search on minibus heights revealed that height of a 30 seater minibus of the type shown in figure 3 and offered by many auto manufacturers ranges between 2.60 meters to 2.75 meters. The internal height of such minibus is in the range of 1.9 meters.



Figure 3: Photo for a standard mini bus offered by many auto manufacturers

The Federal Specification for the star-of-life ambulance (2002) specified the maximum height for ambulance vehicles to be 2.79 meters.

International fire code (2009) identified the minimum vertical clearance for fire trucks to be 4.115 meters which again wouldn't be practical to use if a tangible reduction in vertical clearance is to be offered for the intended compact underpass.

In light of the above, it can be concluded that the reasonable minimum vertical clearance that can accommodate private vehicles, vans, minibuses and ambulance vehicles would range between 2.85 to 3.0 meters.

For the purpose of calculation of structural lengths of retaining walls in this paper a vertical clearance of 3.0 meters will be used. Added to this, a structural slab depth inclusive of pavement of 1.0 meter will be assumed giving a total depth of 4.0 meters.

Looking at grades, it could be argued that the upper range of grades in standards is always avoided in the design of grade separations for the risk that the 10% of vehicle classifications (heavy trucks) will not be able to negotiate it. Looking at AASHTO, for urban collectors the maximum grades for the design speed of 60 KPH are 9%, 10% and 12% for level, rolling and mountainous terrains respectively. For the same road classification AASHTO states that these grades could be increased by 2% if the grade length is less than 150 meters. For Urban Arterials, the maximum grades for 60 KPH speed are 7%, 8% and 10% for level, rolling and mountainous terrains respectively. For the proposed grade separation a grade in the range of 8-10% may be considered reasonable for the intended vehicle classifications having relatively light loads and likely powerful engines.

At 60 KPH speed, the limiting factor for grades for the intended "compact underpass" would be the K-values of the adjoining crest and sag curves to achieve the 4.0 meters deep level-1 grade separation. For a 60 KPH speed the AASHTO K-values are (11 & 18) for crest and sag respectively. While these values are conservative compared to those dictated by several other standards, they will be used in this paper for AASHTO being a recognized standard worldwide, accordingly the required lengths of vertical curves for a 60 KPH may not seem practical. The AASHTO K-values are reduced significantly for a design speed of 40 KPH (4 & 9) for crest and sag respectively. Accordingly, a design speed of 40 KPH can be used for the proposed grade separations which will still result in significant improvement in traffic operations compared to stopping at a signal. In this case, speed reduction measures like rumble strips or similar should be considered at the underpass approaches. Using an 8% grade at a 40 KPH speed provided reasonable structural lengths for underpass retaining structures. The following figures 4, 5 & 6 provide comparisons of lengths of retaining structures for the proposed compact underpass versus a conventional underpass. 1% grade is assumed at level-Zero in the opposite direction as a conservative approach for algebraic difference of grades, hence, the calculation of vertical curve lengths.

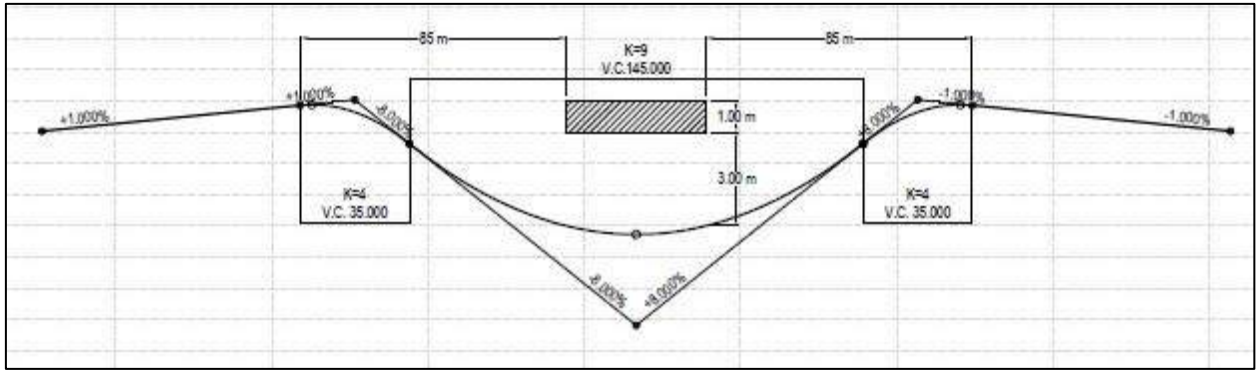


Figure 4: Compact underpass design with 4.0 meters total level difference and 8% grades

Figure 4 shows a compact underpass with a total level difference of 4.0 meters, a design speed of 40 KPH and 8% grades, the structural length required to reach level-Zero is around 85 meters. Such length is quite reasonable compared to the lengths of widening applied to most signalized junctions to add turning movement lanes.

Comparing this to a conventional underpass as shown in figure 5, using 6.5 meters total level difference and 5% grades to accommodate high/ heavy vehicles, the structural lengths are significantly longer requiring around 189 meters.

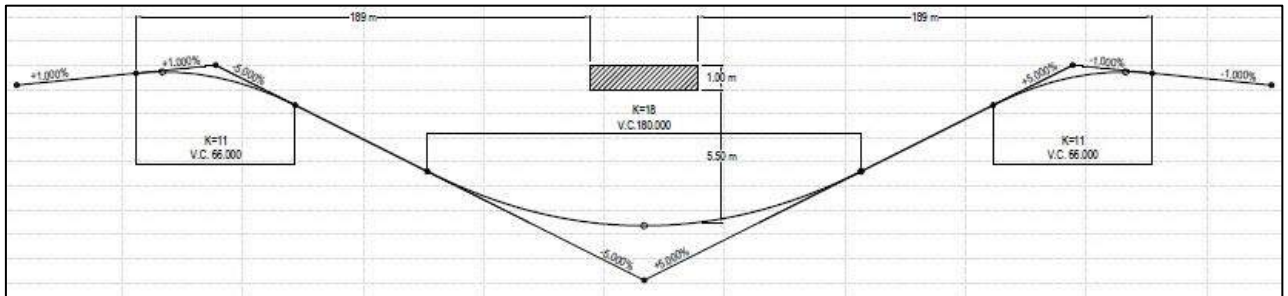


Figure 5: Conventional underpass design with 6.5 meters total level difference and 5% grades

And for the purpose of comparison only, figure 6 shows a grade separation of total level difference 6.5 meters with a grade of 8%, which may not be desirable by most agencies as high/ heavy vehicles will be allowed to use the underpass with potential difficulty for overloaded/ less powerful trucks to negotiate the 8% grade. Such design resulted in a structural length of around 116 meters.

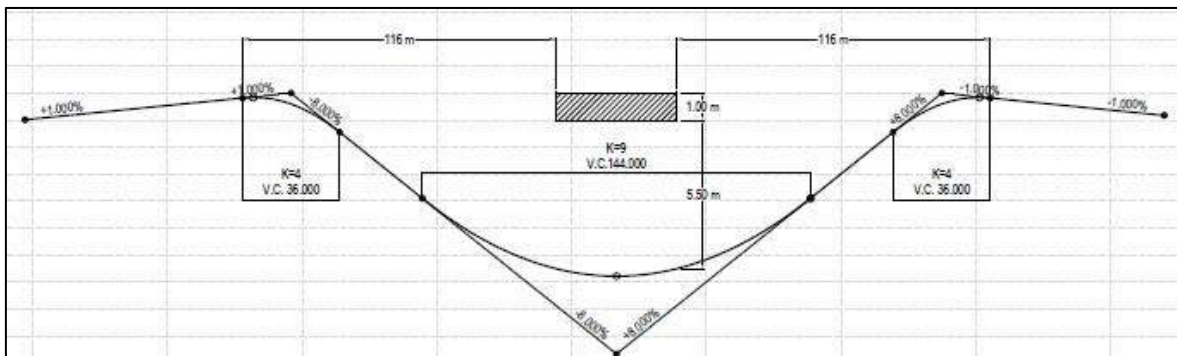


Figure 6: Underpass design with 6.5 meters total level difference and 8% grades

While this paper cannot offer a firm cost comparison between the proposed compact underpass and conventional underpasses as the parameters at each location this design is proposed may vary widely, following are obvious potential advantages compact underpass design may offer over conventional underpass design.

The height of the retaining structure in the covered section of the underpass whether being retaining walls or a trough section if high water table is encountered is less by a minimum of 2.5 meters than conventional underpasses and will result in considerable cost reductions.

The lengths of retaining structures outside the covered section and until level-Zero is reached is significantly less than that of conventional underpasses and will result in significant cost reductions.

The total earthwork volumes for a compact underpass are significantly less resulting in considerable cost reductions.

Additional ROW requirements for compact underpass designs fall in the same limits within the lengths required for widening of at-grade signalized junctions to provide additional turning lengths while additional ROW requirements for conventional underpasses may be required for significantly longer lengths.

Innovative Urban Grade Separations

Figure 7 shows an idea for an innovative urban grade separation proposed by the author. The idea is based on having one of the two intersecting through movements go in a compact type through underpass (Direction A). The through, or left movements or both of the other direction (Direction B) will make a right turn at the junction and will start ramping down to a compact type underpass. At the point where the ramps of the through underpass of direction A are back to level-Zero, the ramped down roads of direction B will make a U-turn under direction A carriageway. The left turn will connect directly at level-1 to the underpass of direction A while the through movement will continue crossing under the full width of direction A and will ramp up again parallel to direction A until it reaches the junction and make another right turn to complete the through movement of direction B. This design can obviously offer big flexibility to traffic engineers based on through and left turning volumes of both directions on which direction is to be taken in a through underpass and which is to be taken in a “U- Turn” underpass. It is argued that this design will literally keep the traffic moving in almost all directions as the only traffic that will be stopped at the signal will be the left turning traffic of one direction only in addition to high/heavy vehicles. The parameters of this design, particularly, sight distance and grades can be adjusted for a design speed of 40 KPH which is still much better than stopping at a signal. The swept path for turning curves could be adjusted to accommodate a city bus (horizontally only).

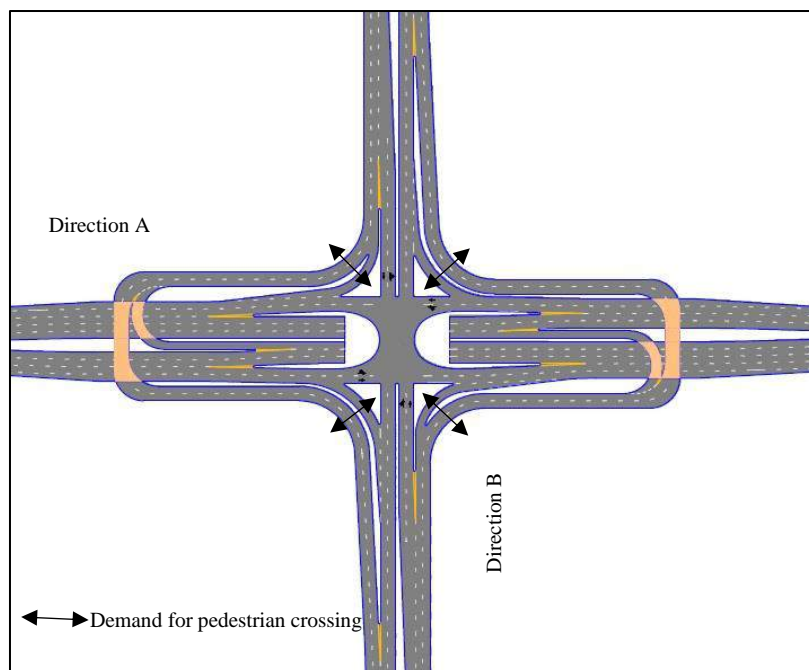


Figure 7: Innovative urban grade separation

One disadvantage of this design maybe the difficulty to achieve safe pedestrian crossing at the four locations identified in figure 7 as they will conflict with the proposed free flow through movement of direction B. In order to achieve safety to pedestrians at these locations one of the following options should be investigated.

- Providing on-demand pedestrian signals at the 4 identified locations in figure 7, which will likely require less stopping time than if vehicles have to stop for the conflicting through and left turning vehicular traffic. However in this option it may be argued that the through direction of direction B will be stopped twice for pedestrian crossing significantly challenging the efficiency of the new design and questioning the value for money for building the proposed U-turn underpasses.
- Providing traffic calming measures upstream of pedestrian crossings with “Give way for Pedestrians” signs. This option will function properly to a certain threshold of through traffic volumes and demand of pedestrian crossing yet may create safety issues for pedestrians and inconvenience to through traffic seeking uninterrupted flow by using the underpasses.
- Providing underground grade-separated pedestrian crossing only from the edge of the intersection to the right turn channelization islands to eliminate the conflict between pedestrians and through (turning right) traffic. Beyond the channelization islands normal at-grade pedestrian crossings could be provided. Although this option involves another additional cost, it has merits for serious consideration to ensure the efficiency of the grade separated lanes, should this design have merits for implementing.

Evaluation Against Recognized Sustainability Standards

Like LEED for buildings, Greenroads is a third party rating system for road projects that recognizes and rewards roadway projects that exceed public expectations for environmental, economic and social performance. It is meant to improve roadway sustainability and provide accountability for sustainability on roadway projects (Muench, Anderson, Hatfield, Koester, & Soderlund et al., 2011).

The Greenroads rating system is a collection of credits, earning points towards those credits would qualify a project for certification. To achieve certification, there are 12 project requirements that are mandatory and 6 categories of credits, each of which has a maximum number of points that can be achieved. Table 6 describes the rating system credits, points and certification categories.

Table 6: Greenroads Rating System

Rating System at a Glance		
Category Name	Credits	Points
Project Requirements (PR)	12	0
Environment & Water (EW)	10	30
Construction Activities (CA)	11	20
Materials & Design (MD)	6	24
Utilities & Controls (UC)	8	20
Access & Livability (AL)	10	21
Creativity & Effort (CE)	4	15
Total Main Categories	57	115
Total w/ CE	61	130
Certification Thresholds	PRs	Points
Bronze	All 12	40
Silver	All 12	50
Gold	All 12	60
Evergreen	All 12	80

Source: Greenroads Manual Version 2

The author made a hypothetical assessment if a normal at-grade signalized junction is to be converted to a compact grade separation by adopting the ideas provided in this paper, how many points such a project could obtain only by adopting these ideas assuming that mandatory project requirements which are all about having certain studies, reports and plans in place like ecological impact analysis, energy and carbon footprint assessment, social impact analysis, quality control plan, etc., will be met and that the project could complete certification requirements from additional areas like recycling, drainage solutions etc.,

Table 7 provides a list of categories and points that could be achieved against each category. As shown, a total of up to 17 points can be achieved only by adopting ideas provided in this paper.

Table 7: Points that could be achieved towards Greenroads certification from ideas in this paper

Category	Credits	Points that maybe achieved
Utility and Control	Traffic Emission Reduction	Up to 3 points
	Travel Time Reduction	Up to 2 points
Access and Livability	Safety Enhancements	Up to 2 points
	Health Impact Analysis	Up to 2 points
	Noise and Glare Reduction	Up to 3 points
Creativity and Effort	Innovative Ideas	Up to 5 points

Possible Disadvantages of Compact Underpasses and Proposed Mitigations

In order to have a complete assessment of the viability of compact underpass design, it is prudent to also investigate its possible disadvantages and check if possible mitigations can be provided or not. Following is an initial assessment of possible disadvantages and possible mitigations. This is of course in addition to what may be looked at as a disadvantage of adding structures discussed at earlier sections of this paper.

Safety: In the onset of adopting the new design by any agency, many high vehicles may enter the underpass by mistake resulting in major risk of hitting the underpass roof slab. This could be mitigated by planned publicity efforts in different media sources for the new compact underpasses, their aims, benefits and allowed users. Also physical gantries, over height detectors and ITS warning systems should be installed at the underpass entries at least for the first few ones constructed.

Access to fire trucks: Fire trucks may be required to access the underpasses to deal with an emergency inside the underpass itself or to bypass the signal to shorten the time to their destination. Increasing the proposed underpass clearance to accommodate fire trucks will discredit the value of compact underpasses as it may require dimensions and lengths that are equal or close to conventional grade separation dimensions.

For an emergency in the underpass itself, the length of the covered section of the underpass should not be longer than what could be reached by the fire truck fire hose from both directions. For an underpass that crosses under 10 lanes inclusive of physical separators the total length of the underpass will never exceed 50 meters which is indeed reachable by fire hoses from both directions. It should also be assumed that NFPA,502-2011 Edition requirement of having hose connections placed so that no location on the road is more than 45 meters from the hose and that hose spacing does not exceed 85 meters is achieved with or without having a compact underpass on the road.

For the problem of inaccessibility of the fire truck to the underpass for the purpose of bypassing the signal, considering the significant amount of traffic removed from the at-grade signal, the signal LOS should experience a remarkable improvement. Additionally, part of the innovative design should include ITS systems that grant immediate right of way to emergency vehicles through traffic signal preemption.

Additional drainage requirements: Introducing underground traffic lanes at road junctions to replace some of the at-grade lanes will of course involve cost of additional drainage measures like pumping, additional piping and/or connection to nearby deep storm water tunnels. As for the cost of structures, the additional cost is countered by the multiple mobility, safety and sustainability benefits offered by providing these underground lanes.

Possible Punishment to shared transport commuters: Some may argue that the new design could be considered a “punishment” to shared transport commuters, particularly on single or double decker public buses which will not be allowed access to the underpasses and this may discourage public transport which is against sustainability goals that always push in the direction of leveraging public transport.

The answer can simply be that in a conventional signalized junction, public one story or double decker busses are going to stop at the signal anyway, however as mentioned earlier, the signal in the new design should have remarkable improvement in LOS. Additionally, public busses may have a designated stop in the vicinity of the junction which regardless of clearance restriction will not allow them to access the underpass.

ITS solutions may also be provided for buses. One option may be a Transit Signal Priority (TSP) equipment that could be added to the busses and traffic signals to give priority, and provide as much as practical green wave for busses (Smith et al, 2005).

Another possible mitigation is to encourage carpooling. One or more of the underground lanes may be dedicated to high occupancy vehicles (HOV). In fact, all underground lanes may be assigned as HOV lanes however such option should be extensively studied by each road agency on a case by case basis as this will prevent all single occupancy vehicles (SOV) from using the underpasses and will force them to use the signal which may significantly reduce the efficiency of the new design.

Conclusion

This paper introduced general thoughts on the possible advantages of providing “compact underpasses” that accommodate “small vehicles” which are argued to constitute around 90% of the traffic mix in most signalized junctions. Although “Compact Underpass Design” has the additional cost of structures and drainage, it significantly contributes to reducing congestion at signals which in addition to its negative impacts has its enormous cost as well. The implementation of ideas provided in this paper whether being a compact through underpass in one direction or the innovative design of a compact through underpass in one direction and U-turn compact underpass in the other direction will indeed be subject to thorough feasibility studies by transport agencies that see merits in such ideas on a case by case basis depending on actual conditions at locations such designs maybe proposed. The author would like to close this paper with a question to bus manufacturers, should bigger busses are to be granted access to the compact underpasses. What is the possibility of manufacturing city busses with the same horizontal dimensions while maintaining the external height of minibuses? And if the problem is providing heights that suit all standees, what is average percentage of passengers that are taller than the minibus internal height of 1.9 meters among commuters? And is it possible to apply the same idea in this paper of maintaining 1.9 meters for the dominant bus standees and allowing priority seats like those provided for elderly and disabled for the small percentage of commuters that maybe taller than 1.9 meters?

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PAPER TITLE	A New Approach for Estimating Pavement Rutting Progression		
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KEYWORDS:

Rutting progression, Deterioration model, Multilevel model, Time series data, Pavement management, Model validation

ABSTRACT:

Every road agency responsible for pavement maintenance activities faces the problem of insufficient funding to perform all required repairs. Accurate prediction models of pavement performance help them in assessing the implications of optimum maintenance and rehabilitation strategies so that practical decisions can be made. The current study aims at delivering the application of a new approach for the development of pavement deterioration models for managing a rural arterial network of flexible pavements. The estimation of pavement rutting progression has been based on longitudinal datasets that contain observations on the condition of a large volume of pavement sections. The study demonstrates how to prepare accurate datasets for light and heavy duty pavements with a wide range of pavement rut depths, traffic volumes, pavement strength and environmental conditions. The study presents hierarchical linear models that can account for the correlation among time series data on the same section and capture the effects of unobserved factors. The contribution and significance of contributing factors in predicting rutting progression are presented and discussed.

A New Approach for Estimating Pavement Rutting Progression

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1 ABSTRACT

Every road agency responsible for pavement maintenance activities faces the problem of insufficient funding to perform all required repairs. Accurate prediction models of pavement performance help them in assessing the implications of optimum maintenance and rehabilitation strategies so that practical decisions can be made. The current study aims at delivering the application of a new approach for the development of pavement deterioration models for managing a rural arterial network of flexible pavements. The estimation of pavement rutting progression has been based on longitudinal datasets that contain observations on the condition of a large volume of pavement sections. The study demonstrates how to prepare accurate datasets for light and heavy duty pavements with a wide range of pavement rut depths, traffic volumes, pavement strength and environmental conditions. The study presents hierarchical linear models that can account for the correlation among time series data on the same section and capture the effects of unobserved factors. The contribution and significance of contributing factors in predicting rutting progression are presented and discussed.

2 INTRODUCTION

Rutting is described as depression in the transverse profile of the road pavement surface (Haas et al. 1994). It arises from a permanent deformation in pavement layers and/or wearing courses or in the subgrade, which is caused by either consolidation or lateral movement of pavement materials. Permanent deformation or rutting of flexible pavements has a major impact on structural and surface performance of pavement under different conditions. It reduces the useful service life of the pavement as it affects pavement integrity and produces safety hazard for highway users. It affects vehicle handling characteristics and the accumulation of rainwater in ruts leads to hydroplaning. According to Freeme (1983), there are three phases for the development of rutting and includes initial densification or bedding-in phase, gradual deterioration phase and accelerating deformation phase.

The resistance of pavement structure to permanent deformation is dependent on a number of parameters which include one or more of the following or the combination of all: traffic loads, pavement structure, environment and construction process. These factors influence the modelling of performance prediction at different rates through their effects on the initiation and progression phases of various pavement distress modes (Toole et al. 2009). Deterioration models allow road managers to assess current condition and to predict future conditions of their pavement networks. Once accurate prediction models are identified, then the implications of optimum maintenance activities and rehabilitation strategies can be assessed and practical decisions made.

3 OVERVIEW

Many road agencies around the world are facing complex decisions about highways maintenance, repair and rehabilitation in the most cost efficient and effective ways. Therefore, they need to know how pavement condition will change over time by using accurate deterioration models to maintain high levels of service. This change in pavement condition can be due to observed variables or unobserved variables (Hong 2007). The former can be captured by including identified causal factors in the deterioration models; however, the latter arises from factors beyond the already identified causal factors.

A wide range of factors can contribute to rutting progression. These factors generally include traffic loading, climate condition, pavement strength, road type and geometry, maintenance practice, pavement type and construction quality. Actually, database for any network is unable to capture all possible factors affecting pavement deterioration process and thereby some critical factors could not be incorporated in models for predicting pavement deterioration. To overcome this problem, it is required to develop models that are able to account for the effect of unobserved factors (i.e. unobserved heterogeneity in pavement condition).

The heterogeneity between one pavement section and another may be due to variances in construction quality, climate condition and maintenance activities applied in different ways at different times. There are also differences between the highways from which the sections are extracted which may be caused by variations in materials properties. Also, there are variances between pavement classes, due to factors such as class duty function (light or heavy duty pavement). Therefore the data structure is hierarchical with four levels of variation within any network. Time series observations (level-1) are nested within sections (level-2), which are nested within highways (level-3), which are nested within classes (level-4). The development of a network pavement deterioration model must allow for variation at all these four levels. An approach to capture the effect of variance at multilevel is presented in the current study. The dataset that has been used in this study was selected from four types of road classes of the rural arterial network of the State of Victoria, Australia.

4 STUDY OBJECTIVES

This study aims at delivering the application of a new approach for the development of rutting progression model for flexible pavements of rural arterial network. The estimation of pavement rutting progression has been based on longitudinal datasets that contain observations on the condition a large volume of pavement sections. The study aims at incorporating the effect of unobserved factors in the developed models to increase the models' robustness. In addition, the study has been focused on assessing the effects and contributions of the different relevant factors on pavement rutting progression, simulating the developed models to understand their performance, and validating the models to ensure their ability to predict future conditions accurately.

5 DESCRIPTION OF SELECTED NETWORK

Representative samples of highways from Victoria's rural arterial network are considered. The selected sample network is from 40 highways with a combined length of more than 2,300 km. The coverage of the network sites in terms of relevant modelling parameters is presented below:

- ❖ The flexible pavement of all sections have granular bases and sub-bases with single or double coat spray/chip seals but have different thicknesses reflecting the different traffic volumes to which they are subjected.
- ❖ Different climatic zones covering different ranges of Thornthwaite Moisture Index (TMI).
- ❖ Drainage condition in terms of good or poor. As all roads considered herein are rural, their drainage systems consist of table drains or ditches with sealed (fully or partially) or unsealed shoulders.
- ❖ Four types of road classes (M, A, B and C) with different duties, geometric standards, and traffic volumes. Road class (M) represents freeways and motorways, road class (A) represents major arterials, road class (B) represents main arterials which are state highways connecting major cities, and class (C) roads represent minor arterials which are rural roads connecting smaller towns (VicRoads 2015). Typically, class M and A roads are classified as heavy duty pavements and class B and C roads are classified as light duty pavements. The selected network includes 7 road sites of class M, 11 road sites of class A, 10 road sites of class B, and 12 road sites of class C.

6 DATA DESCRIPTION AND PREPARATION

Model formula can be used effectively in pavement management system when it depends on variables that can be easily measured or collected at network level. Therefore, based on the availability of information for the sample network, time series data for the variables considered in this study include rutting as the dependent variable (DV) and traffic loading, climate, pavement strength, and drainage condition as the independent variables. Time series datasets have been extracted from relevant road authority's databases to address the study objectives. Time series data (2004 to 2011) for each 100m-section were collected from a minimum of four surveys for the following variables:

Rutting: In Victoria, rut depth data is collected using a multi-laser profilometer and considering the maximum depth under a 1.2 m simulated straight-edge center at both wheel paths method (Moffatt & Hassan 2007). The rutting data used was in terms of average rut depth (RD, mm) in both wheel paths (outer and inner) for every 100m section.

Traffic loading: Traffic volume data in terms of number of Heavy Vehicles (HV) for the selected sections was extracted from relevant database for the relevant years. Estimates of traffic data for missing years were obtained for each highway by using the average growth factor for all its segments over the period for which data is available. HV numbers at the time of construction for each section were estimated using current HV, section age and average growth rate (Equation 1). This data was then used to determine cumulative traffic loading in terms of million equivalent standard axles (MESA) using Equation 2 (Jameson 2012) in conjunction with relevant parameters from VicRoads' code of practice (VicRoads 2013). MESA was determined for each year condition data is available, for each section. The cumulative growth factor, in each year for which condition data was available, was calculated using Equation 3.

$$HV_{at\ const} = HV_{current} / [(1 + GF)^{Age\ at\ current\ HV\ year}] \quad (1)$$

Where:

$HV_{at\ const}$ = number of heavy vehicles at time of construction.

$HV_{current}$ = number of heavy vehicles in any year of actual traffic is available.

GF = average annual growth rate of heavy vehicles.

Age = pavement age.

$$MESA = [365 * HV_{at\ const} * DF * LDF * CGF * NHVAG * (ESA/HVAG)]/10^6 \quad (2)$$

Where:

MESA = cumulative ESA (equivalent standard axle loads) from construction time to any year of condition data.

DF = direction factor =1, (proportion of the two-way HV travelling in the direction of the design lane).

LDF = lane distribution factor, (proportion of heavy vehicles in design lane) =1 (VicRoads 2013). According to VicRoads' code of practice document (VicRoads 2013); the LDF value is considered as 1 when the number of road lanes is less than 3 in one direction. Class M roads have two lanes in each direction, whereas class A, B, and C roads have one lane in each direction.

$$\text{Cumulative Growth Factor (CGF)} = [(1 + 0.01 * GF)^{Age} - 1] / (0.01 * GF) \quad (3)$$

NHVAG = average number of axle groups per heavy vehicle = 3.1 (VicRoads 2013).

ESA/HVAG = average ESA per heavy vehicle axle group = 0.82 for class M and A and 0.66 for class B and C (VicRoads 2013).

Pavement strength: Insufficient pavement strength or deformation and displacement in the upper layers of pavement can contribute to pavement deterioration (Paterson 1987). As pavement deflection data for the selected sections were not available to calculate pavement structural number, the following steps were used to estimate the modified structural number (SNC_i) at different ages:

- ❖ Cumulative traffic loading data at design life ($MESA_{DL}$) was determined using Equation 2 and relevant parameters. Cumulative Growth Factor (CGF_{DL}) over the design life (DL) was estimated using Equation 3. According to VicRoads (2013), the design life for class A, B and C roads is 20 years and for class M roads is 30 years.

- ❖ Initial value of the structural number (SNC_0) at time of pavement construction (Age = 0) was calculated using the following expression (Equation 4) derived from NAASRA (1979) and Hodges et al. (1975) (cited in Chen & Martin 2012) This expression is based on the cumulative traffic loading ($MESA_{DL}$) that is expected over pavement design life.

$$SNC_0 = 0.55 * \text{Log}_{10}(MESA_{DL} / 120 * 10^6) + 0.6 \quad (4)$$

- ❖ The modified structural number (SNC_i) at any age (i) was estimated from SNC_0 , pavement age (Age) and design life (DL), using the following relationship (Equation 5) (Martin 2008).

$$SNC_i = SNC_0 * (2 - \text{EXP}(0.33 * Age / DL)) \quad (5)$$

Climate condition: Thornthwaite Moisture Index (TMI) deals with engineering applications that lie on or beneath the ground surface, such as road pavements (Byrne & Aguiar 2010). It is defined as the combination of annual effects of precipitation, moisture deficit, evapotranspiration, soil water storage and runoff (Thornthwaite 1948). Historical climate time series data in terms of TMI was extracted from the climate extraction tool developed by Byrne and Aguiar (2010). It is provided as an Excel database which uses latitude and longitude values to access relevant data over time for each 100m road section. TMI values were extracted along all highway sections from 2004 to 2011. Generally, a positive sign of TMI refers to a wet area while a negative sign of TMI refers to a dry area.

Drainage condition: It has been identified as an important factor for both the functional and structural performance of road pavement (Pearson 2012). The condition of drainage system for the selected sections was extracted from relevant database; rated as good or poor.

Maintenance activities: The prediction model should account for the maintenance activities which affect the condition and the rate of deterioration, which could be in a positive or a negative way. However, this influence should be removed from the model, if information on maintenance activities is not available or not accurate (AASHTO 2001). In this study, the effects of periodic maintenance works were removed for this reason. Sections that were subjected to maintenance over the study period (relevant survey years) were removed using the Linear Rate of Progression (LRP) tool (Martin & Hoque 2006). The output of this tool includes the sections (100m) with positive progression only together with their progression rates. As a result, a significant number of sections were excluded from the datasets.

As mentioned earlier in this paper, the three phases of rutting development are initial, gradual and rapid deterioration phases (Freeme 1983). In this study, rutting progression is modelled during the gradual phase only. The initial phase considers the first deterioration in road pavement after construction. Based on typical construction standards, the initial surface condition of a new pavement can be assumed (Toole et al. 2009). For Victoria’s rural road classes, the initial rut depth can be assumed to be 1mm for class M roads and 2mm for A, B and C road classes (Toole et al. 2004). According to Smith et al. (1996), the transition from gradual deterioration phase to the rapid deterioration phase is limited for each road class by their terminal rutting values. The terminal rutting value is considered to be 20mm for M and A road classes and 25mm for B and C road classes (Smith et al. 1996).

In the current study, the initial and terminal condition values mentioned above were used to establish boundary limits for the gradual deterioration phase. Hence, all sections with rutting values within the initial phase (below mentioned values) were removed to ensure pavement deterioration had passed the initial phase and entered the gradual phase. Also sections with rutting data that have passed the terminal condition values (above mentioned values) were removed to ensure pavement deterioration did not enter the rapid phase.

Due to the variation in chainages between different years, rutting data was aligned and adjusted by checking the start and end chainages of each 100m segment over consecutive survey years and ensuring that they have the same distance to fixed closeby reference points. After available data for all relevant variables were extracted from different databases, for each 100m segment, the chainages of rutting data (dependent variable) were treated as the base and chainages of data related to contributing factors (independent variables) were matched to them for all relevant years. Good and Hardin (2003) recommended that one-fourth to one-third of the data should be set aside for validation purposes. Therefore, random dataset split was utilized to divide the dataset (for each road class) into two parts; approximately 70% of the data was used for model development and the remaining 30% of the data was used for model validation. Statistics of variables used for developing rutting progression model are presented in Table 1.

Table 1. Statistics for variables used for developing rutting progression model within gradual phase

Statistics	Rutting (mm)	Traffic loading (MESA)	Pavement strength (SNC _i)	Climate (TMI)	Age (years)
Mean	6	2.992	2.543	1	10
Minimum	1	0.010	1.224	-38	1
Maximum	25	36.220	3.725	100	47

7 MODELLING USING MULTILEVEL ANALYSIS

As mentioned before, there are four levels of variance within the existing panel dataset for the sample network and nesting of these data is required. The Hierarchical Linear Modelling (HLM) approach handles models with datasets that have at most a four level nested structure (Raudenbush et al. 2011). Theoretically, the effect of variance (heterogeneity) can be captured by allowing randomness over the model parameter(s) (Raudenbush & Bryk 2002, Greene 2004, Field 2009). A study conducted by Hong (2007)

showed that unobserved heterogeneity could potentially be accounted for through the intercept and other regression parameters (i.e. slopes). In this study, the random parameter approach is used by allowing the intercepts to vary at level-2, level-3 and level-4, and the slope of time factor to vary at level-2.

If Y is a measure of pavement rutting (RD) and X is a time factor, the pavement rutting progression in a multilevel model with random intercepts and random slope for the time factor is proposed as:

$$\begin{aligned}
 \text{Level-1: } & \text{RD} = \beta_0 + \beta_1 * \text{Time} + e \\
 \text{Level-2: } & \beta_0 = \beta_{00} + r_0 \\
 & \beta_1 = \beta_{10} + r_1 \\
 \text{Level-3: } & \beta_{00} = \beta_{000} + u_{00} \\
 \text{Level-4: } & \beta_{000} = \beta_{0000} + v_{000} \\
 \text{The final mixed model is:} \\
 \text{RD} = & \beta_{0000} + \beta_{10} * \text{Time} + \text{Time} * r_1 + e + r_0 + u_{00} + v_{000} \quad (6)
 \end{aligned}$$

Where:

RD: is the predicted rutting progression value in terms of RD (mm), (dependent variable).

Time: is the time variable in years, (independent variable).

e: is the error value (random variable).

β_0 and β_1 : are the fixed and unknown coefficients, where β_0 is the intercept and β_1 is the slope.

r_0 and r_1 : are the level-2 random effects and u_{00} : is the level-3 random effect.

β_{00} and β_{10} : are the level-2 fixed coefficients and β_{000} and β_{0000} : are the level-3 and level-4 fixed coefficients, respectively.

In equation (6), only time is included as a predictor in the model. The model estimates the average growth occurring throughout the year and it could be referred to as a growth model. By incorporating all available variables that are considered in this study, the above multilevel models can be extended by including variables that vary over time within each section (traffic loading, pavement strength and climate) at level-1 and variables that vary from one section to another (drainage condition) in level-2. They are expressed as:

$$\begin{aligned}
 \text{Level-1: } & \text{RD} = \beta_0 + \beta_1 * \text{Time} + \beta_2 * \text{MESA} + \beta_3 * \text{SNC}_i + \beta_4 * \text{TMI} + e \\
 \text{Level-2: } & \beta_0 = \beta_{00} + \beta_{01} * \text{DR} + r_0 \\
 & \beta_1 = \beta_{10} + r_1 \\
 \text{Level-3: } & \beta_{00} = \beta_{000} + u_{00} \\
 \text{Level-4: } & \beta_{000} = \beta_{0000} + v_{000} \\
 \text{The final mixed model is:} \\
 \text{Y} = & \beta_{0000} + \beta_{10} * \text{Time} + \beta_2 * \text{MESA} + \beta_3 * \text{SNC}_i + \beta_4 * \text{TMI} + \beta_{01} * \text{DR} + \text{Time} * r_1 + e + r_0 + u_{00} + v_{000} \quad (7)
 \end{aligned}$$

Where:

MESA: is the traffic loading variable in terms of Million Equivalent Standard Axles load /lane.

TMI: is the climate condition variable in terms of Thornthwaite Moisture Index.

SNC_i : is the pavement strength variable at time (i), in terms of modified structural number.

DR: is the drainage condition variable (good = 0 and poor = 1).

All other variables are as defined previously.

The final mixed model in equation (7) that incorporates all possible available variables is referred to as a conditional model. β_{0000} , β_{000} , β_{10} , β_2 , β_3 , β_4 and β_{01} are fixed effect parameters, whereas, e , r_1 , r_0 , u_{00} and v_{000} are random variables.

To develop the rutting progression model for the sample network, three types of models were fitted, null model, growth model and conditional model; details of these models are presented below:

1) Null model: This model predicts the outcome variable with no specified predictors (only intercept).

The null model should be created first as a primary step in a hierarchical data analysis for the following purposes (Raudenbush & Bryk 2002, Field 2009, Garson 2013):

- ❖ To provide an estimate of the grand mean of rutting value within gradual deterioration phase for the selected network.
- ❖ To use as a baseline model for model comparisons when adding predictors to the model, based on a deviance statistic test.
- ❖ To estimate the proportion of variance at each level in the dataset used to predict rutting progression and to test whether multilevel modelling is needed. The proportion of variance could be estimated using the following formulas (Raudenbush & Bryk 2002):

$$\text{a) Proportion of variance within level-1, } \text{PVO} = \text{Ve} / (\text{Ve} + \text{Vr}_0 + \text{Vu}_{00} + \text{Vv}_{000}) \quad (8)$$

$$\text{b) Proportion of variance within level-2, } \text{PVS} = \text{Vr}_0 / (\text{Ve} + \text{Vr}_0 + \text{Vu}_{00} + \text{Vv}_{000}) \quad (9)$$

$$c) \text{ Proportion of variance within level-3, } PVH = Vu_{00} / (Ve + Vr_0 + Vu_{00} + Vv_{000}) \quad (10)$$

$$d) \text{ Proportion of variance within level-4, } PVC = Vv_{000} / (Ve + Vr_0 + Vu_{00} + Vv_{000}) \quad (11)$$

Where:

Ve : is the variance of level-1 random variable

Vr_0 : is the variance of level-2 random variable

Vu_{00} : is the variance of level-3 random variable

Vv_{000} : is the variance of level-4 random variable

- 2) Growth model: This model predicts rutting progression as a function of time variable to study the progression rate over time. As time is the most important factor in time series data, the growth model is estimated with only time as a predictor with the intercept and slope regarded as random. The model estimates the average growth of rutting per year.
- 3) Conditional model: In this model, available independent variables are added to the growth model as predictors. A backward variable selection procedure has been followed, in which all predictors are added to the model simultaneously and evaluated together. Then, any non-significant fixed effects are removed one at a time to determine which variables to include in the final model.

With any regression model, the first step is to assess the normality of the included variables (Garson, 2013). In this study dataset, it was observed that the rutting variable (RD) and traffic loading (MESA) variable were positively skewed. Therefore, these variables were transformed using suitable transformation function to minimise skewness. The natural logarithm of RD (LN (RD)) and natural logarithm of MESA (LN (MESA)) were found to be the most appropriate transformation functions. In order to remove bias caused by fitting the model to the log transformed rutting data, predictions of rutting are multiplied by the correction factor (CF), which is based on level-1 variance (Ve) and expressed as (Stow et al. 2006):

$$CF = \text{Exp} (Ve / 2) \quad (12)$$

8 DEVELOP RUTTING PROGRESSION MODELS

Multilevel analysis was used to develop empirical deterministic models to predict pavement rutting progression over time as a function of a number of contributing variables using full maximum likelihood estimation. The analysis was performed using Hierarchical Linear Modelling (HLM7) software (HLM7 2015) and Statistical Package for Social Sciences software (SPSS 2015). A four level model is utilized to find the results of the three fitted models which are presented below:

- 1) Rutting null model: The results of the fixed and random effects parameters for the rutting null model are shown in Table 2. The final mixed model is: $\text{LN} (RD) = \beta_{0000} + e + r_0 + u_{00} + v_{000}$

$$\text{Estimated rutting null model is: } \text{LN} (RD) = 1.6334 \quad (13)$$

Where:

LN (RD): is the natural logarithm of rutting (RD).

Based on Ve (0.0833) for the null model, the CF is 1.043 ($\text{Exp} (0.0833/2)$). The null model in equation (13) estimated that the rutting (RD) grand mean value for the network sample was 5.34 mm ($\text{Exp} (1.6334)*1.043$). The four variance components (Ve , Vr_0 , Vu_{00} and Vv_{000}) were statistically significant ($p < 0.001$). Based on equations 8, 9, 10 and 11, the proportions of variances at each level were calculated using variance components (for random effect variables) from the null model results, as shown below:

- a. Proportion of variance within time series observations (level-1) is: $PVO = 34\%$
- b. Proportion of variance among sections within highways (level-2) is: $PVS = 54\%$
- c. Proportion of variance among highways (level-3) is: $PVH = 7\%$
- d. Proportion of variance among classes (level-4) is: $PVC = 5\%$

These results indicate that there is a high variance among sections within highways ($PVS = 54\%$) and a high variance within time series observation ($PVC = 34\%$). Also, 7% of the variance was found among highways, and around 5% among classes. These results indicate that there is a significant variance between observations, sections, highways and classes for rutting condition variable. This confirms that there is statistical justification for using multilevel analysis approach rather than depending on traditional regression analysis to produce a rutting progression model, by capturing the variance between levels correctly.

2) Rutting growth model: The results of rutting growth model are shown in Table (2). Including only the time predictor, the final growth mixed model is: $LN(RD) = \beta_{0000} + \beta_{10} * Time + Time * r_1 + e + r_0 + u_{00} + v_{000}$.

$$\text{Estimated rutting growth model is: } LN(RD) = 1.3175 + 0.0841 * Time \quad (14)$$

With all variables as defined previously.

This growth model is statistically significant with $R^2 = 47\%$ (i.e. 47% of explainable variance is accounted for by the predictor (time) in the model). The rutting growth model suggested that for each additional year, the log RD increased by 0.0841 mm. On average, the RD value increases by 8.77% $[(EXP(0.0841) - 1) * 100\%]$ for every additional year. Based on $Ve(0.0444)$, the correction factor (CF) for the growth model is 1.022 $(Exp(0.0444/2))$ which must be applied to the rutting predictions. For example, if $Time = 12$ years then the predicted $RD = [Exp(1.3175 + 0.0841 * 12)] * 1.022 = 10.47$ mm.

3) Rutting conditional model: The results of the fixed and random effects parameters for this model are shown in Table 2. The final developed rutting progression model as a function of the available contributing variables is presented below:

$$LN(RD) = \beta_{0000} + \beta_{10} * Time + \beta_2 * LN(MESA) + \beta_{01} * SNC_i + \beta_{02} * TMI + Time * r_1 + e + r_0 + u_{00} + v_{000}$$

Estimated conditional rutting progression model is:

$$LN(RD) = 2.3652 + 0.0561 * Time + 0.0503 * LN(MESA) - 0.3795 * SNC_i + 0.0014 * TMI \quad (15)$$

With all variables as defined previously.

The model is statistically significant with $R^2 = 51\%$ (i.e. 51% of explainable variance is accounted for by the predictors in the conditional model). The p -values in Table 2 for the Likelihood Chi-Square test show that the variables time, MESA, SNC_i and TMI have significantly influenced pavement rutting. Also, there are significant variance components within the random effects variables. However, drainage condition (DR) is not significant ($p > 0.05$), hence excluded from the model.

The model indicates that MESA and time are positively related to rutting progression (i.e. the log RD increases with time and MESA), whereas SNC_i is negatively related to rutting progression (i.e. rutting increases with decreases in SNC_i). The positive sign of TMI indicates that pavement in wet zones experience higher rutting progression than in dry zones.

The correction factor for predicting rutting value from equation (15) is 1.021 $(Exp(0.0406/2))$. The t -ratios are the regression coefficients divided by their standard errors and their absolute values represent the effect size of each predictor. The t -ratios suggest that the effect of time is stronger than SNC_i and effect of SNC_i is stronger than the effect of traffic loading on rutting progression. However, the effect of climate is limited compared to the other variables. The effect of each factor on rutting progression from the conditional model can be explained as follows:

- On average, for every additional year, the rutting value increases by 5.77% $=[(EXP(0.0561) - 1) * 100\%]$, when controlling all other variables in the model.
- For a one percent increase in MESA, a 0.05% $=[((1.01) ^ (0.0503)) - 1] * 100]$ increase in rutting value is expected, when all other variables are held constant.
- For a decrease of one SNC_i unit, about 31.58% $=[(1 - EXP(-0.3795)) * 100\%]$ increase in rutting value is expected, when controlling all other variables.
- On average, rutting value will be 0.14% $=[(EXP(0.0014) - 1) * 100\%]$ higher for pavements in wet climate than for pavements in dry climate.

9 MODEL ASSESSMENTS AND VALIDATIONS

Assessments:

To compare two nested models, the deviance statistic test which is based on the maximum likelihood estimation procedure is used (Anderson 2012). In this study, in order to assess the developed models, the null model is compared with the growth and conditional models to determine if the independent variable(s) improve the fit of the model or not. The difference between the deviances for any two models follows an approximate chi-squared distribution with degrees of freedom computed as the difference of the models'

degrees of freedom. The greater the reduction in the deviance value, the greater the improvement in fit. The significant p -values for chi-square (χ^2) test indicate that there is evidence in the data to suggest an association between rutting progression (RD) and the included IV(s). This indicates that the model fit is significantly improved by adding these variables (Garson 2013). Table (3) shows the results of the deviance test for predicted rutting. In this table, it should be noticed that there is always a reduction in deviance from the null model to the growth model and from growth to conditional model. The likelihood ratio tests show that these changes are both significant at $p < 0.001$. These results indicate that the time variable in the growth model and all IVs in the conditional model have improved the models' descriptions of the data.

Table 2. Estimation of the fixed effect variables and variance components for the rutting dataset within gradual phase

Rutting null model	Fixed effect variable	Coefficient	Standard error	t -ratio	Degree of freedom (df)	p -value*
	<i>Intercept</i>	1.6334	0.0223	73.23	3	<0.001
	Random effect variable	Standard deviation	Variance component (V)	Degree of freedom (df)	Chi-Statistic (χ^2)	p -value*
	e	0.2887	0.0833			
	r_0	0.3637	0.1323	10933	76781.62.52	<0.001
	u_{00}	0.1333	0.0178	35	937.08	<0.001
	v_{000}	0.1056	0.0123	3	20.60	<0.001
Rutting growth model	Fixed effect variable	Coefficient	Standard error	t -ratio	Degree of freedom (df)	p -value*
	<i>Intercept</i>	1.3175	0.0210	62.73	3	<0.001
	<i>Time</i>	0.0841	0.0006	152.08	10933	<0.001
	Random effect variable	Standard deviation	Variance component (V)	Degree of freedom (df)	Chi-Statistic (χ^2)	p -value*
	e	0.2108	0.0444			
	r_0	0.4415	0.1949	10933	52248.89	<0.001
	r_1	0.0172	0.0003	10977	12576.87	<0.001
u_{00}	0.1249	0.0156	35	920.34	<0.001	
v_{000}	0.1054	0.0215	3	20.03	<0.001	
Rutting conditional model	Fixed effect variable	Coefficient	Standard error	t -ratio	Degree of freedom (df)	p -value*
	<i>Intercept</i>	2.3652	0.0701	33.72	3	<0.001
	<i>Time</i>	0.0561	0.0011	50.13	10933	<0.001
	<i>LN(MESA)</i>	0.0503	0.0051	9.93	19653	<0.001
	<i>SNCi</i>	-0.3795	0.0122	-31.14	19653	<0.001
	<i>TMI</i>	0.0014	0.0005	2.91	19653	0.004
	Random effect variable	Standard deviation	Variance component (V)	Degree of freedom (df)	Chi-Statistic (χ^2)	p -value*
	e	0.2064	0.0406			
	r_0	0.4146	0.1719	10933	48222.08	<0.001
	r_1	0.0266	0.0007	10977	13144.53	<0.001
u_{00}	0.1567	0.0246	35	1453.91	<0.001	
v_{000}	0.1111	0.0123	3	21.33	<0.001	

Dependent variable is LN (RD)

* All predictors were statistically significant ($p < 0.05$) with 95% level of confidence

Table 3. Deviance test results for predicted rutting progression models

Predicted rutting model	Deviance test (χ^2)	Number of estimated parameters	Model comparison test with null model		
			χ^2 statistic	Degrees of freedom	<i>p</i> -value (at 95% confidence interval)
Rutting null model	36123.49	5	-	-	-
Rutting growth model	17554.80	8	18568.69	3	<0.001
Rutting conditional model	16583.04	11	19540.45	6	<0.001

Validations:

To ensure that the developed models have the ability to predict future conditions precisely, both apparent and internal validation methods are used to test the developed models. These two methods were conducted as described below:

1) Internal validation

As mentioned before in this paper, approximately one-third of the data (30%) is set aside to use for model validation. This dataset is used to develop a validation model with the same variables that are used for the developed models. Multiple statistical testing using a Bonferroni correction is applied when checking whether the coefficients of the validation model fall within the 99% confidence intervals for the coefficients of the developed model, or not. The confidence interval estimate (CI) provides a range of likely values for each of the model coefficients. Based on the general form of a confidence interval, the lower and upper bounds of the 99% confidence intervals are calculated using the following formula:

$$99\% \text{ confidence interval} = \text{estimated coefficient} \pm 2.576 * \text{standard error} \quad (16)$$

The results of internal validation for the growth and conditional progression models are presented in Table 4. These results refer that all coefficients of the rutting models based on the validation datasets fall within the upper and lower bound intervals for the coefficients of the developed models. This means that all the models exhibit internal validity.

Table 4. Results of internal validation for growth and conditional rutting progression models

Model fit	Variables	CDM ¹	<i>p</i> -value (Developed model)	Standard Error	99% CI ³ LB ⁴	99% CI UB ⁵	CVM ²	<i>p</i> -value (Validated model)
Rutting growth model	Intercept	1.318	<0.001	0.021	1.263	1.372	1.326	<0.001
	Time	0.084	<0.001	0.001	0.083	0.086	0.083	<0.001
Rutting conditional model	Intercept	2.365	<0.001	0.070	2.185	2.546	2.287	<0.001
	Time	0.056	<0.001	0.001	0.053	0.059	0.058	<0.001
	LN(MESA)	0.050	<0.001	0.005	0.037	0.063	0.044	<0.001
	SNC _{<i>i</i>}	-0.379	<0.001	0.012	-0.411	-0.348	-0.350	<0.001
	TMI	0.001	0.004	0.0005	0.0002	0.003	0.001	0.029

1: Coefficient of developed model.

2: Coefficient of validated model.

3: Confidence interval

4: Lower bound

5: Upper bound

2) Apparent validation

In this validation method, the fundamental assumption of the initial developed regression model is tested by checking the assumptions on the random errors and using diagnostics plots to evaluate the model fit. The developed regression model assumes that the residuals (i.e. the differences between the predicted values and the actual values for the dependent variable) are independent and normally distributed and have a zero mean within each section (Raudenbush & Bryk 2002). The residual plots allow for the graphical

evaluation of the goodness of fit of the selected model. The random errors can be regarded as a random sample from a distribution by checking whether the residuals might have come from a normal distribution or not. Thus, two plots (Figure 1 and Figure 2) have been examined to evaluate rutting conditional model fit (equation 15). Figure 1 shows the frequency histogram of the residuals and indicates that the residuals are normally distributed with mean very close to zero. This suggests that the normality assumption is not violated. Figure 2 shows the actual DV versus predicted DV with the line of equality. From Figure 2, it is observed that the predicted rutting data against observed data are very close to the line of quality with $R^2 = 90\%$.

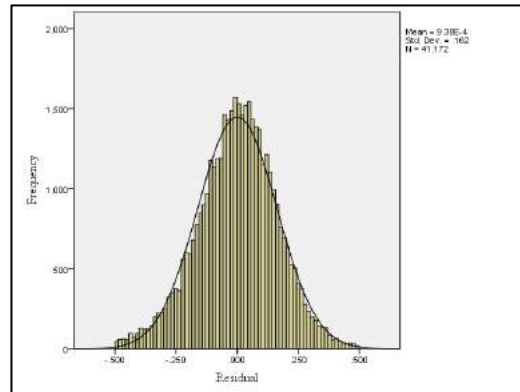


Figure 1. Residual histogram for the developed rutting conditional model

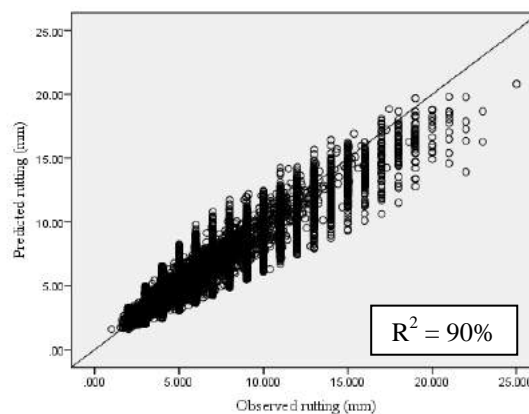


Figure 2. Predicted rutting values versus observed rutting data

10 MODEL SIMULATIONS

In order to understand the behaviour of the developed rutting models under different conditions, a deterministic simulation was used. This simulation analysis was done for the developed conditional rutting model over time. In simulating the conditional rutting model, mean, maximum and minimum values for traffic loading and pavement strength were used with wet and dry climate. These statistics are from the dataset used for developing the models, as presented in Table 1.

Figure 3 shows the conditional model under different conditions. When using this model, for example, for the pavement condition with average traffic loading ($MESA=2.99$) and average pavement strength ($SNCi= 2.543$), the maintenance activities for rutting value of 15 mm could be scheduled at 22 years and 23 years of the design life for pavements in wet climate (assumed $TMI=20$) and dry climate (assumed $TMI= -20$) (see simulations 1 and 2), respectively. However, these predictions could be more or less in different conditions when considering other values of $MESA$ and $SNCi$. These deterministic simulation plots for developed model indicate that the model predict the expected rutting progression over time, under different selected conditions.

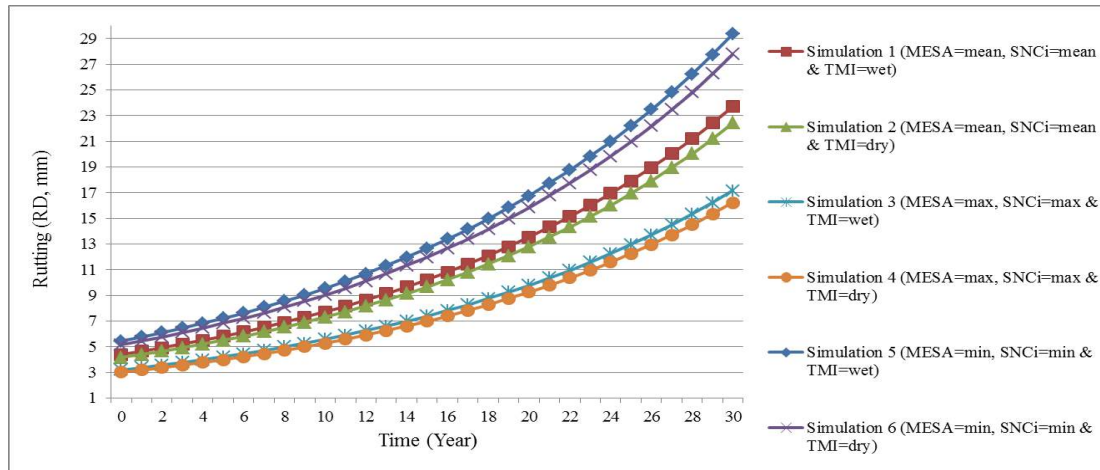


Figure 3. Simulation of the rutting conditional model under different conditions

11 CONCLUSIONS

The reported study was undertaken to apply a new approach to develop rutting prediction models for sealed granular pavements at network level. Particularly, the study presents hierarchical multilevel models that can account for the correlation among time series data of the same section and capture the effect of unobserved variables. The analysis results show that it is an effective approach to present rutting progression model over time.

The developed models are statistically significant and the parameter estimates are significant and have correct signs. More than half of the percentage of explainable variance in the log transformed rutting values was accounted for by the predictors in the conditional model with R^2 value more than 50%. The conditional model indicates that time, traffic loading and climate condition have positive contributions to rutting progression within the gradual phase. However, pavement strength has negative contribution to rutting progression. Drainage condition has no significant contribution to pavement rutting in the selected network. The analysis also shows that the effect of time is stronger than the other variables on rutting progression. Deterministic simulation plots for the developed growth and conditional models indicate that the models predict the expected rutting progression over time under different selected conditions. In addition, both validation methods supported all developed models. As the results show that there is a significant variance among the four road classes (M, A, B and C), it is recommended that a separate model for each road class to be developed.

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PAPER TITLE	Lessons from Liberia: Monitoring and Supervision of a 10-Year Performance Based Road Contract		
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KEYWORDS:

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ABSTRACT:

The first three years of this multiple international donor funded 10-year contract have seen the reconstruction of approximately 178km of new highway extending from the outskirts of the capital Monrovia to within approximately 68km of the border of Guinea.

Opus is the appointed Monitoring and Supervision Consultant for this project, supported by Arup Nigeria and Arup South Africa. Over this time we have encountered a wide variety of unique challenges including the coordination of team resources, management of construction delays, emergency events, force majeure due to the Ebola epidemic, maintenance of health and safety standards, construction quality, and environmental compliance.

This paper will outline how these challenges have been met and the lessons that have been learnt from supervising a performance based road construction and maintenance project of this scale, in a remote location, and where the availability of supporting services is very limited.

Lessons from Liberia: Monitoring and Supervision of a 10-Year Performance Based Road Contract

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1. INTRODUCTION

The Government of Liberia has received the assistance of the World Bank and other donors to implement an Output and Performance based Road Contract (OPRC) for the Road Asset Management of sections of the national road network. This form of contract was selected in order to increase the efficiency and effectiveness of road operations within the life span of the road.

A fundamental feature of the OPRC form of contract is that the contractor is responsible for the design, construction, and maintenance of the scheme in order to comply with the required service levels. These levels are defined from a road user's perspective, and include factors such as average travel speeds, riding comfort, safety features, etc.

A 10-year contract was signed on the 25th January 2012 by both the Liberian Ministry of Public Works through their Infrastructure Implementation Unit (IIU), and a large international Contractor for the Design, Rehabilitation and Maintenance of approximately 178km of the main road between the capital city Monrovia and to a point approximately 68km from the border with Guinea. A second similar contract was awarded to a separate contractor for the balance of the road to the border itself.

A separate 10-year contract was awarded to Opus International Consultants, supported by Arup Nigeria and Arup South Africa in April of 2013 for the monitoring and supervision of the construction and routine maintenance works. Under this contract the monitoring consultant is responsible for the day-to-day supervision of the construction works, health and safety compliance, environmental compliance, and general contract management. Once the routine maintenance phase commences in early 2017, the Consultant's role will change to auditing the service level compliance being achieved by the Contractor on a monthly basis. Where service levels fall below defined thresholds in the contract, then payment deductions may be applied to the monthly routine maintenance payment unless the recorded defects are rectified within the required response times.

2. PROJECT ROAD

The project road is located on rolling terrain and consists of an existing two lane single carriageway, with width's varying from 7.0 to 7.4 m with 1.0 to 1.5 m shoulders. The pre-construction pavement structure consists of 50 mm thick asphaltic concrete wearing course, sometimes covered by a thin slurry seal. This asphaltic layer overlies 100 mm of granular base and 150 mm thick laterite sub-base course. Prior to the commencement of the rehabilitation phase in 2013, the road pavement and surfacing was in very poor condition and several of the major bridge structures had been destroyed during the Liberia civil wars (1989 – 2003).

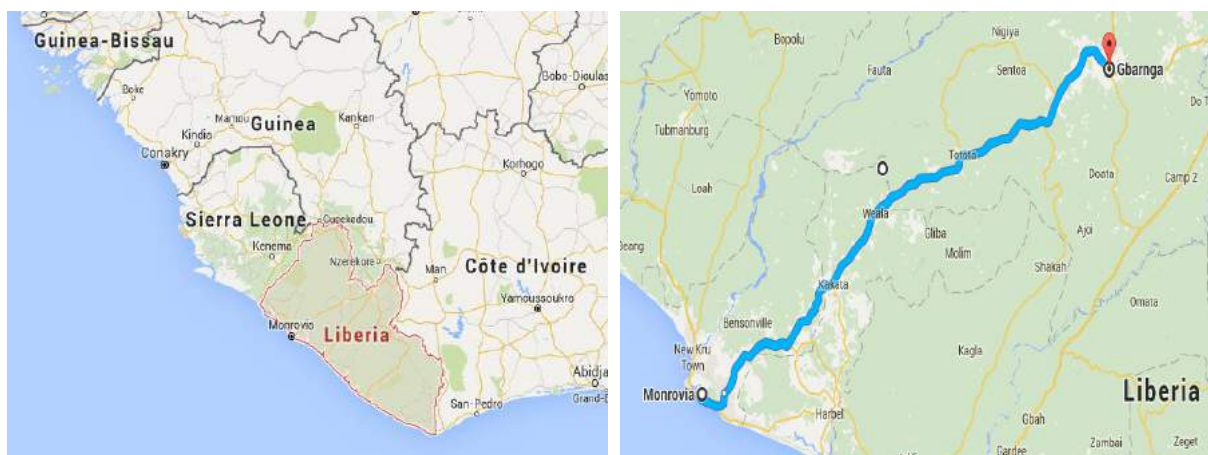


Figure 1: Location of the Project Road



Figure 2 Existing Pavement – Pre-rehabilitation.



Figure 3: Destroyed Bridge Structure

3. CONTRACT FORMAT

Under this form of performance based contract, the contractor is responsible for the detailed design of the construction works including all pavements, surfacings, bridges, culverts and other structures necessary to meet the requirements set-out in the specifications and conceptual designs provided by the Ministry of Public Works.

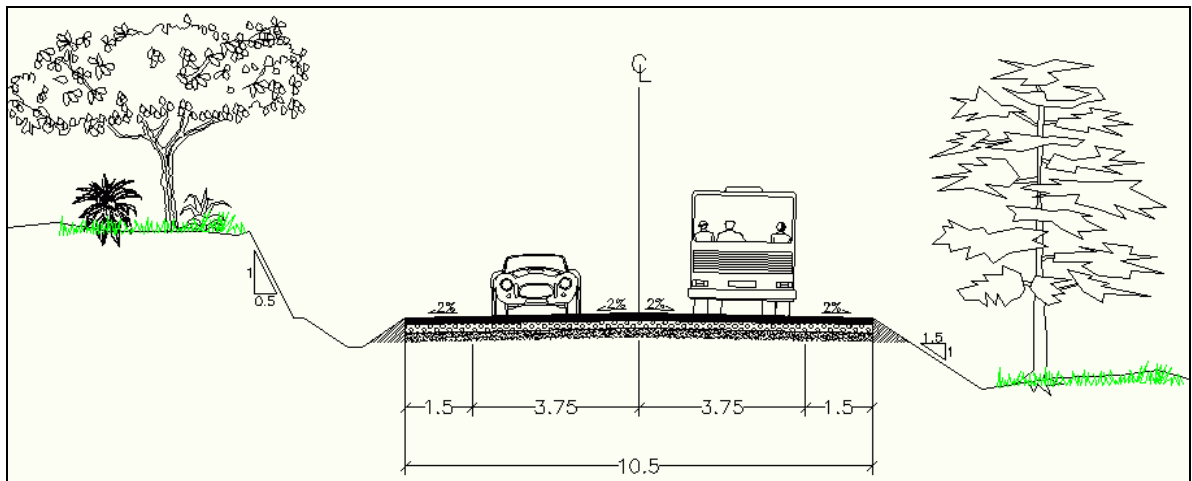


Figure 1: Conceptual Design for Rehabilitation Works

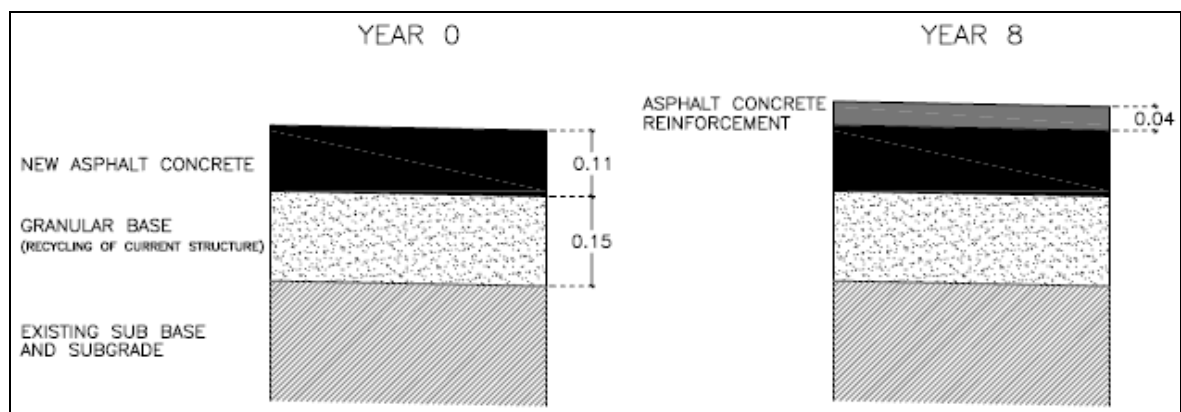


Figure 2: Conceptual Pavement Design

The contract works have been broken into four distinct phases:

- i) Six month mobilization and detailed design phase providing sufficient time for the Contractor to survey and design at least 15km of rehabilitation works for approval and enabling construction works to commence.

- ii) Three year rehabilitation phase for the complete reconstruction of all new pavements, surfacing, culverts and bridges
- iii) One year periodic maintenance phase (scheduled for the second to last year of the 10-year contract) requiring the Contractor to construct a full width 50mm Asphalt Cement (AC) overlay over the entire 178km of the project road
- iv) Seven year routine maintenance phase commencing from the end of the rehabilitation phase.

Bidders were required to provide a single lump-sum (LS) price covering all four phases of the contract. This lump-sum price was then artificially split as follows, and the apportioned percentages used to define the basis of payment to the Contractor:

Rehabilitation works:	55.4%
Periodic Maintenance works:	17.6%
Routine Maintenance works	27%

For the rehabilitation and periodic maintenance works, the Contractor is paid a nominal rate per km of completed works based upon the following formula:

$$\frac{\text{LS Price} \times \text{Apportioned \%}}{178\text{km}} \quad (1)$$

For the routine maintenance works, the Contractor is paid a monthly sum based upon the following formula:

$$\frac{\text{LS Price} \times \text{Apportioned \%}}{84 \text{ months}} \quad (2)$$

These apportioned percentages are determined through a financial model which is used to achieve the following outcomes:

- i) Spread some of the Contractor’s profit from the rehabilitation and periodic maintenance phases to the routine maintenance payments to incentivize the Contractor to remain focused on maintaining the specified service levels through to the end of the contract, and;
- ii) Determine to value of an advance payment which will minimize the level of borrowing and financing costs required by the Contractor and therefore included in their price.

Price adjustment for inflation is applied after the first 12 months of the contract based upon agreed and reported price indices for input items.

4. PROJECT CHALLENGES AND OUTCOMES TO DATE

This project has now entered its 4th year, and the Contractor is expected to complete the rehabilitation phase by the end of 2016. The improvement to the existing road in terms of ride and travel time have been significant with the resulting benefits to the road users and the general economy now being accrued.



Figure 6: Rehabilitation Works



Figure 7: Completed Construction

The specific challenges that have confronted the IIU, the Consultant and the Contractor during the rehabilitation phase of this project have been;

- i) The completion of the required Resettlement Action Plans that needed to be undertaken by the Government of Liberia, including the compensation to Project Affected People. The time required to identify, consult, agree compensation terms for resettlement, pay compensation, and carry out the necessary clearance works before any construction works could commence, has been much longer than anticipated due to the number of people involved and associated logistics. In an effort to reduce the potential for further delays, the Contractor has been willing to assist the IIU with the actual clearance works throughout the rehabilitation phase.
- ii) Ebola epidemic and Force Majeure suspension of the rehabilitation works. With the outbreak of the Ebola in mid-2014, the Contractor was forced to suspend the construction works for several months until the epidemic was brought under control. Lost time and additional costs were incurred in evacuating and then repatriating contract personnel as well as through the actual suspension of the rehabilitation works. The Contractor again provided assistance to the Government of Liberia during this crisis, and maintained a residual team in-country during this time to maintain the road, plant, and camp facilities. This saved time in remobilizing and re-starting the works once the spread of Ebola virus was contained and the crisis ended.

Due to the pressures of the contractual program, exacerbated by the annual variances in the duration of the rainy season (typically May through to October) construction quality has been constantly monitored to manage the risk around sub-optimal asset performance. The risk of reduced asset lifecycle occurring as a result of poor construction practices or the use of sub-standard materials has been reduced through the following:

- i) Payment for completed works subject to the provision of all specified construction test results by the Contractor
- ii) Oversight and inspection of the construction works by the Consultant, including independent sampling and testing to verify compliance with construction standards



Figure 8: Post Construction Testing



Figure 9: Verification of Minimum Layer Depth

- iii) The Contractor remaining responsible for the performance of all of the construction works and its maintenance for a further seven years beyond the end of the rehabilitation phase, which provides a further incentive to maintain construction quality standards and workmanship.

Work site health and safety compliance has been an important focus of both the IIU and the Consultant. In an effort to maintain acceptable standards the Contractor is required to nominate a safety supervisor at every work site who is responsible for ensuring the site is safe and all personnel are provided with the correct personnel protection equipment.

Where any site is found to be deficient in this regard, then the work site supervisor may be dismissed by the Contractor. Further repeated breaches of work site health and safety standards will then result in the Consultant shutting down of the work site completely until such time as the Contractor can demonstrate he has implemented acceptable safety standards and procedures to avoid further reoccurrences.



Figure 10: Acceptable Level of Worksite Personal Protection Equipment



Figure 11: Common Worksite Traffic Management.

The extent and impact of the construction works have presented a number of social and environmental challenges over the duration of the rehabilitation phase especially on the residents adjacent to the road construction corridor. While the Contractor, Consultant and the IIU have endeavoured to avoid or mitigate these impacts, these efforts have not always been successful in minimising the impact of the project. Where adverse environmental or social affects have occurred, then the Contractor has been instructed to carry-out acceptable remediation measures to address the harm that has occurred. The specific issues encountered to date have included:

- i) Storm water run-off and siltation into adjacent properties during construction works
- ii) Spillages of bitumen from storage drums on land and into water courses
- iii) Disposal of waste materials and spoil in unauthorised dump sites
- iv) Failure to provide bunds around chemical storage areas
- v) Inadequate maintenance of work camp septic tanks
- vi) Quarry blasting noise and vibrations causing damage to near-by buildings and structures.



Figure 12: Bitumen Drum Stockpile



Figure 13: Runoff and Sediment into Adjacent Land

While these issues have been isolated in extent, they have nonetheless had a significant impact upon the lives of the adjacent communities, and they have taken considerable time and effort to resolve to the satisfaction of the parties involved.

5. LESSONS LEARNT - SO FAR.

While not all of the problems could have been foreseen during the project development phase, a number have arisen as a result of a lack of adequate enforcement or compliance mechanisms within the contract. The development and inclusion of these mechanisms would have applied a greater level of financial incentive to the Contractor, and enabled pressure to be applied on the maintenance of minimum acceptable health, safety, social and environmental standards throughout this phase of the contract.

As a result of the issues that have been identified, the following aspects should be carefully considered in the development and implementation of similar projects in the future:

- i) The overall project program must clearly identify all required activities that need to be completed before the contract is awarded, thereby minimizing the risk of delays to the contractor's program. It is crucial that the commencement of the any Resettlement Action Plan, and compensation program for communities impacted by the project, occurs sufficiently in advance of the construction works to avoid this risk.
- ii) The engagement of a monitoring and supervision consultant (if required) should be aligned with the contractor's program to avoid overlaps or misalignment. Early engagement can assist with the identification of the risks that may otherwise adversely impact on the overall project progress along with the timely implementation of any appropriate mitigation measures.
- iii) The development of contract documentation and specifications must be sufficiently detailed and robust, especially around the sharing and management of identified contract risks between the various parties.
- iv) The definition of affordable service levels, both before and after any reconstruction phase, should be included to ensure the road is adequately maintained at all times, and with clearly linked payment and compliance mechanisms. Such an approach should include the an appropriate level of payment to the contractor to ensure he is able to carry-out necessary maintenance works at all phases of the contract.
- v) The development of clear protocols for the road controlling agency and the contractor to use to manage emergency and other unforeseen events should be completed. In particular these processes must ensure important contractual timeframes and related costs are recorded, and these are readily available in support of requested time extensions, claims for costs etc.



Figure 14: An Earth Batter Failure during Construction

- vi) The setting and agreement of appropriate metrics around health, safety, social, and environmental standards to enable clear contract performance measures to be reported over the duration of the contract should be undertaken. These metrics should also include adequate community consultation around activities that have the potential to upset, disrupt or cause damage to communities and their infrastructure, e.g. property access, storm water management, quarry blasting etc.
- vii) The development and implementation of a post-construction safety audit procedure to mitigate the risk of higher vehicle speeds and increased rates of road crashes is strongly recommended. The risk of increased vehicle crash rates is likely to arise where significant improvements to the road network have been carried out under the project. Ideally this strategy should be developed in conjunction with enhanced enforcement by the traffic police, and regularly reviewed until such time as the local road users become more familiar with the improved level of service being provided.
- viii) The development of a set of visual guidelines, with photographs that clearly illustrate what is acceptable in terms of asset condition, and the defects that would be recorded as a non-conformance is recommended. This will greatly assist with the understanding of the service level performance measures applied during the auditing process, along with agreement on any payment deductions that may be subsequently applied.



Figure 15: Example of a Shoulder Drop-off Defect and Measurement Method

- ix) Incentivize the contractor to achieve and maintain the above standards throughout the contract by incorporating robust but simple compliance mechanisms linked to his regular payments regime. The application of these mechanisms as part of the auditing process must be clearly understood by all parties, and then rigorously applied. Joint workshops or training exercises at the outset of the contract are recommended.
- x) Identify and implement appropriate training programmes and other incentives to build capacity within the road controlling agency and then to retain these skilled personnel over the duration of the project.
- xi) Align and implement supporting asset management systems (e.g. databases for the storage of inventory and condition information) to ensure those used by the road controlling agency and the contractor are compatible as soon as possible after the contract commencement. Adequate time should be made available for system implementation and training to be undertaken for both contractor, and contract management personnel.
- xii) Design new signage and road furniture assets to minimise the risk of theft or vandalism, thereby minimizing the risk of the contractor having to carry the cost of on-going replacement or repair.
- xiii) Avoid time-response criteria in the compliance management system where possible to minimise the amount of time required to check and verify that corrective actions have been undertaken by the contractor. It is recommended that an allowable defect per km approach is adopted which can then trigger payment deductions once a specified threshold has been exceeded. This allows for single visual audit to be undertaken which is simpler and requires less time to manage.

6. SUMMARY AND CONCLUSIONS

The concept behind these long term performance based road contracts, which can embody improvement, rehabilitation, periodic and routine maintenance works, is to drive improved asset management practices and value for money outcomes through:

- i) Transferring the responsibility for design, asset performance and long term maintenance to the contractor, which in turn drives an increased focus on construction quality and innovation;
- ii) Clearer definition of roles and functions within the road controlling agency organization, with a focus away from instructing the contractor on what to do to, to one of long-term asset management and governance;
- iii) An enhanced understanding of the road assets that are managed by the road controlling agency through the adoption of improved data collection, condition prediction and reporting systems;
- iv) Defined asset service levels leading to an improved understanding of asset deterioration, residual lives and lifecycles;
- v) Improved forecasting of future investment needs thereby increasing the probability that sufficient funding for both renewals and routine maintenance requirements will be available at the optimal time.

The issues and challenges identified in this paper are not out of context considering the scale and complexity of such a large road construction project, and they have been valuable in identifying opportunities for improvement in the design and implementation of these forms of performance based contract in the future. The

development of improved metrics for health and safety, social and environmental outcomes is one area in particular that advances need to be made to further mitigate the potential for adverse impacts inherent with projects of this scale.

Looking forward there are still a number of risks facing the Contractor, the Government of Liberia, and donor agencies with regard to the impacts of the delays and associated costs that have been incurred over the last three years. Nonetheless there is ample evidence that the improvements to this strategic section of the national highway network is already delivering significant benefits to the road users and the economy of Liberia.

The challenge over the next seven years is to now ensure the road network continues to meet the service level expectation of the road users in Liberia, with an optimal level of investment by the Contractor, and that the lessons learnt are applied to other similar projects in the future.

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TRACK	Intelligent Transport		
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ABSTRACT:

Planning road and railway projects is a complex task with numerous quantitative and qualitative criteria to be satisfied. It is acknowledged that the greatest opportunities to contain overall costs and minimise environmental and social impact are at the planning stage. However most planners/designers are still using the traditional manual alignment planning and selection method which is time consuming, inefficient and sometimes could miss a valuable corridor. The traditional method relies heavily on planners' professional experience and judgement. Increasing environmental concerns and growing pressure on project budgets have made the route selection process more complex than ever before. The lack of purpose-specific tools has greatly limited the planner's capacity to consider all data and constraints and to compare and analyse all viable alternatives within a limited time period.

An innovative intelligent transport planning tool has been specifically developed to address the complex planning problem. The planning tool begins with a digital terrain model of the study area and other relevant geographical information, and then by defining the input parameters such as design standards and cost parameters such as cut/fill rate and bridge/tunnel unit costs and other constraints such as avoid zones and crossing requirements, the planning tool will automatically search all possible alternative alignments within the study area. Unlike the traditional planning approach, it explores millions of alternatives and quickly returns a range of 3D alignment models with automated earthworks; structures such as bridges, tunnels attached, and associated volumes and costs calculated. The process can be operated iteratively with new constraints added during the planning process which can provide a swift response to public feedback and re-evaluate alternatives within hours instead of weeks and months. It also empowers planners and designers to minimise the negative environmental and social impacts of road projects, whilst reducing construction and life-cycle operation costs. When the project approaches the construction phase, it can also be used as a valuable engineering tool to refine the final alignment within the corridor, or perform a vertical optimization to reduce earthworks and other construction costs. The system also equipped with a 3D visualization engine that instantly generates a road or rail model with cut/fill and bridge and tunnel structures attached, so that the planner can perform a virtual drive through before finalizing the alignment.

This paper illustrates how the intelligent transport planning tool can shorten planning time, increase productivity and reduce project cost, whilst achieving results that have better environmental and social outcomes. A number of case studies are presented to demonstrate the innovative planning process and its adaptability to different project needs.

Intelligent Transport Planning System

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1. INTRODUCTION

Planning transport infrastructure projects is a complex task with numerous quantitative and qualitative criteria that must be taken into consideration. Aside from operational requirements of vertical gradients and horizontal and vertical curvatures, other factors such as legislative, environmental, and community constraints and project budgets must be considered during the planning process.

Conventional global practice for road and rail alignment planning is predominantly intuitive, relying in the first instance upon professional experience and judgement to identify the likely route. Increasingly, Geographical Information System (GIS) packages are being used to assist the initial corridor selection. Whilst most GIS packages enable constraints and anticipated high cost zones to be weighted, and therefore avoided on a priority basis, they cannot deliver 3D alignment solutions and therefore cannot accurately estimate the cost of the alignment, and thus only provide an indication of likely low cost routes.

On the other hand, Computer Aided Design and Drawing (CAD) Software are well advanced in a very competitive market. However, most CAD software developed serve for the purpose of detailed design after the initial routes or corridors had been selected, and normally have data limitation, and are not suitable for broader searches for good alignments.

The lack of purpose-specific tools during the planning process has frequently compromised the accuracy of route selection, for example, a lack of comprehensive study of all potential corridors, or excess curvature and gradients of the final alignment selected, all without much concern for the longer term consequences. The missing link is the planner's capacity to adequately consider all data and the numerous complex engineering, social, environmental and budget constraints.

The advances of geospatial imagery technology that provide quality and affordable data are running ahead of transport route planners' understanding and their ability to use it. It requires appropriate tools to take advantage of the availability of geospatial data and to effectively convert them into better engineering and environmental solutions.

An intelligent transport planning tool has been specifically developed to address the complex planning problem through geospatial technology and alignment optimization. This approach has changed the paradigm of alignment planning by taking tasks that were previously difficult, time consuming and therefore potentially very costly, and making them easy, fast and affordable. It uses state-of-the-art technologies to automatically generate low cost alignments that satisfy defined environmental, community and cultural heritage constraints. It can be used for corridor analysis and alignment selection; optimized alignments can be fed into a CAD system for detailed engineering design. The system is unique in its capacity to optimize 3-dimensional routes simultaneously in a larger scale whilst accommodating both broad-based and localised issues during the planning process.

The successful application of such a planning tool around the world has been independently confirmed by users, documenting better engineering and environmental outcomes and significant reductions in project planning time and construction cost.

2. GEOSPATIAL IMAGERY TECHNOLOGY

Modern alignment optimization processes require a study area with digital elevation models (DEM) or digital terrain models (DTM) as base data - now easily available from geospatial technologies sourced from

satellites and aerial surveys. The convergence of new technologies – geospatial and alignment optimization techniques – as a flow on from increased computing power have already made great impacts on the conventional transport planning process.

The resolution and quality of satellite images is offering increased opportunities for transport planners to take a strategic perspective of alignment locations. The new generation of imaging satellites provides spatial resolutions that have never before been available to the public and planners.

The advantages of satellite imaging are that it covers a large area compared with aerial photography so that it is possible to see the study area in context. Different resolutions are suitable for different stages in the planning process. The lower resolution can be used for initial route corridor selection, while the higher resolution can be used for final route alignment.

The 90-meter resolution SRTM global digital elevation data (Web-1), originally produced by NASA and first released in 2003, is a major breakthrough in the digital mapping of the world, and provides a major advancement in the accessibility of high quality elevation data for large portions of the tropics and other areas of the developing world. SRTM elevation data has now been released for the entire earth surface (Web-2).

In October 2011, the release of the Advanced Spaceborne Thermal Emission and Reflection Radiometer (ASTER) Global Digital Elevation Model Version 2 (GDEM V2) was announced following the first version of the GDEM released in June 2009. The improved GDEM V2 adds much more stereo-pairs, improving coverage and reducing the occurrence of artefacts. The refined production algorithm provides improved spatial resolution, increased horizontal and vertical accuracy, and superior water body coverage and detection (Web-3). The GDEM data provided an unprecedented level of access to the public and also provided an opportunity for transport planners to conduct scoping and pre-feasibility studies using GDEM as base data or platform.

In April 2015, Airbus Defence and Space has officially launched its WorldDEM Digital Terrain Model (DTM), a highly accurate standardized representation of bare Earth elevation that can be made available for any point on the globe (Web-4). A key feature of this DEM is its unrivalled vertical accuracy at 2m (relative) / 4m (absolute) in a 12m x 12m raster resolution covering the entire land surface of the Earth.

Other satellite sources such as Quickbird and IKONOS provide more detailed data at sub-meter resolutions which can be used for feasibility studies. The latest WorldView-3 satellite was successfully launched on August 13, 2014 and is the first multi-payload, super-spectral, high-resolution commercial satellite. It is capable of discerning objects on the Earth's surface as small as 31cm in the panchromatic mode. This will have the highest resolution of any commercial imaging system (Web-5). Table 1 shows a list of satellite sourced DEMs available for most scales from early stage project scoping study to the detailed project scale (Web-4, 5, 6).

At the very detailed level, the LIDAR (Light Detection and Ranging) technology which is capable of capturing sub-meter resolution DEMs, can be used for detailed design and construction. LIDAR technology has become a valuable tool for measuring and recording elevation data for use in topographic mapping and 3D terrain/surface modelling with much improved accuracy and bare earth surface. Its accuracy is further improved with enhancement in GPS technology (Web-7).

By utilizing the latest geospatial data, planners and environmental groups can perform more advanced analysis in the early stage of a project, which provides the greatest opportunity to reduce the project cost and avoid potential environmental problems.

Table 1. Summary of Satellite Sourced DEMS and Accuracy (Source: Web-4, 5, 6)

Source	Spatial resolution (m)	Accuracy (m)	Availability	Planning Stage
SRTM	30-90	6-8	Entire earth surface	Scoping studies
ASTER(GDEM)	30	variable	Most land area	Scoping studies, pre-feasibility studies
SPOT HR	20-30	8	From archive	Scoping studies, pre-feasibility studies
Airbus Defence and Space	12	2	New capture Entire earth surface	Pre-feasibility studies, feasibility studies
ALOS PRISM	5	3-5	From archive	Pre-feasibility studies, feasibility studies
WorldView-1,2,3,4 GeoEye-1,2	<1	0.3-1.0	New capture	Feasibility studies

3. TRADITIONAL ALIGNMENT PLANNING PROCESS

The alignment planning process is a complex balancing task. It has normally relied heavily on planners' professional experience and judgement; traditionally it is a manual process. Every project is unique; designers are challenged by different engineering requirements, complex topography, geological conditions, community values, safety and environmental concerns. Additionally, road alignments are 3-dimensional spline structures; a road project may have infinite alternative alignments between the start and end points, and thus there is no simple solution to the complex problem. In the past, this issue has been simplified by converting 3-dimensional problems to 2-dimensional problems.

When using a manual method, designers must generate a few horizontal alignments based on topographic maps or digital terrain models and other knowing factors, and then attempt to fit the vertical alignment along the profile, before checking the cross sections along the route to ensure horizontal and vertical alignments are properly coordinated, all whilst considering all the factors relating to the project including earthworks and cost. The process is tedious and time consuming and has a vast amount of constraints to take into consideration, especially for complex terrain. If problems are encountered in horizontal or vertical alignments such as violations of radius of curvature or design grade or inadequate cross sections, this process may require to be repeated all over again which may result in a prolonged process or compromise the final solution.

Even with the extensive use of GIS and CAD packages and some kind of digital terrain model, the initial corridors and alignments are predominantly selected manually by experienced experts. Furthermore, increasing environmental and construction cost concerns have made the alignment selection process more complex than ever before. The lack of purpose-specific tools has greatly limited the planner's capacity to consider all the data and constraints, and compromises the ease of the comparison and analysis of all viable alternatives within a limited time period. An intelligent automated alignment optimization model is much needed and it could greatly improve the planning process in terms of productivity, cost savings and better environmental outcomes.

4. INTELLIGENT TRANSPORT ALIGNMENT OPTIMIZATION TECHNOLOGY

Using DEM as base data, together with geometric standards and other constraints such as crossing requirements with existing linear features or no-go zones, the generation of sets of low cost alternatives can be readily achieved by employing one of a number of stochastic optimization techniques. Originally developed to optimize discrete systems by swapping the contents of randomly chosen pairs of locations, the methods can be adapted to working in a continuous space. The technique is similar to the methods employed

by designers working interactively with standard packages, who move an intersection point defining the alignment subject to defined constraints, however the automated technique has an advantage because it is considering the optimal horizontal and vertical alignment simultaneously rather than the traditional two stage process whereby defining horizontal alignment is followed by vertical alignment design.

The cost of the modified alignment is calculated and compared with the cost of the old alignment, and a decision is made to accept or reject the change. Once a decision has been made, another intersection point is selected and the process repeated. The rules governing the acceptance or rejection of a point provide the mechanism by which the alignment can escape from local sub-optima. The algorithm does not guarantee that the alignment will escape; it merely makes it possible. However, careful selection of the parameters controlling the process can greatly increase the probability of reaching the global optimum. By locating intersection points in a continuous three dimensional space, the final alignment can be specified to arbitrary accuracy. However, in practical situations, continuing the optimization beyond a given accuracy does not contribute significantly to the value of the solution. Accordingly, optimization can stop when the spatial range of the acceptable moves made by the intersection points declines to a value from which planners and designers can work without having to worry about the macro location of the route.

The Quantm system (Gipps et al. 2001) is specifically designed as a planning tool to address the complex planning problem. It empowers road planners who need to be able to determine routes that consider numerous relevant variables including terrain, design standards, existing infrastructure, geological cost variances, and community and environmental constraints. The system considers literally millions of alternatives to enable planners to consistently identify significantly improved alignments that deliver considerable cost savings while protecting environmental and community values.

The system owns a patented optimization engine which is cost driven, but the 'cost' of an alignment is multi-dimensional with many costs being unquantifiable social and environmental impacts. Consequently, the system focuses on objective costs and generates sets of low-cost alternatives that meet the planner defined constraints, rather than a single least cost solution. This enables planners to balance environmental and social impacts against monetary costs for routes using different parts of the study area (Gu 2010).

Objective costs can arise during the construction from the volumes of earthworks, the length and height of structures, and from life-of-project factors such as fuel consumption and maintenance. Planners currently use CAD packages to attempt to balance earthworks, minimize structures and haulage and generally produce an alignment that should be inexpensive to construct, while meeting the specified geometric standards. However, the results produced by the CAD operators are based on volumes and lengths, not dollars, and are often far from the best that can be achieved for construction. The impact of particular alignment decisions on the life cycle cost is often not part of the brief held by planners despite the fact that these may be many times larger than the capital cost of construction. On the construction side, unit rates are used to convert the quantities and lengths involved in construction into dollars. Similarly, forecast traffic and alignment geometry can be used to evaluate impact on operating costs.

Road and rail projects face numerous constraints on their alignment that must be satisfied before they are allowed to proceed. There are many factors which can have a major impact on the optimum alignment of a route that need to be considered during planning. Unless a model can cope with a minimum range of factors, it cannot provide the tools necessary to produce realistic solutions. The Quantm system incorporates all information required to produce 3D alignments that meet all design requirements and other constraints. Figure 1 shows the application of the Quantm system in the 250km Kyrgyzstan railway Connect Uzbekistan - Kyrgyzstan to Kashgar, Xinjiang, China. It shows automatically generated multiple alignments, including the plan, profile and cross section as well as earthwork volumes and associated costs of the alignment identified. Alignments generated in the system can also be visualised in real-time by a 3D visualization engine (Figure 2). This can be used for technical analysis of the alignment quality and presentation to stakeholders and the public consultation.

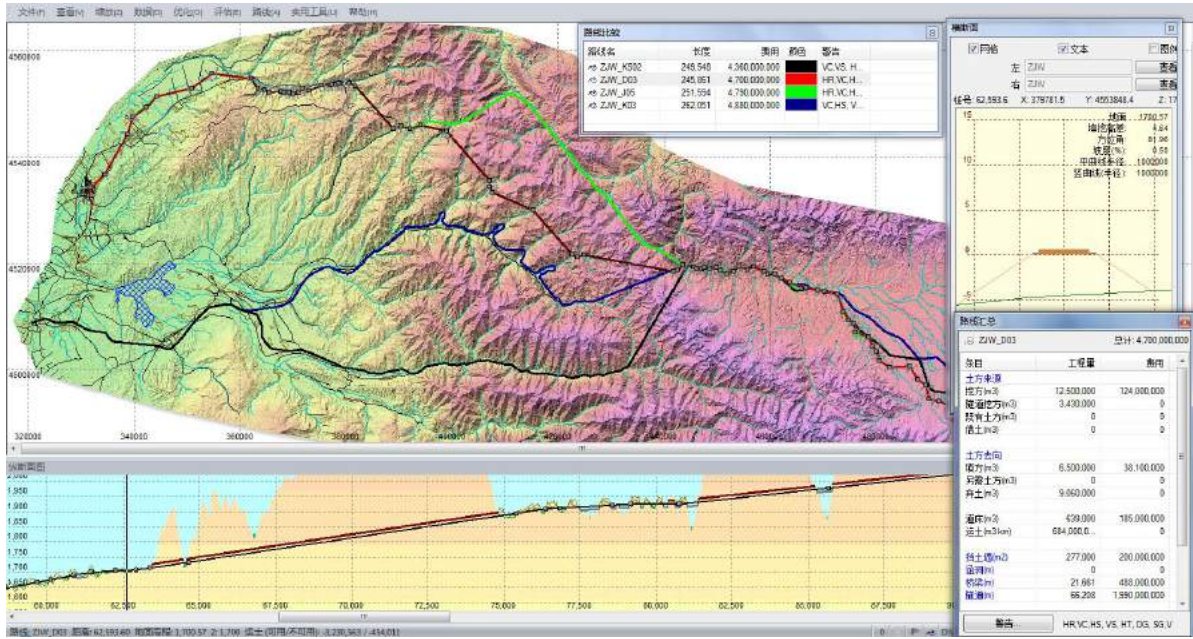


Figure 1. Quantm 3D Alignment Optimization for a 250km Kyrgyzstan Railway Project

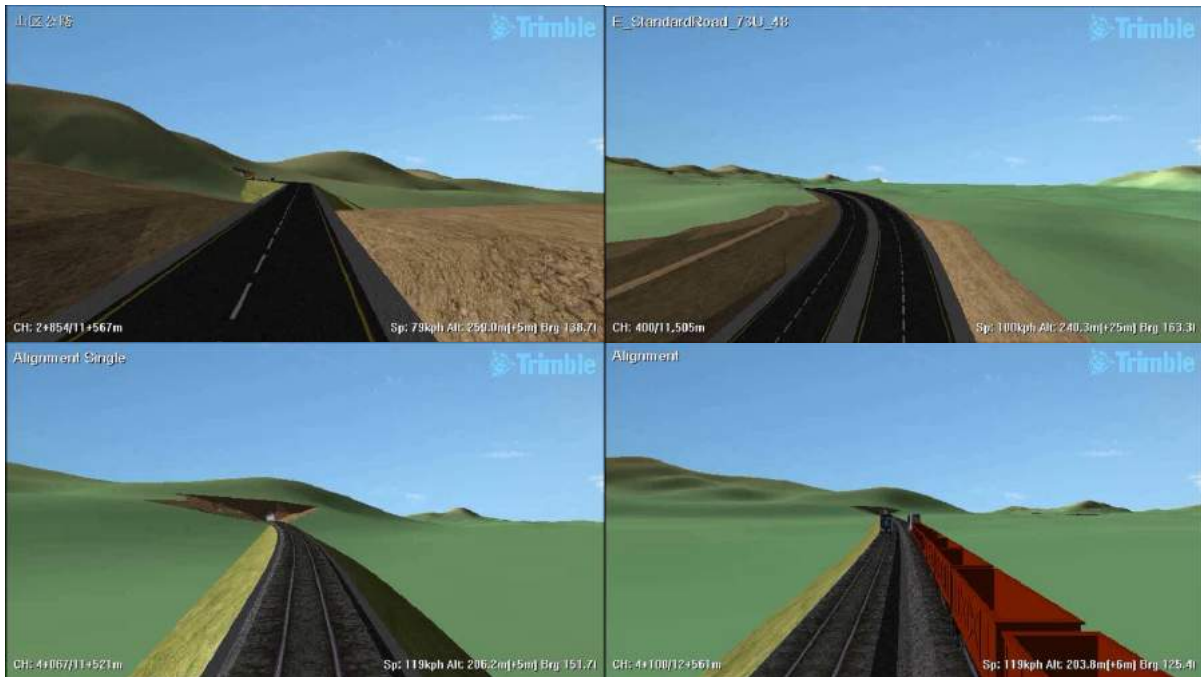


Figure 2. Quantm 3D Alignment Visualization

5. BENEFITS OF THE NEW TECHNOLOGY

When all of the constraints feature and cost data exists, the system is fast, effective and consistent. The speed depends on the number of constraints and the level of detail in the model, but comprehensive analysis and identification of preferred alignments have been completed within weeks rather than months compared to traditional alignment selection processes. Planning time and community and environmental consultation benefits are relevant regardless of terrain type. Users of such a system have also frequently stated that the system has enabled them to deliver unexpected solutions that had not been identified using the conventional planning approach, where investigation is severely limited and guided by the intuitive skill of the planner.

The new alignment optimization technology such as the Quantm System, allows planners and engineers to collaborate with various road project stakeholders and deliver improved outcomes. The process maintains positive community relations, considers environmental impact, and reduces project construction costs - all while maintaining the momentum of the road project.

Improved productivity - Once a project is initialised the system can generate better, less expensive alignments in minutes, whereas using conventional means could take weeks to produce the same result. Amending constraints and parameters is fast and reruns are completed immediately. Consequently, it is very easy to consider the impact of variations suggested during consultations on community and environmental issues. In many large projects, the time and cost involved produces natural resistance to considering variations when they are not considered to be viable. The impact of considering these variations and getting back rapidly with cost effective alternatives will significantly reduce community stress and planning blight that often hampers adoption of proposed alignments.

Improved community relations - Communities globally are increasing the influence they have over where new roads are created. The combination of community input and the need to avoid certain urban areas, cultural heritage zones and areas of environmental sensitivity have made the planning process extremely complex. The new technology allows community input to be integrated into the planning study. Community "no-go" constraints can be added in just minutes. Cost implications and alignment alternatives can be reviewed within hours depending on project scale.

Better environmental outcomes - The Quantm System was identified as one of eight emerging technologies by the Transportation Research Board (TRB) (Web-9) that support the integration of environmental considerations into transportation planning. The Quantm system can rapidly consider new environmental or social constraints as the project progresses and demonstrate to stakeholders that their input has been integrated into the study. Planners can be more responsive to public feedback and re-evaluate alternatives within days instead of weeks or months.

Reduced project planning time and delays - The current industry approach to road and rail planning addresses each constraint issue in series, which often leads to conflict between the stakeholders and requires numerous subsequent impact studies. This can lead to a planning spiral that delays, or even terminates, a project. The new technology encourages cooperation between the road project planners and stakeholders from the onset. Planners can quickly investigate alignment and cost implications of numerous "what if" scenarios simultaneously. As a result, planners can demonstrate that they have considered all feasible alignments and substantially improve the quality of the alignment without project delay or cost increase.

Construction cost savings - The new technology delivers substantial alignment construction cost savings while meeting the defined environmental, community, heritage and design constraints. Consultants that have adopted the technology have found that the more thorough alignment searches available through the computerized technique have allowed them to reduce the project costs, offering a better solution to their clients.

6. APPLICATIONS AND CASE STUDIES

In infrastructure planning projects in China, Australia, New Zealand, the United States and Europe, the Quantm system has demonstrated that it provides an unprecedented capacity for planners to dramatically improve financial outcomes whilst aligning project outcomes with formal Government policies in Ecological Sustainable Development. It also enabled the agency to put a dollar value on protecting local environmental, community and heritage values.

Projects of all types and sizes can take advantage of the benefits of the new technology. Examples range from regional, state and national transportation infrastructure planning to small bypasses and road realignments. In addition, the system can be used for mining, forestry and utility industry infrastructure.

6.1 Scoping Study: Beijing-Shenyang High Speed Rail Project

The 660km Beijing-Shenyang High Speed Rail Project (Figure 3) is part of the China national high speed rail network with a design speed of 350km/h. The scoping study was conducted in 2008 with the Third Railway Survey and Design Institute Group Corporation (TSDI). The objectives of the study were to identify

viable corridors and alignments that meet the requirement of economic development and environment constraints en route and to control the project cost.

Based on existing DTM and GIS data, design standards and user defined constraints, the Quantm system was effectively applied to conduct a larger scale area search for all viable corridors and to evaluate different alignment options. The project team was able to consider all the constraints, evaluated ten different sectional options comprehensively and identify the best corridor and preferred alignments in just three weeks. The application of the Quantm system enables the project team to gain greater depth of information on which to make their recommendations and to be able to present alternatives and the rationale for decisions to stakeholders and the community.

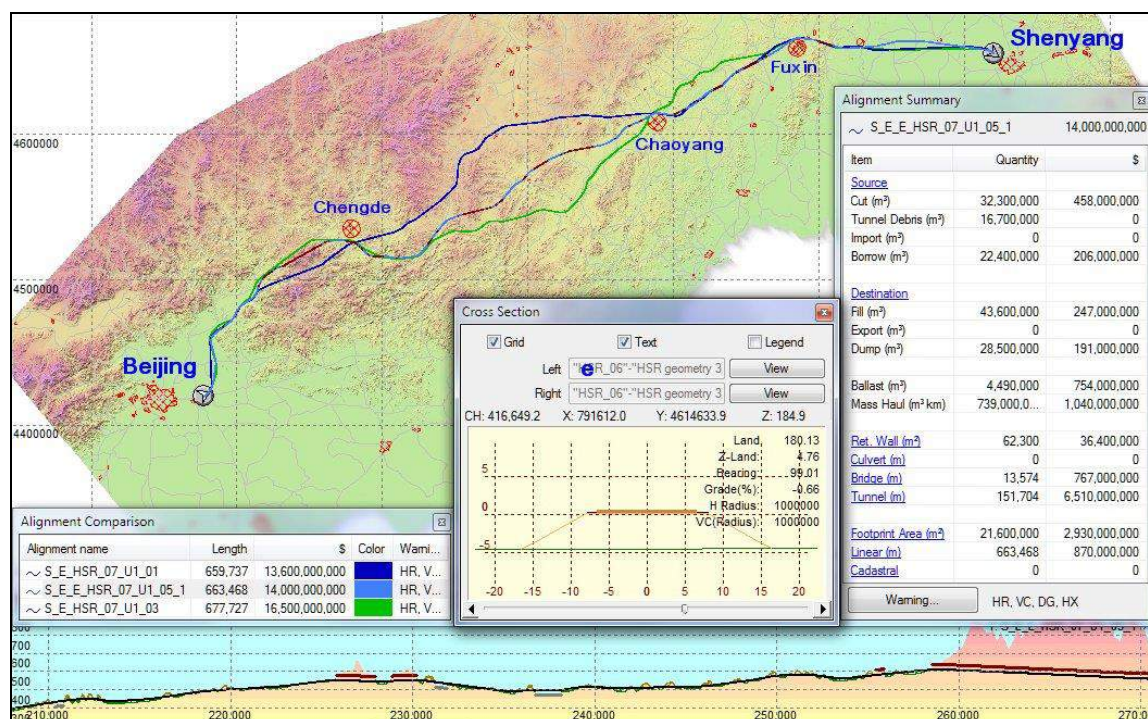


Figure 3. Quantm application on the Beijing-Shenyang high speed rail project

6.2 Feasibility Study: Xichang-Shangrila Mountainous Highway Project

The Xichang-Shangrila mountainous freeway project (Figure 4) was conducted by the Sichuan Transport Design Institute (STDI) in China. The project was located in Sichuan and Yunnan provinces with mountainous terrain. Due to complexity and construction constraints, the design team spent three months with several site visits and developed a recommended alignment option based on manual selection process, but the design team was not fully satisfied with the alignment option they had identified. The challenges of the project are to reduce bridge and tunnel ratio in order to control the project cost to stakeholders, and in the meantime, to maintain the design standard and meet the requirement of construction constraints, i.e., maximum bridge height is no more than 350m and maximum tunnel length no more than 16km. The original design alignment was developed based on traditional alignment selection process with a total length of 363km, bridge/tunnel ratio is 68%, maximum tunnel length was 14km and maximum bridge height is 350m.

Within three days of data preparation and optimization using Quantm technology based on existing DTM and GIS data with required constraints, a better corridor was identified and a new alignment finalised which satisfied all constraints including design standard and construction requirements. Compared to the original design alignment derived from a manual selection process, the optimized alignment was significantly improved in terms of length and alignment quality. The total length of the alignment reduced to 320km, bridge/tunnel ratio reduced to 66%. While the maximum tunnel length is increased from 14km to 15.5km, it was worthwhile to reduce the total length of the alignment by 43km. The shorter alignment significantly reduces both the construction and life-cycle operation costs.

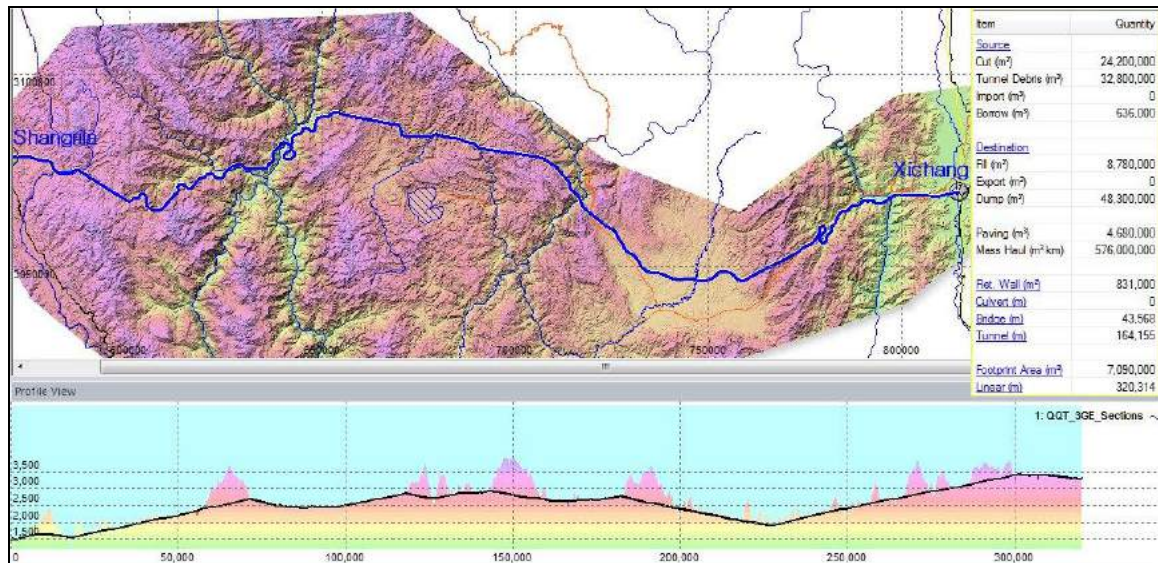


Figure 4. Quantm application on the Xichang-Shangrila Highway Project

6.3 Value Engineering Study: Boulder City Bypass in Nevada

Carter & Burgess and the Nevada Department of Transportation (NDOT) commissioned the Quantm System as a value engineering tool on Phase 2 of the Boulder City Bypass, Nevada USA. The Boulder City Bypass, Nevada project (Figure 5) will eventually result in a four-lane divided highway route for U.S. 93 traffic bound for Hoover Dam and points beyond. The new route will eliminate the need for through traffic to pass through densely developed areas of Boulder City (Phase 1). It will also connect the end of I-515 in Henderson to the start point of the Hoover Dam Bypass between Boulder City & Lake Mead (Phase 2) (Web-11).

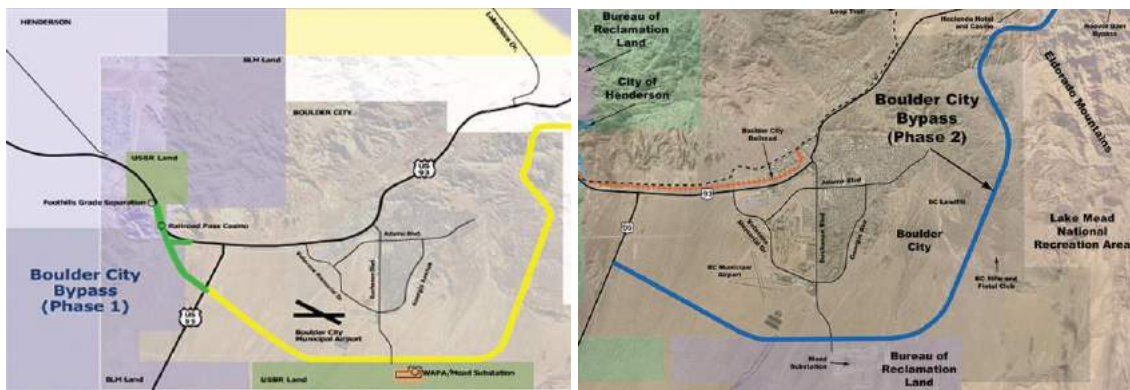


Figure 5. Phase 1 and Phase 2 of the Boulder City Bypass

Phase 1 of the 2.5 mile bypass was completed by NDOT – Carson City. The project's purpose was to improve operations and safety near Railroad Pass. A corridor was chosen and the project was estimated to cost US\$170 million. The project imposed several challenges, including difficult, hilly and undulating terrain, and numerous stakeholders (including NDOT, USA Department of Transportation, Regional Transportation Commission and many environmental bodies). With a corridor already chosen in Phase 1, the primary purpose of utilizing the Quantm system at this stage (Phase 2) in the project was to reduce construction cost. With the Quantm system, the project team was successfully able to achieve cost savings of 10-15% within a short period of time.

6.4 Design Study: G207 Highway Jingmen Section Project

The G207 Highway Jingmen Project is a 77km section in Hubei Province China. It is part of the Hubei highway network with 4-lanes in each direction and a design speed of 80km/h, maximum design grade of 4%. Jiangsu Transportation Research Institute (JSTI) was commissioned for the design work of the project.

Using Quantm technology, JSTI conducted full analyses on a 14km section of the project to compare the results between Quantm and traditional methods. The purpose of the study was to verify the existing corridor and the alignment from the feasibility study which derived manually using traditional method; and identifies potential better alignments within the corridor on 1:20K data; and finally, with the site visits and survey data, to determine the recommended alignment for presentation to the project owner.

The input parameters and constraints for the projects include geology conditions; crossing requirements with existing railways, highways and waterways; cost data (unit cost for earthwork and structures); environmental (built-up areas, avoid zones) and social requirements (local urban planning, land/property acquisition, mining areas, built-up factories and reservoirs).

The speed of the Quantm system allows the JSTI project team to conduct a sensitivity test to analyse the best design standards for the project (best combination of different minimum horizontal radii and maximum sustain grades): 4 different sustained grades (1.5%, 2.0%, 2.5% and 3.0%) and 6 different horizontal radii (250m, 400m, 550m, 650m, 750m and 1000m) were tested. The results were compared for the 24 combinations. By taking consideration of these sensitivity results together with safety and tunnel requirements, within just one day the project team was able to quickly determine the 2% as a sustained grade (maximum grade at 4%) and 650m as the minimum horizontal radius as the design parameter for the section. This would be a very time consuming task using traditional methods.

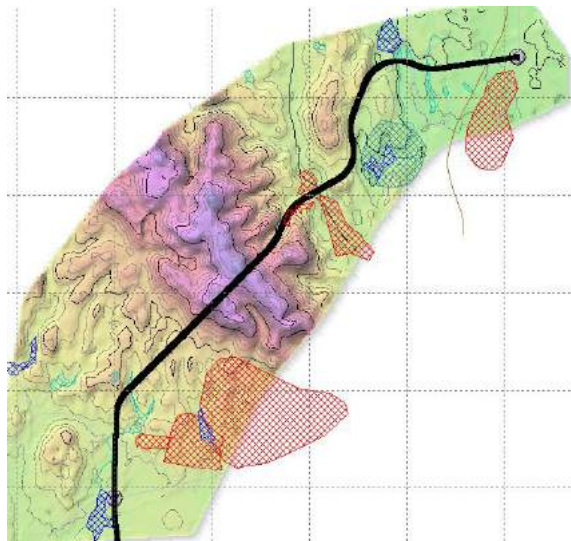


Figure 6. Manual Alignment from Feasibility

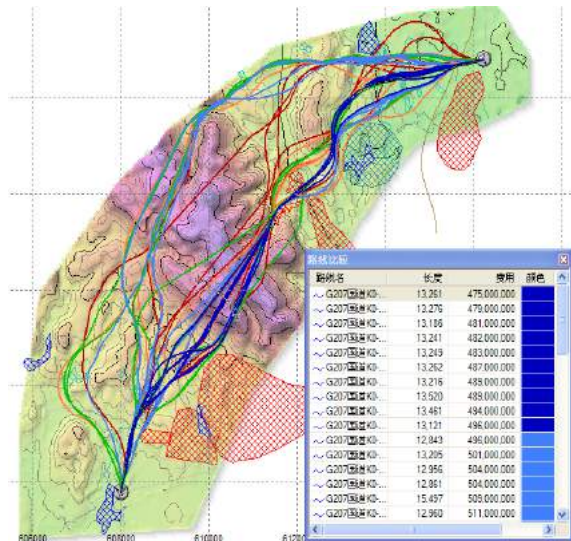


Figure 7. Quantm Generated Alignments

The optimization results based on 1:50K ASTER data and 1:10K local DEM data show that the corridor identified was the same as the one from the feasibility study; which provided the supporting evidence of the feasibility study to the project owner (Figure 6 and Figure 7).

By implementing further optimization based on 1:2K local DEM data, the system is able to deliver better alignments (Quantm 1, Quantm 2 and Quantm Final) compared to the manual alignment as shown in Table 2. The optimised alignments not only reduced earthworks, but also reduced the number of bridges and divided long tunnels into smaller tunnels which resulted in an overall reduction in structure length by over 1km and saved a total alignment cost of 20%.

In the JSTI project report on Quantm application, the benefits of the new technology are summarized as follows:

- Proving sensitivity tests to determine the best geometric fit to the terrain
- Identifying valuable corridors easily
- Estimating earthworks and structure relatively accurately to reduce project risk at early stage
- Quick response to new constraints, improving communications between stake holders, therefore reducing overall planning time
- Reducing construction costs

Table 2. Alignment Comparison: Quantm vs Manual

Item	Unit	Quantm 1	Quantm 2	Quantm Final	Manual alignment
Cut	10 ³ m ³	733	714	780	1,490
Fill	10 ³ m ³	667	858	792	565
Dump	10 ³ m ³	725	560	687	1,740
Bridge	m/number	1,798/7	1,636/5	1,934/5	2,779/7
Tunnel	m/number	2,897/2	3,005/2	2,899/5	3,232/2
Length	m	13,536	13,608	13,484	13,803
Cost	¥ 10 ⁶	588	593	597	745

7. CONCLUSIONS

Technological advancements have now made it possible for road planners to quickly identify better alignments that minimize the amount of cut, fill, tunnels and bridges while simultaneously meeting all of the planner-defined constraints. Successful applications of intelligent transport planning technologies have demonstrated that planners can now explore the whole study area thoroughly, investigate all feasible routes without missing any valuable corridors and without influence of preconceptions of where the route should run and quickly review all alternative strategies. This can be achieved within considerably faster times and enable planners to deliver more cost-effective viable solutions to their clients in every stage of a project, starting from early scoping studies, feasibility studies to value engineering studies. The speed of optimization enabled changes to be made to the underlying constraints and evaluated rapidly. The system also enables users to quickly conduct comprehensive sensitivity analyses with a combination of design variables such as limiting grades and horizontal curvatures to analyse the impact on alignment construction costs and life-cycle operation costs.

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PAPER TITLE	Road Surface Condition Surveys at Traffic-Speed		
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LCMS, MTD, MPD, SPTM, ELAtextur

ABSTRACT:

Road networks may not only be large in nature, but there is also a large investment in the existing and expanding pavement infrastructure, and this large investment has to be managed as one would do with any other investment. Road Authorities must decide on how to spend their available budgets effectively in combination with safety standards and allowable noise levels.

The assessment of the structural and functional pavement condition is nowadays fully machine based and non-destructive equipment does cover all the data input required for a modern PMS approach. A Multi-Laser Vehicle is capable of real time continuous traffic-speed measurements of condition data regarding cracking, raveling, rutting, roughness, macro texture and road geometrics in a single run using line-scan camera & ROW based images for an automated analysis. The system of the Multi-Laser Vehicle equipped with a Laser Crack Measurement System (LCMS) is the latest development in 3D camera technology. The rating of the severity and area of the distresses can be adjusted according to any existing visual distress procedure.

This paper discusses the results of the equipment used and does show the continuous measured macro texture based on the MTD as index using the LCMS device in addition to the automated crack detection. A major advantage is that no conversion is required as the volumetric MTD is a direct output. The LCMS based automated MTD analysis is a machine based result not influenced by a human factor such as the Sand Patch. The results allow as well the automated analysis of loss of aggregate which is a major advantage in rating the severity and extent of raveling in comparison to the non-consistent wind-screen surveys and manual rating of collected images. The MTD does relate as well to the frictional characteristics of a road surface and the noise level caused by the tires of a vehicle, which allows to distinguish between the noise reduction achieved using different types of open graded surface courses.

Road Surface Condition Surveys at Traffic-Speed

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1 INTRODUCTION

Road networks may not only be large in nature, but there is also a large investment in the existing and expanding pavement infrastructure, and this large investment has to be managed as one would do with any other investment. To be able to manage these investments it is essential to have information about the structural and functional condition.

The assessment of the structural and functional pavement condition is nowadays fully machine based and non-destructive equipment does cover all the data input required for a modern PMS. The falling weight deflectometer (FWD) defines the structural capacity of a pavement structure, friction testing equipment provides continuous self-wetted coefficients of slip friction and a Multi-Laser Vehicle is capable of real time continuous traffic-speed measurements of surface distresses like cracking, raveling, longitudinal roughness (IRI), rutting, macro texture, road geometrics using high-speed line-scan camera & ROW based images for an automated visual assessments.

This Multi-Laser Vehicle is equipped with a Laser Crack Measurement System (LCMS) the latest and most enhanced technology for collecting pavement surface condition information. As the name does say it the first application was focussed on all types of cracks visible at the pavement surface based on sampling transverse texture profiles at a 2.5-5mm interval. The LCMS does measure extremely accurate rutting with a transverse profile sampling interval of 1mm and equipped with inertial motion units (IMUs) the LCMS does not require an additional multi laser profiler anymore for the analysis of IRI as well as other indices. Figure 1 does show the vehicle with LCMS sensors.



Figure 1. Vehicle with LCMS Sensors

The surface images produced by the LCMS analysis software are (see Figure 2):

1. Intensity (2D): the reflective properties of the pavement surface.
2. Range: the distance from the sensors to the road surface corrected to the same reference level.
3. 3D: merging the Intensity and Range images.

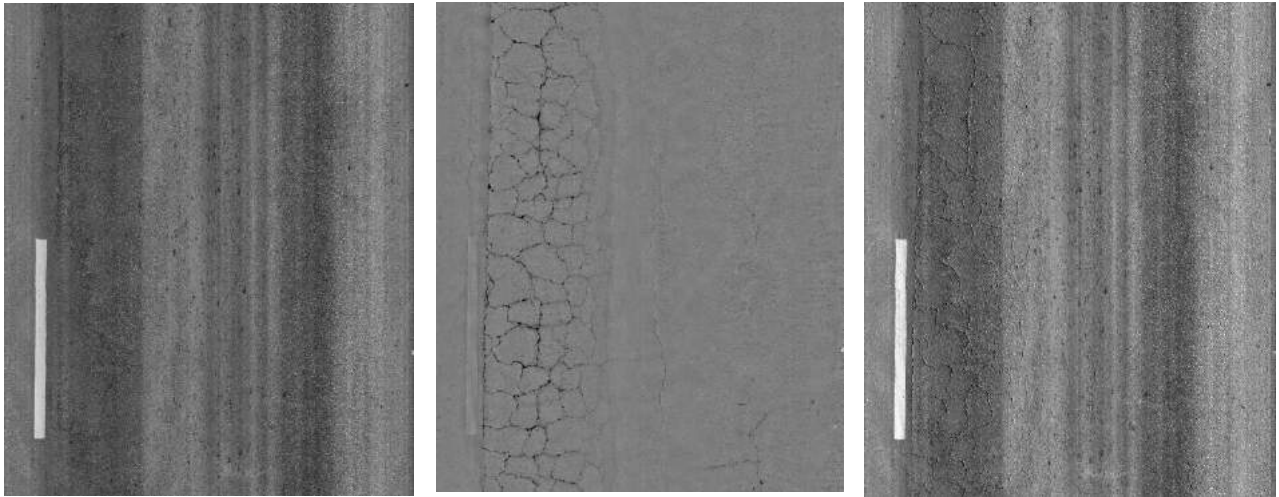


Figure 2: From left to right – Intensity/Range/3D pictures

The greater the distance from the sensors to parts of the pavement surface the more black the grey-scale range picture will be. The range image is the perfect tool to observe cracks as shown in Figure 2 in the middle. Figure 3 does show an Expressway location with cracks and the resulting analysis.

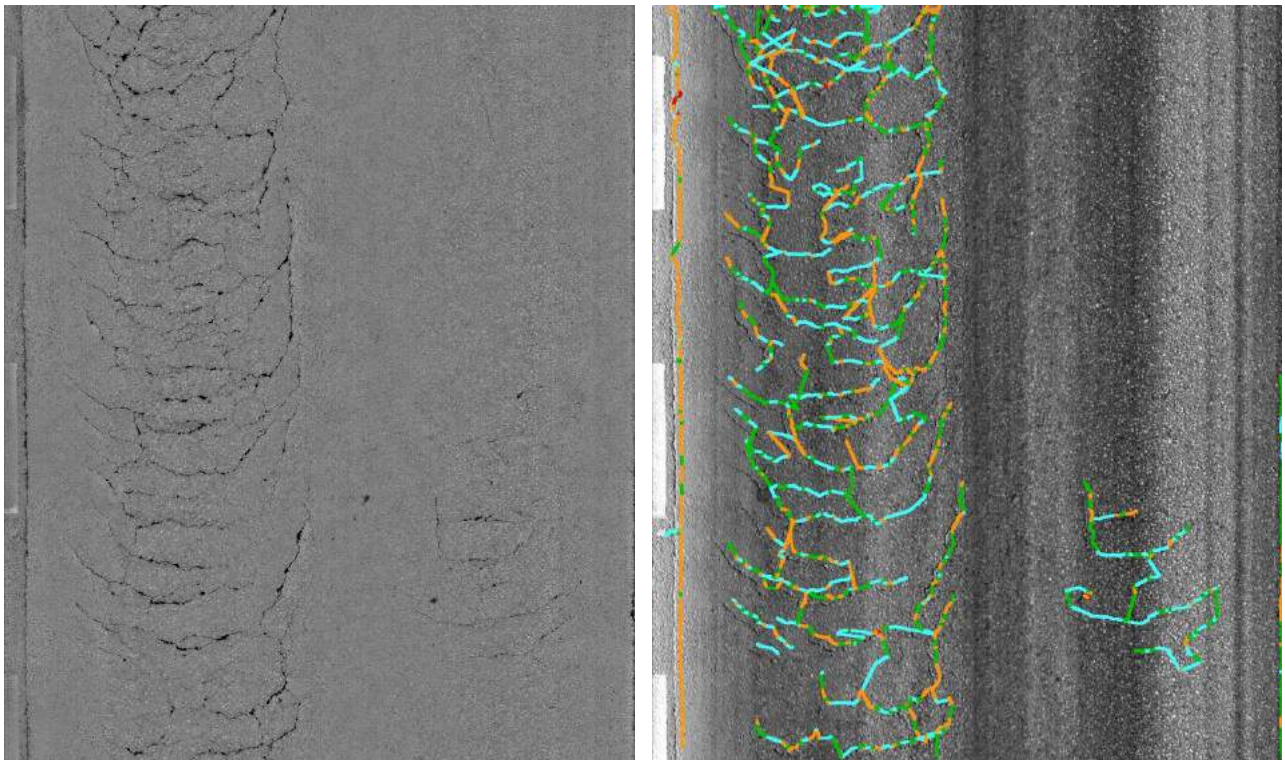


Figure 3: Range (left) and Overlay 3D (right)

An important characteristic of a pavement surface is texture. Texture does relate to friction, noise and ravelling and does change over time due to aging, contamination and loss of aggregate. As such it is an important characteristic triggering maintenance measures when it does not meet the requirements anymore. The standard procedure is to measure texture based on the Mean Profile Depth (MPD), which is a single point measurement along a longitudinal track sampled at a high frequency. The actual requirement is a volumetric 3D based result to be directly representative for the characteristics required without the need of a conversion from a 2D based test.

The results presented are sampled to validate and compare MTD's sampled at traffic-speed using a LCMS with the results of the static Sand Patch Test Method (SPTM). The LCMS does measure the volumetric properties of the road surface continuously over its full width and length based on 250 x 250mm squares. The evaluation did include static SPTM's and a static ELAtextur device measuring MPD. The test were conducted over different time periods and multiple runs at a validation site used for calibrating testing equipment operating at roads in Singapore.

2 TEXTURE

The texture of a surface layer is an important mechanical characteristic related to noise, friction and reduction of spray. The specific texture based wearing courses such as Stone Mastic Asphalt (SMA), Porous Asphalt (PA), Open Graded Friction Courses (OGFC) or Noise Reducing Road Surfaces (NRRS) do all have one major disadvantage: their service life is reduced due to ravelling. This is the main distress type that dictates the timing of maintenance. However, the development of ravelling is not linear model but closer to an exponential development of loss of aggregate which does make the prediction of the timing of maintenance very difficult. In addition to this the quantification of the severity of ravelling using a manual visual distress approach or using pictures is far from accurate and does lack consistency. This makes it as well difficult to notice the onset of ravelling. Each wearing course has its own surface characteristics (finger print) which make it almost impossible to notice a change when using a manual based approach.

The picture shown in Figure 4 does make the rating of severe raveling not that difficult but properly rating all the severity levels of ravelling of different surface courses does make it clear that the actual practice is much more difficult using a 'manual' based approach.



Figure 4. Observed raveling Expressway Singapore

Raveling is the loss of aggregate due to trafficking and /or environmental conditions. But the onset of raveling can as well be caused by segregation which does already take place during construction (McGhee *et al.*, 2003). Uniformity of production is as such not part of the compliance testing based on an objective procedure. Figure 5 taken from (McGhee *et al.*, 2003) shows a clear picture of what is meant by segregation.

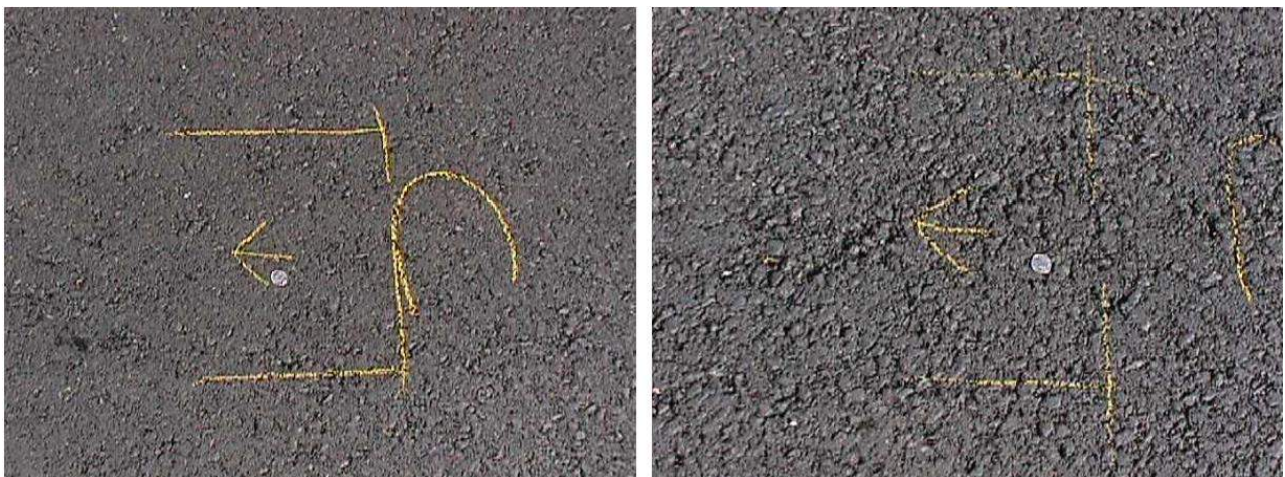


Figure 5. Segregation example (right)

Segregation is difficult to spot but LCMS data collected at roads in the Netherlands do show that it does exist but goes quite often unnoticed and when detected will be classified as raveling. The multiple 3D transverse profiles

collected by the LCMS sensors do allow for measuring texture as a volumetric characteristic instead of the by default used single profile longitudinal texture lasers.

The greater the distance from the sensors to parts of the pavement surface the more black the grey-scale range picture will be. The range image is the perfect tool to observe cracks, loss of aggregate (ravelling) and segregation. Figure 6 does give an example of segregation of a PA and shows the same lane but in between the pictures is about 25 meter. The segregation, visualised by the black speckles, had a length of about 10 meter but did grow in size each consecutive year as prove that segregation does trigger ravelling at an early age.

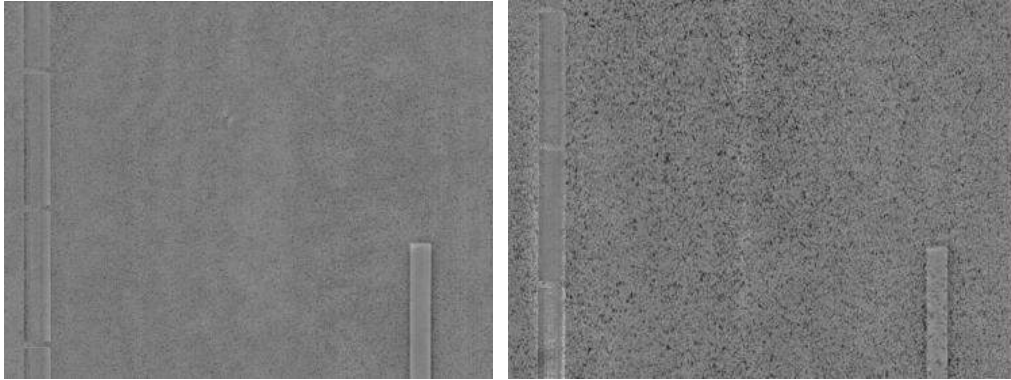


Figure 6. Segregation on the right

The time between detecting ravelling and speed at which it will develop into a serious defect is quite often very short. This does make the accuracy of comparing differences between successive surveys extremely important requiring a (3D) procedure to measure the level of detail of the surface to identify loss of stone (McRobbie et al., 2015). The only way achieving this is using a volumetric mechanical based survey procedure to measure the condition of the surface objectively.

3 SAND PATCH VS LCMS COMPARISON

The SPTM (ASTM E965-96, 2006) is a static procedure for measuring the MTD by spreading a known volume of material over a pavement surface. The MTD is calculated based on the sample volume divided by the average diameter of the area covered by the material. Figure 7 shows a picture of the execution of the SPTM at the validation site.



Figure 7. SPTM in execution

According to (ASTM E1845-09, 2009) an Estimated Texture Depth [ETD], a value close to MTD, can be calculated based on the equation: $ETD (MTD) = 0.2 + 0.8MPD$ to transform a 2D result into a volumetric SPTM result. Several publications can be found relating MPD's to ETD's, however number of publications does match the number of different relationships found which does show that this approach is not very accurate or relates only to the surface type used in the research.

The SPTM is a time consuming relatively inaccurate procedure requiring lanes to be closed for traffic and as such not very useful for measuring a change in the texture of a road surface over time. Twelve SPTM's have been executed in the validation site having a length of 300 meter, split over the wheel tracks and in-between the wheel tracks. Figure 8 shows the location of SPTM No 8 and the pavement surface automatically divided in 250 x 250mm squares over its total width and length.

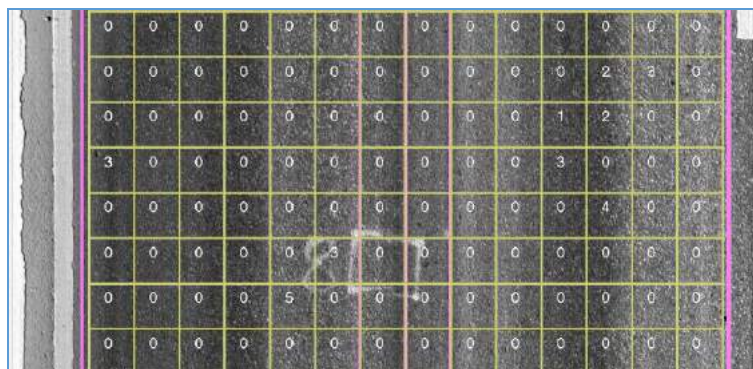


Figure 8. Location of SPTM No 8

The LCMS automatically detects the marking to locate the squares always at the same transverse location independently of the lateral wander. The index number in the squares does in this case relate to the presence of raveling. The SPTM as well as the ELAtextur results have been compared which those of the LCMS in calculating the MTD's of 250 x 250mm squares as shown in Figure 9. The computed R^2 of respectively 0.889 and 0.881 showing a very good linear correlation when taking into account the relative inaccurate procedure of the SPTM plus conversion of the ELAtextur MPD into MTD and the slightly larger area of the LCMS squares.

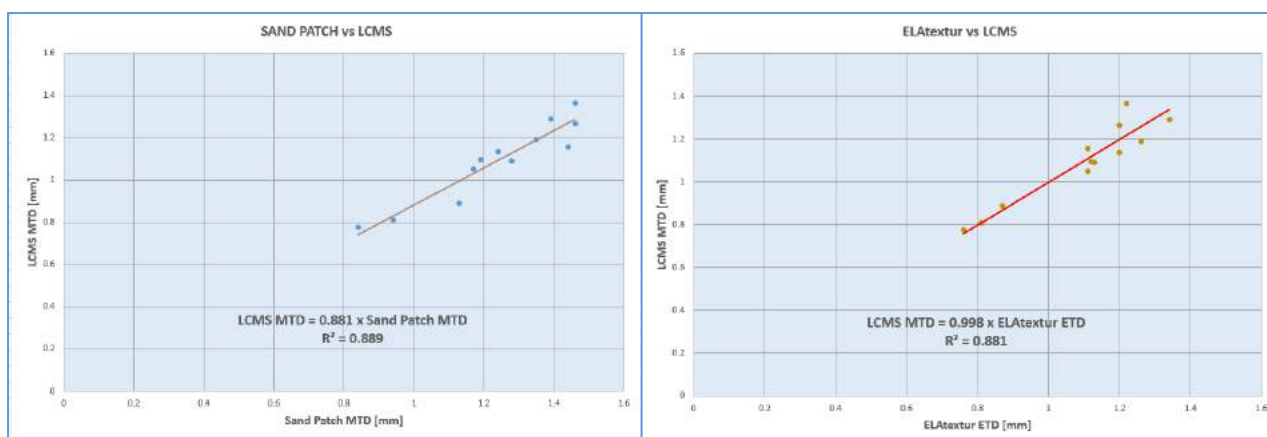


Figure 9. Correlation LCMS MTD with SPTM and ELAtextur

4 LCMS BASED MTD DATA

The width of a lane can be divided into multiple longitudinal bands based on usage. For instance AASHTO is recognising the central band, both wheel tracks and the bands outside the wheel tracks resulting into 5 bands. By default the central band width is 1000mm and both wheel tracks 750mm, however in the software analysing the LCMS data this can be adjusted transversely based on the requirements. The output of the analysis (MTD, MPD, and Raveling) can be averaged longitudinally for every 250mm. To match the position of a texture point laser in the Road Surface Profiler (RSP) attached to the front of the vehicle the central band width has been adjusted to 1500mm and the wheel track bands to a width of 250mm as shown in Figure 10.

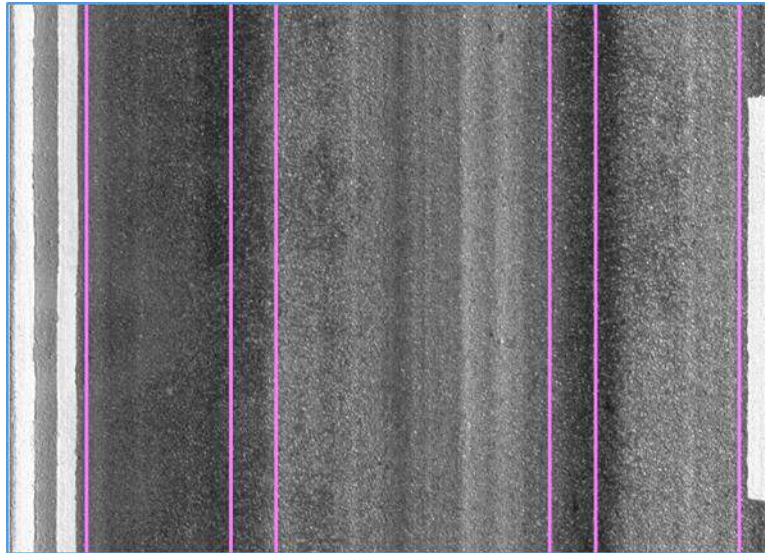


Figure 10. Bands as shown by the purple lines

To get a better insight in the repeatability of multiple LCMS runs over the same road section the results of the MTD's of 5 runs sampled at the validation site are shown in Figure 11. These MTD results are calculated for the left wheel track band (nearest to the verge) shown in Figure 10 being the average of a 250x250mm square.

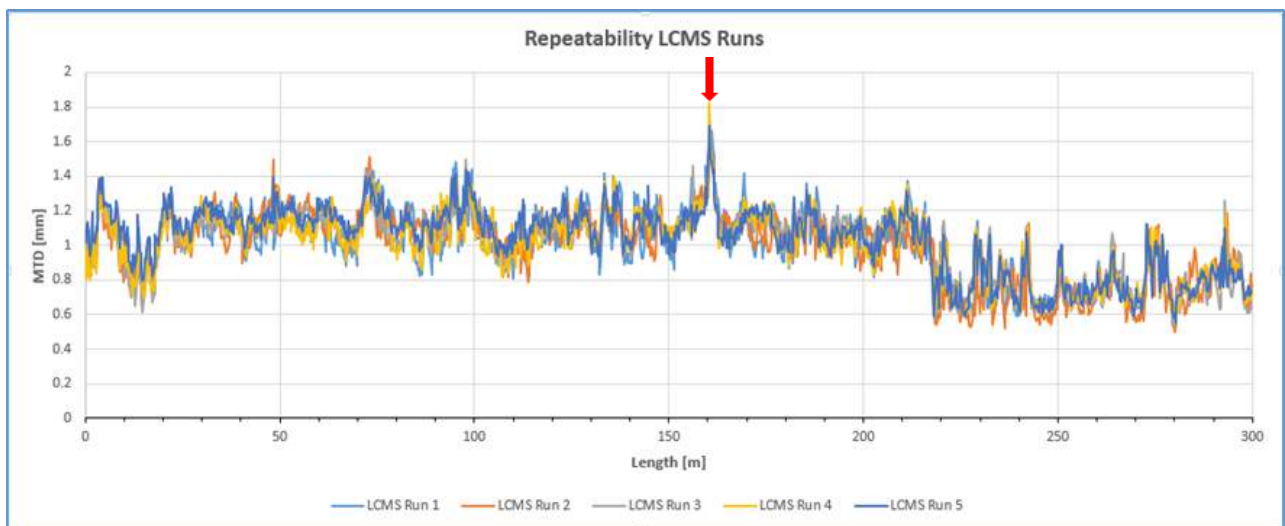


Figure 11. MTD results of 5 runs validation site

The quality of the repeatability is checked by comparing each run against the average of all runs. This does show that all runs are comparable having a very good repeatability with R^2 -values of respectively 0.854, 0.831, 0.900, 0.917 and 0.936. Figure 12 does show the repeatability of run 5 as example.

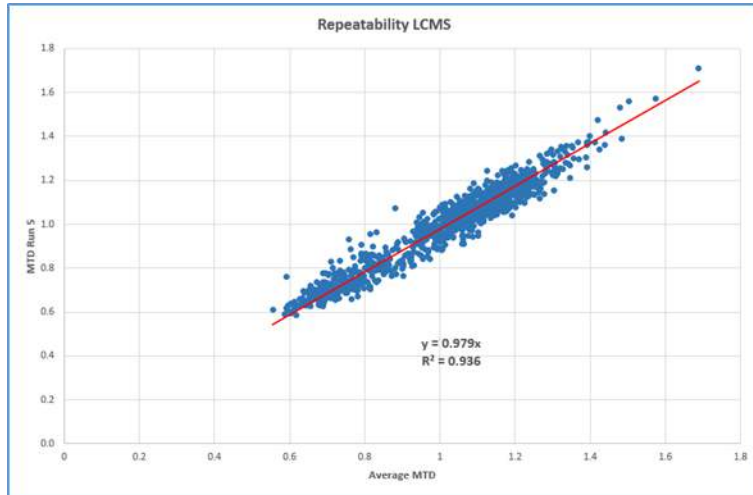


Figure 12. Repeatability MTD run 5

The validation site does show that the wearing course is having two different textures types. The 3D images of the LCMS do make this clear by a construction joint at approximately 218m which was difficult to see based on ROW images only. Of the 12 SPTM's 3 are located in the section after 218m (the 3 lowest MTD values shown in Figure 9).

The validation site has a dense mix surface course showing only locally loss of aggregate. There was however one distinct small location at approximately 160.5m in the left wheel track (band 2 250mm wide) showing a higher severity level of raveling. Figure 13 does show this location with in the top part a camera picture of this location and the range image overlaid by the 250mm raveling indices (94) as the bottom part. Figure 11 shows the same location as a peak in the MTD-values (see red arrow) at a distance of approximately 160.5m from the start of this section.

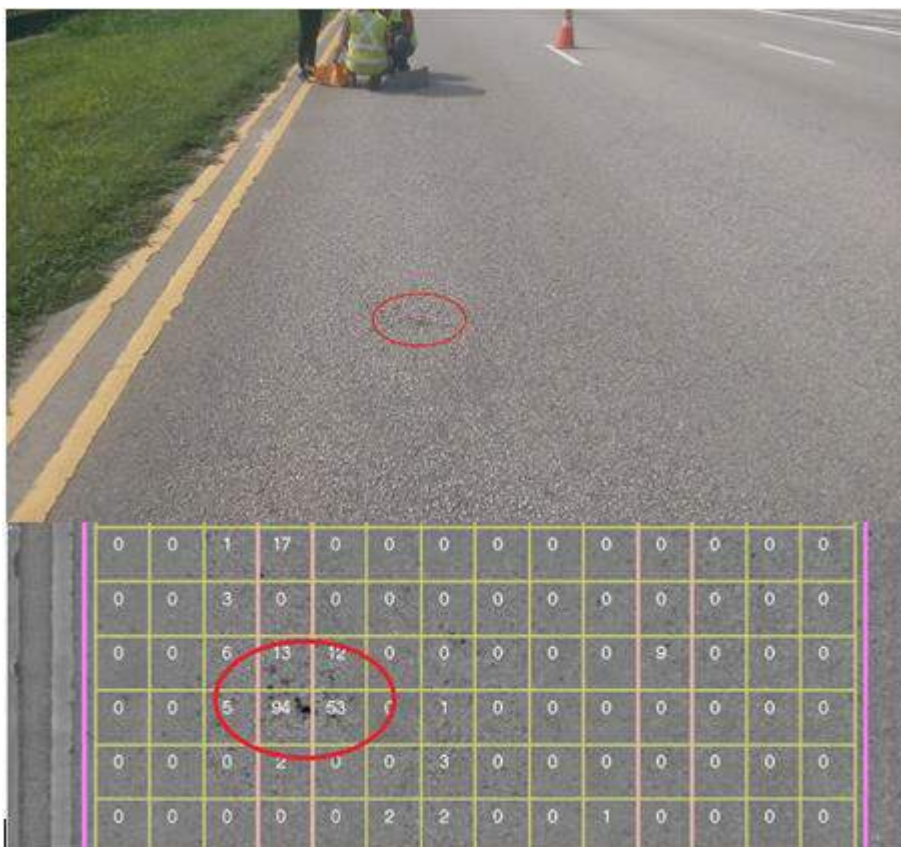


Figure 13. Detected raveling

A second set of data was collected at the Expressway shown in Figure 4 having an OGFC as surface layer. There was a very pronounced sub-section showing high severity raveling. In Figure 14 the measured texture is presented based on the MTD as sampled by the LCMS.

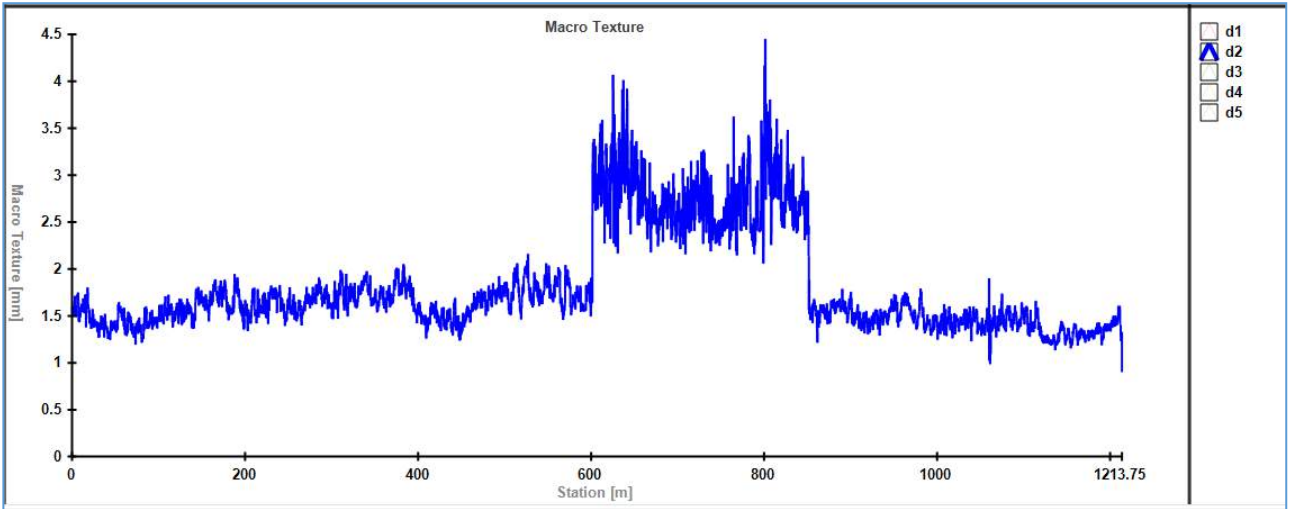


Figure 14: LCMS MTD

To have a check on the repeatability of this OGFC surface course over the same road section the results of the MTD's of 5 runs over a length of 300m (375-675m according to Figure 14) are shown in Figure 15. The LCMS based MTD results are calculated for the same left wheel track band (nearest to the verge) showing a very good repeatability (average R^2 around 0.95). The sub-section with the high severity raveling is of course showing more variation in the results.

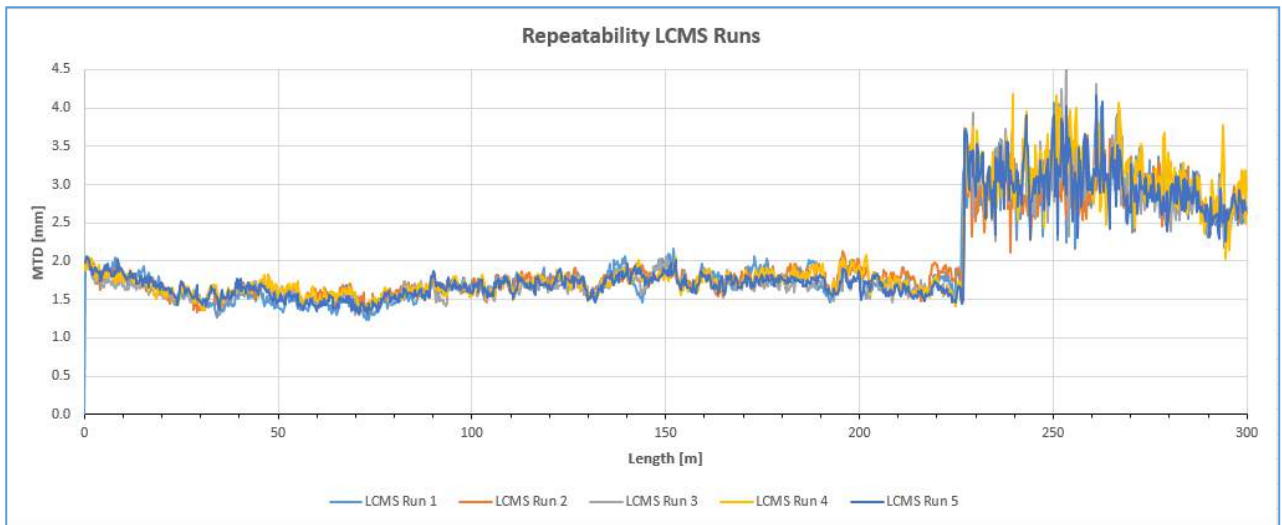


Figure 15. MTD results of 5 runs sampled at the Expressway

The LCMS data of the OGFC surface course has been analyzed for rating the level of raveling. In Figure 16 a range image is shown located in the first 225 meter of Figure 15. The top part does show the range image without overlay whereas the bottom part is showing the raveling index per 250 square. The range in index values is representative for the variation in texture. A shift factor has to be used to correct the index into an absolute level of raveling which will require the 'finger print' of the type of surface course as well as its initial texture value/porosity.

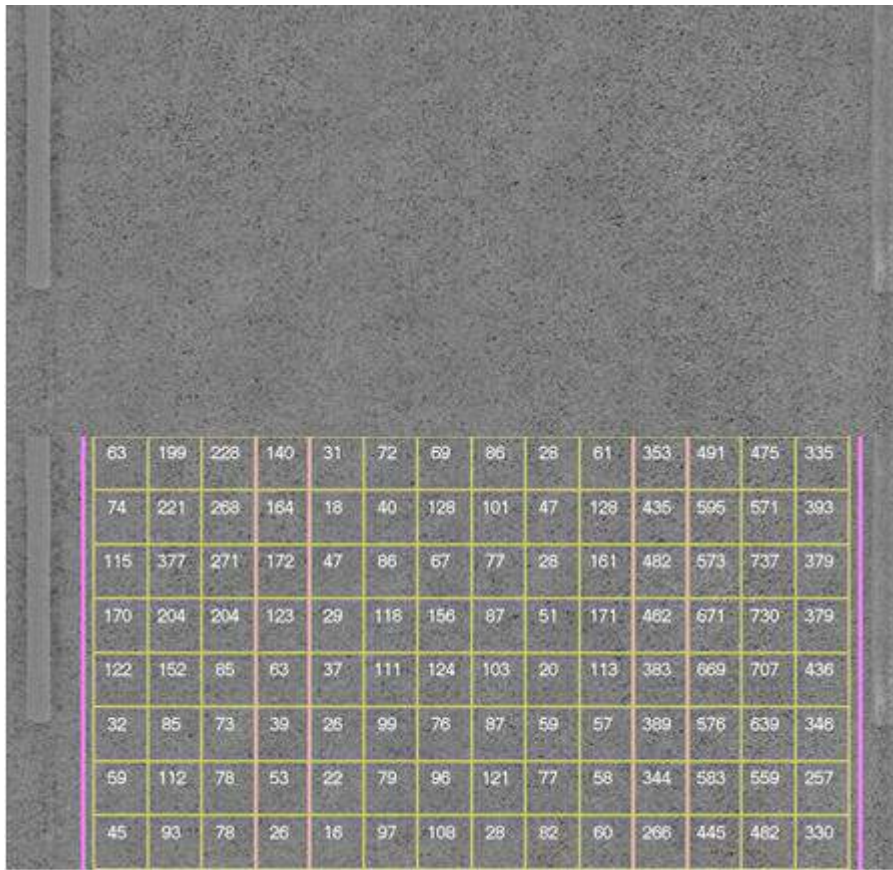


Figure 16. 'Non-raveled' range image

Figure 17 does show the same OGFC but at the transition of the 'non-raveled' to the raveled sub-section (see Figure 4). The majority of the types of surface course do have a MTD ranging between 0.5 and 2.0mm being shifted by the level of raveling. In a process of surveying a road network data of different types of surface courses with a difference in age are periodically sampled. This database of surface courses will allow for a detailed analysis of the variation of the texture and level of raveling.

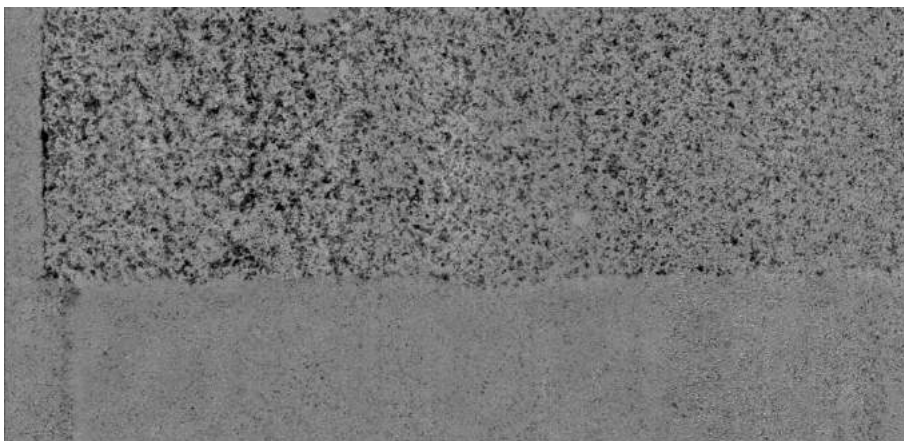


Figure 17. Transition from 'non-ravelled' to ravelled

5 CONCLUSIONS

When (macro) texture is discussed the volumetric component (3D) is actual fact being meant instead of the single point laser (2D) value. As a reliable LCMS-based volumetric measurement of texture at traffic-speed is now available a move from MPD to MTD should be aimed at to overcome the inconsistency of multiple correlations. Based on LCMS data analyzed over the last 5 years the following conclusions and recommendations can be made:

- LCMS texture data collected over the total width and length of road sections does show that there is much more variation than thought, starting already when a pavement surface is new.
- Windscreen surveys or HD pictures are capable of giving only a relative subjective rating of the level of raveling and are unable to detect changes in texture.
- Segregation of a surface layer does take place more often than anticipated and an objective traffic-speed based approach measuring MTD should be made part of a compliance testing procedure.
- The variability of texture will have a negative influence on the noise reducing characteristics of a surface course.
- The variability of texture will influence the sensitivity for raveling, influenced by the mix quality, construction procedures and the weather conditions during construction.
- The variability of texture will influence the frictional characteristics of a surface course.
- The LCMS measured MTD data is compatible with the SPTM and highly repeatable.
- All defects are detected and progression of the severity can predicted based on the results between consecutive surveys.

In the next phase the rutting and IRI as measured with the LCMS will be validated.

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PAPER TITLE	Design, Production, Construction and Performance Evaluation of Recycled Asphalt Concrete		
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ABSTRACT:

With increased demand and limited aggregate and binder supply, hot mix asphalt (HMA) producers realized that reclaimed asphalt pavement (RAP) is a valuable component in HMA. As a result, there has been renewed interest in increasing the amount of RAP used in HMA. While a number of factors drive the use of RAP in asphalt pavements, the two primary factors are economic savings and environmental benefits. RAP is a useful alternative to virgin materials because it reduces the use of virgin aggregate and the amount of virgin asphalt binder required in the production of HMA. Recycling of high RAP content mixtures could further preserve more natural resources such as virgin aggregate and asphalt binder. The majority of highway agencies are concerned with how the high RAP content influences pavement performance. Three test sections were constructed in this study to determine if the higher percentage of RAP materials can be used on highways. This paper presents the design, production, and construction and preliminary performance data, discusses the lessons learned from this experience and develops quality guidelines for inclusion of high RAP content. To meet mix design requirements for mixtures with high RAP contents, it was necessary to fractionate the RAP materials. Mixtures containing up to 40% RAP were successfully designed, produced, and constructed after proper procedures were followed and attention to detail was paid during design, production and construction. An in-depth investigation of pavement performance involving three typical distresses, ride quality, friction and rutting, was conducted. The overall evaluation revealed that a well-design mix up to 40% RAP could perform as satisfactorily as that produced with virgin materials to meet the in-service performance requirement.

Design, production, construction and performance evaluation of recycled asphalt concrete

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1 INTRODUCTION

Asphalt recycling has become popular since the mid to late 1970's. Prior to this time relatively few asphalt mixtures were recycled since there was little incentive to recycle. As a result of the latest economic changes and the development of recycling technologies, there is a trend towards increasing the use of Reclaimed Asphalt Pavement (RAP) contents in asphalt mixtures. The use of increased RAP contents not only reduces construction costs, but also preserves natural resources such as aggregate, binder and energy. According to the European Asphalt Pavement Association (2010), over 100 million tons of RAP are generated each year worldwide.

Recycled asphalt concrete should be designed to produce a hot-mix asphalt mixture (HMA) having the same properties as that in a new mixture. The first step in designing a recycled mixture is to obtain samples of materials to be used in the recycled mixture. In a current practice recommended by the Asphalt Institute (2014) in the U.S. is that, when up to 15% RAP blended with a virgin binder, there is no change in binder grade. When 15% to 20% RAP was used, the new asphalt cement should be dropped on grade lower. When 25% or more RAP is blended with a virgin binder, a blending chart should be used to analyse the combined asphalt cement. Highway agencies in Switzerland allow using a maximum of 70% RAP in sub-base courses, 60% in base, and none in the hot mix surface courses (Bressi et al. 2015). In Taiwan, the Public Construction Commission relaxes its specifications to allow the use of up to 40% RAP in dense-graded asphalt mixtures.

There are five commonly used asphalt recycling methods, including hot in-place recycling, cold mix recycling, cold in-place recycling, full depth reclamation, and hot mix recycling (Santucci 2007; Bressi et al. 2015), the latter being the most common. In this method, RAP content, virgin aggregate, asphalt binder, and/or recycling agents are blended in a central mix plant to produce a recycled mix. Recycling agents are normally used to help soften aged RAP binder and restore the physical and chemical properties of the old binder. This research project focuses on the hot mix recycling approach.

Historically, highway agencies limit RAP in HMA based on RAP percentage by weight of total mix. The percentage of RAP binder blended with virgin binders, named asphalt binder replacement (ABR), is different from the percentage of RAP mixture in mixtures, as it was calculated as the percentage of RAP binder divided by the mixture's total binder content. To be truly sustainable and to reduce life-cycle cost, mixes with high ABR levels must perform equivalent to virgin or low recycle HMA. Because of the cost reductions for construction, the percentage of allowable ABR increases gradually. Up to 60% ABR would be allowed by some highway agencies (Maupin et al. 2009; Tran & Hassan 2011).

While many highway agencies increased the amount of RAP used in hot-mix asphalt mixtures (HMA), the use of high RAP mixtures is still not common. The major concerns involved in the use of high RAP contents in wearing courses are related to the quality of the combined binder and its performance during service life. The use of RAP as a raw material for asphalt mixtures involves the combination of an aged and a virgin binder. Combining RAP with virgin materials introduces unique challenges as the RAP materials contain aged bitumen, and are therefore often considered as inactive "black rock." However, "black rock" is not really inactive. Aged bitumen could be mobilized, and the blending of old bitumen and recycling agent

can be optimized so that RAP mixes with virgin materials thereby reducing the amount of new bitumen needed (Chen et al. 2015).

Although RAP use varies considerably, the average RAP content was estimated to be around 12% in the U.S. in 2007. The most recent survey in 2011 indicated little change in RAP use specified by highway agencies, while contractors did report a significant increase in mixtures using 20-30% RAP with no change in mixtures containing other percentages of RAP (Copeland 2011). For the purpose of this study, high RAP is defined as using 25% or more RAP in an asphalt mixture by weight of total mix. The majority of highway agencies in Taiwan require mixtures that incorporate RAP to meet all conventional mix design requirements. However, most highway agencies place restrictions on the amount of RAP used overall as well as in certain mix types and pavement layers. Conditions may be placed on the asphalt binder grade, aggregate type, and nominal maximum aggregate size for use with RAP.

As highway agencies are interested in being greener and using environmentally sustainable products, they are looking way to incorporate higher percentages of RAP into asphalt mixtures without sacrificing the pavement quality. The majority of the highway agencies are concerned with the greater potential for cracking due to the stiffer RAP binders. Highway agencies have expressed concern over the lack of guidance on the use of high percentages of RAP (high RAP) in mixtures as well as information on their performance. There is a need for guidance on best practices when using RAP and documented information about long-term performance of high RAP pavements. This study is the result of RAP design, production, construction and performance to provide information for including higher percentages of RAP in asphalt mixtures.

2 MIX DESIGN

The basic principle of the mix design is that mixtures with and without RAP meet the same requirements. The experimental factors included one new aggregate source, one RAP source, two RAP contents, one rejuvenator, one softening agent, and one virgin binder. Limestone aggregate was used for dense-graded asphalt mixtures with the LA abrasion value less than 30% to possess sufficient toughness. The aggregate provided by the RAP was included in determination of the mix gradation and basic properties. In addition, flat and elongated proportions were limited to be a minimum value, and fracture faces were required to provide an aggregate structure with high internal friction. Limestone met the current criteria of no more than 15 percent 3 to 1 and no more than 5 percent 5 to 1 flat and elongated particles.

Fractionation is the act of processing and separating RAP into two sizes, i.e., a coarse fraction (+12.5 mm) and a fine fraction (-12.5 mm). A fine RAP gradation was used in this study because, when RAP particles are coarse or oversized, they are likely not breakdown completely during the mixing and placement. For the RAP stockpile, the asphalt binder content and aggregate gradation were examined. The asphalt binder content of RAP was determined to be 4.5% according to AASHTO T 164. Aged asphalt was recovered using trichloroethylene to separate bitumen from mineral aggregate according to AASHTO T170. The recovered binder was then characterized by the viscosity, the penetration value and the softening point, as presented in Table 1. As specified by the Public Construction Commission, the aged binder content inside RAP and the penetration value of old bitumen is required to be higher than 3% by weight of RAP and 15 dmm, respectively. The RPA material used in this study met the above specifications.

Table 1. Properties of aged binder

	Specific gravity	Viscosity (60°C, poise)	Pen. (25°C, dmm)	Soft. pt. (°C)
Aged binder	1.02	120,800	16	72

Table 2. Properties of virgin asphalt and recycling agents

Source	Viscosity (60°C, poise)	Pen. (25°C, dmm)	Soft. pt. (°C)	Saturate (%)
AC-20	1986	67	51	na
AC-10	972	89	46	na
RA	65	na	na	12

na = not applicable

Two types of recycling agents were used to restore the aged binder. The two agents used were one type of soft asphalt cement and one type of oil-based rejuvenators. The softening agent was AC-10 viscosity grades. An AC-20 binder was selected as the control binder because this is typical asphalt used for flexible pavements in Taiwan. Table 2 gives the physical properties of the softening agent and AC20 asphalt cement used in this study. The rejuvenating agent used in this study was RA which is a petroleum hydrocarbon. Table 2 summarizes the basic properties of the rejuvenating agent.

For asphalt mixtures containing RAP, a blending chart was used to select the appropriate grade for the virgin binder. The blending chart developed by the Asphalt Institute (2014) is logarithmic linear graph, and was used to estimate the viscosity of the blended binder. The selection of 2,000 poises at 60°C as the target viscosity is due to the fact that most highway agencies in Taiwan choose this value for recycled asphalt concrete. Two recycling agents, i.e., AC-10 and RA, were used to restore the old binder to the AC-20 classification range.

Asphalt mixtures were prepared by mixing RAP, fresh aggregate, virgin binder (AC-20), and recycling agent. The gradation of the RAP aggregate was determined after the aged binder was extracted from the RAP mix. Two RAP contents were evaluated in this study, i.e., 10% and 40%. Figure 1 illustrates the blended gradations of RAP mixtures. The master gradation bands for 19-mm nominal maximum aggregate size (NMAS) were based on the field test road selected. For clearer presentation, mixtures mixed with 0%, 10% and 40% RAP were designated as RAP0%, RAP10% and RAP40% mixtures, respectively. Satisfactory inclusion of up to 40% RAP was achieved for well-designed mixtures with good gradation characteristics, and with the use of recycling agents. RAP0% represents virgin aggregate mixed with AC-20, and is used as the reference mix.

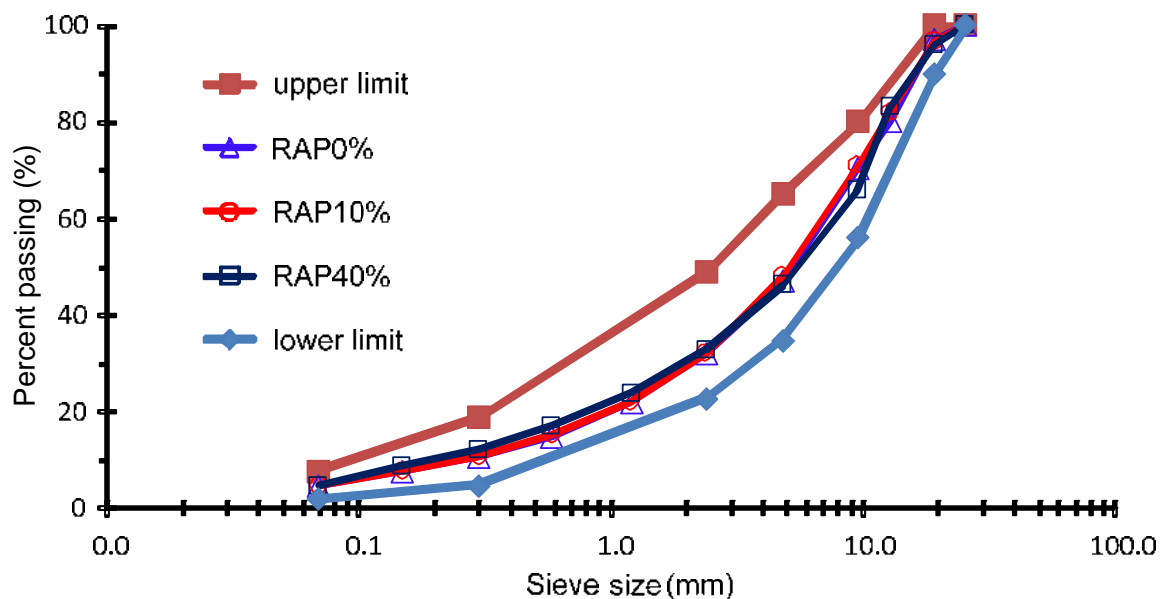


Figure 1. Gradation curves of virgin and RAP mixtures.

Table 3. Various binder contents in asphalt mixtures

Source	ID		
	RAP0%	RAP10%	RAP40%
Virgin binder (AC-20)	4.9	0.4	1.5
Softening agent (AC-10)	0.0	4.0	0.0
Rejuvenating agent (RA)	0.0	0.0	1.6
Aged binder	0.0	0.5	1.8
Total binder	4.9	4.9	4.9
Binder replacement	0.0	10.0	36.7

All mixtures had similar blended aggregate gradations, and the percentage of mineral fillers used for all mixtures was set at 5 ± 0.2 percent. The job mix formula was decided using the Marshall mix design method. Specimens of 100mm diameter and 63.5mm height were prepared by applying 75 blows on each face. The asphalt content of all mixtures was determined as that resulting in 4% air voids. The total binder content for all asphalt mixtures was determined to be 4.9% by weight of total mix. The asphalt binder replacement (ABR) level in the experiment varied from a low level of 10% to a high level of 36.7% as listed in Table 3. In this study, voids in mineral aggregate (VMA) of asphalt mixtures were controlled at 14.8 ± 0.2 percent. The mixing and compaction temperatures for all asphalt mixtures were selected corresponding to 0.17 Pa.s and 0.28 Pa.s viscosities, respectively. Laboratory-produced mixtures may not reflect the effect of all of factors affecting high RAP content materials, evaluation of plant-produced mixtures would be more elastic.

3 PRODUCTION OF RECYCLED ASPHALT MIXTURES

After the asphalt mixtures were designed and checked, all mixes were produced according to the job mix formula (JMF). Since the moisture in a RAP stockpile would take more fuel for drying, the RAP stockpile was covered to reduce the amount of moisture in the RAP as shown in Figure 2. The moisture content of the reclaimed material was kept low by preventing rain from falling on the RAP by keeping it under a roof. Due to the inherent variability of RAP materials and uncertainty of RAP aggregate true gradation, actual blend gradation against the design blend was regularly checked.



Figure 2. Covered RAP stockpiles.

Production of recycled asphalt mixtures is similar to that for virgin asphalt mixtures. RAP is basically treated as an aggregate with some modification to the way it is fed through the asphalt plant. A batch plant is used in this study and has to be modified, as shown in the schematic in Figure 3, to add the RAP directly to the pugmill. The modifications require the addition of RAP cold-feed bin, RAP conveyor, RAP dryer, RAP weigh hopper, and the appropriate instrumentation to control the RAP mix. The feed of the RAP was separated from the feed of the new aggregate. The material from the bind was deposited onto a gathering conveyor for transport to a RAP dryer. The RAP was indirectly heated using the high-temperature exhaust collected from the burn set up for the virgin aggregate. The RAP entered the pugmill through the weight hopper.

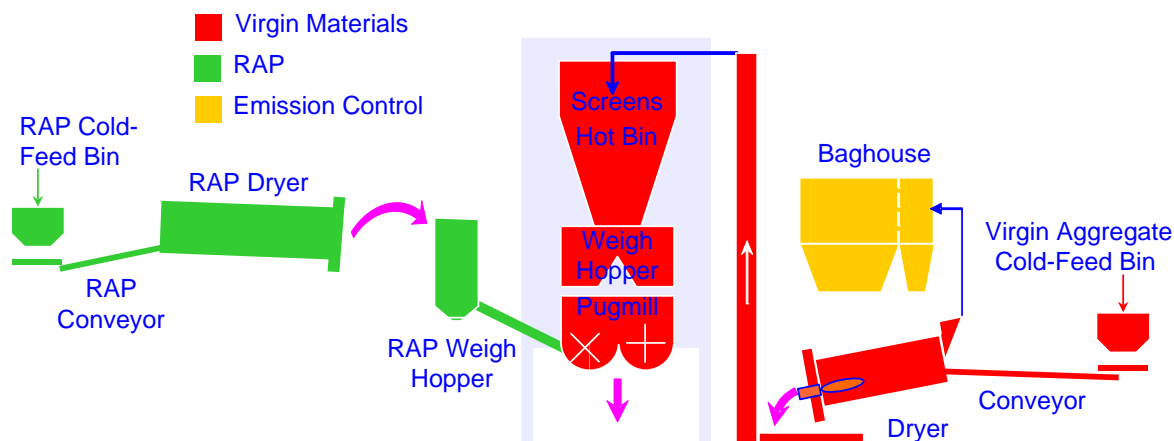


Figure 3. Major components of a batch plant for production of RAP mixtures.

Three components of a RAP mixture need to be heated, i.e., fresh aggregate, binder and RAP. Establishing the sequence of putting materials into the pugmill is chiefly aimed at producing the final homogeneity of a RAP mix. Mixing constituents consist of dry mixing and wet mixing. The wet-mix time began as soon as the rejuvenator was added to the pugmill. Wet-mix time was necessary to uniformly distribute the rejuvenator RA and coat the RAP. RAP wet-mix time is important to allow the rejuvenator to penetrate and diffuse into the inner circle of aged bitumen, and to provide a good degree of blending. The total mixing time of a cycle for RAP0%, RAP10% and RAP40% mixtures was 60, 70, 95 seconds, respectively, and included actions shown in Figure 4 for the production of RAP40% mixtures.

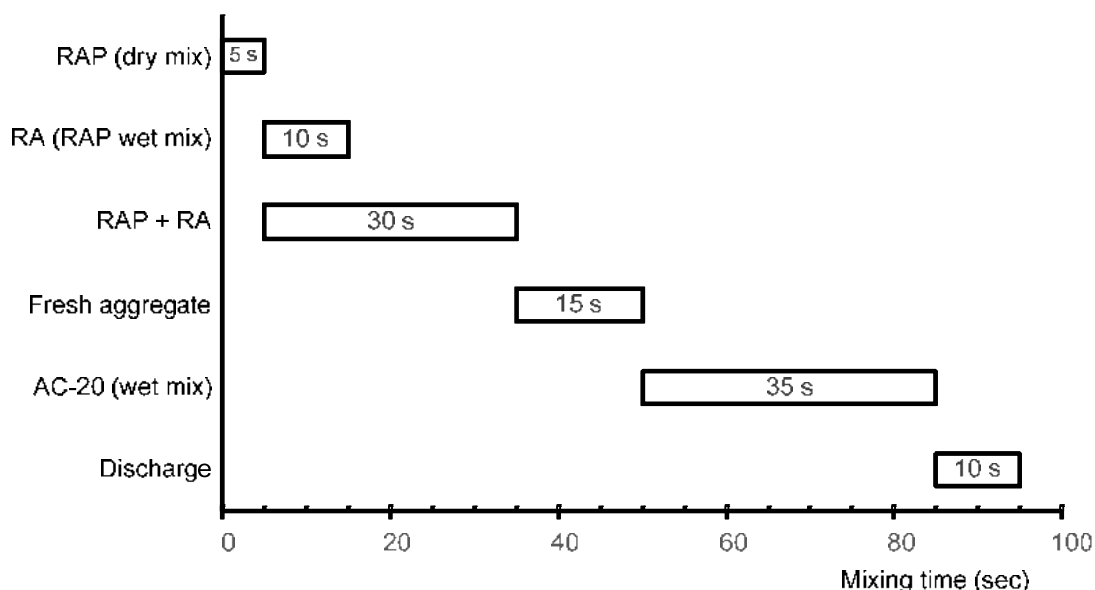


Figure 4. Batching sequence of RAP40% mixtures.

As shown in Figure 4, RAP40% dry mixing started the moment RAP was deposited into the pugmill, and it ended the moment the rejuvenator injection began. RAP wet mixing started when RA entered the pugmill. Typically 10-second injection time was needed for all the rejuvenator to be discharged from the weigh bucket. Wet-mix time at this stage should be long enough both to coat the RAP completely with the rejuvenator, and to allow adequate blending between aged asphalt and recycling agent. In this case, a 30-second wet-mix time could uniformly distribute the rejuvenator and coat the RAP. Enough RAP wet-mix time enabled the RAP binder to act as part of the cohesive binder. A significant amount of blending, if not total blending, could occur during production. After the addition of the RAP, the following sequence of

mixing fresh aggregate and virgin bitumen (i.e., AC-20) was similar to that of new asphalt mixtures (i.e., RAP0% mixtures).

The total cycle time to produce the RAP40% mix was 95 seconds (assuming a 10-second gate-opening, mix-discharge, and gate-closing time). The increased cycle time (95 seconds compared with 60 seconds for the RAP0% mix) would decrease the amount of mix produced in a pugmill with a capacity of 2 tons from 120 tons to 76 tons per hour. Thus, the dry- and wet-mix times could significantly affect the amount of a RAP mix produced by a given plant and the cost of producing that mix.

In a batch plant, the new aggregate temperature upon discharge from the dryer was about 165°C in this study to keep from driving off internal moisture in the aggregate. Since the amount of RAP incorporated into the mix was 40%, the RAP temperature was heated up to 140°C to remove the moisture from the RAP and to heat the moisture to the desired discharge temperature. The mix discharge temperature was about 155°C. The plant production was monitored and adjusted for moisture content.

4 CONSTRUCTION OF TEST SECTIONS

Accordingly, after verifying test results obtained from the asphalt plant, the Taiwan Highway Bureau began construction of a test road. These test sections as shown in Figure 5 were located along a county road. Pavement sections in the east bound were selected for the field study. Haul time from the batch plant to the construction site ranged from 40 to 50 minutes over a distance of approximately 45 km depending on traffic conditions. The route traverses plan terrain and a two-lane route having approximately 3.5-m lanes. The existing lane was milled prior to a truck and a paver. A vibratory steel roller, a pneumatic tire roller, and a statics steel roller were used a breakdown, intermediate, and finish roller, respectively. All the sections were completed with typical equipment and operations similar to those for other flexible pavement construction. Each section was consisted of a 5-cm dense-graded asphalt concrete surface course over a 30-cm crushed-stone base course. Cores were taken for quality control and quality assurance. All test sections were reported passing and within the acceptance range for compaction (98% to 100% of the reference density). An in-place air voids target to 8% or lower was achieved against each type of reference density.

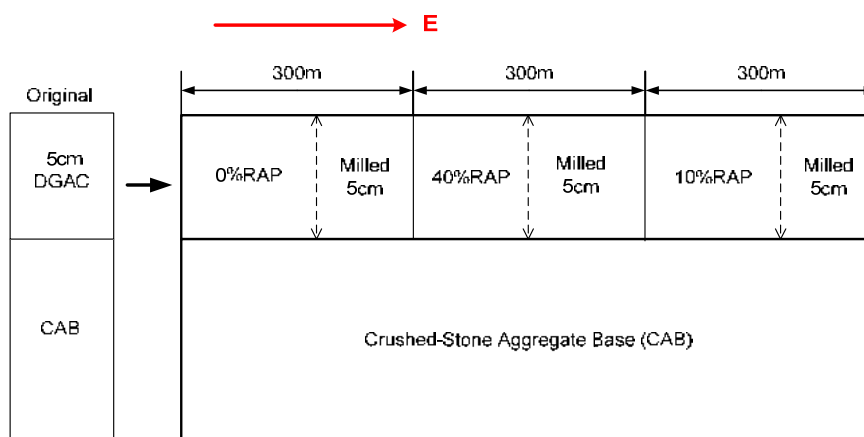


Figure 5. Test sections of virgin and RAP mixtures.

Monitoring in-service pavements is one of the best methods for gaining data on how pavements built with high RAP contents will perform over time under real environmental and traffic conditions. Three test sections named as RAP0%, RAP10%, and RAP40% were completed in 2014 with typical construction equipment and operations. This route has an average traffic volume of 1,200 vehicles per day with approximately 12% truck traffic. Surveys of pavement performance including skidding resistance, permanent deformation, and smoothness were conducted at scheduled intervals. The pavement performance of RAP sections is examined based on a comparison with the reference section, i.e., HMA with virgin materials.

5 FRICTION

Pavement skid resistance was measured by the British Pendulum Tester according to ASTM E303, and expressed by a British Pendulum number (BPN). Figure 6 shows that the measurement of the friction showed an initial BPN of 63 to 64 right after construction. Each bar represents the average of three replicates, and the error bars indicate on standard deviation from the average value. Skid resistance was relatively low just after construction because of the bitumen film coating aggregate particles on the pavement surface. As a consequence of the disappearance of the binder film covering the surface of the aggregate, skid resistance was improved after test sections open to traffic. A BPN value higher than 45 is considered sufficient and safe for roadway pavements. According to test results in Figure 6, all test sections provide reasonably good wet weather friction. The BPN value of test sections built with RAP appears to provide good skid resistance after two years in service. Because of the significant amount of macrotexture produced on RAP pavement surfaces, RAP layers maintained adequate frictional characteristics even after become condensed.

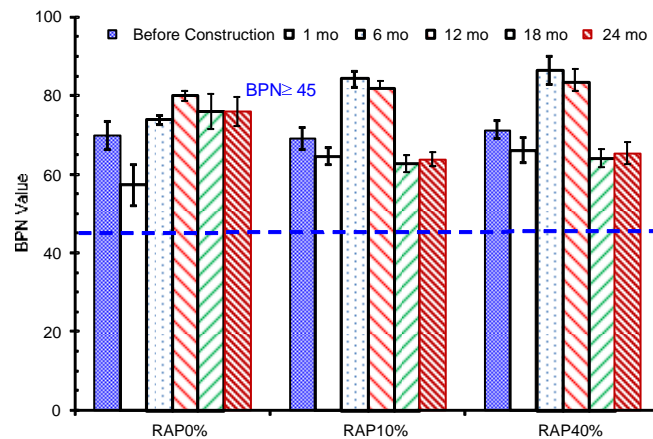


Figure 6. Friction characteristics of pavement surfaces.

6 RUTTING

A straightedge was placed across the wheelpaths at each trench location to measure rut depth according to ASTM E1703. As shown in Figure 7, rutting was reduced right after construction. The severity level is considered to be low when the mean rut depth is less than 12.5 mm, moderate when rutting is between 12.5 and 25 mm, and high when rutting is higher than 25 mm. Test results indicate that all test sections possess good resistance to plastic deformation, although rutting increases with increasing service time. Linear blending appears to be applicable to the high RAP content mixes. The presence of RAP binder is shown to be beneficial to maintain the mix stiffness even with the addition of a softening agent and a rejuvenator.

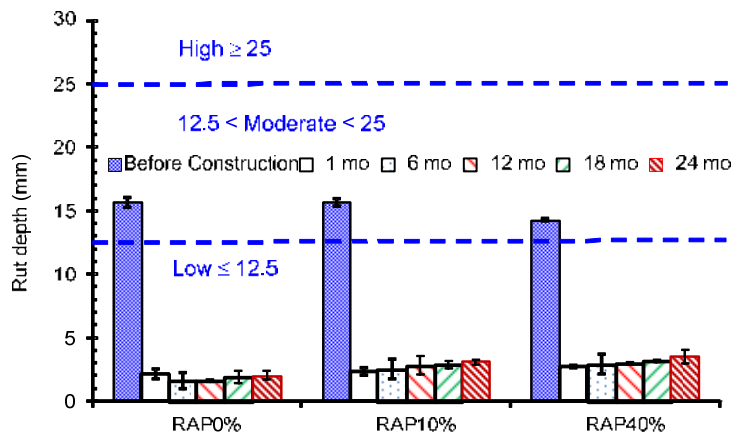


Figure 7. Permanent deformation on pavement surfaces.

7 RIDE QUALITY

Roughness is an important index of pavement performance evaluation, which affects the comfortableness of drivers and passengers. It is an index involving human-vehicle-road interaction, often evaluated by the International Roughness Index (IRI) and expressed by m/km. For these sections, the IRI value increases with increasing service time as shown in Figure 8. The initial IRI value is considered to be acceptable if it is lower than 3.5 m/km right after pavement construction. The IRI value of the RAP0% section was the lowest, while the roughness of the RAP40% section was larger than that of RAP10%. No raveling or cracking has been observed on the test sections since open to traffic. Field results suggest that RAP be a viable pavement surface type for use on roads to provide good performance, including good friction, reduced rutting, and improved durability.

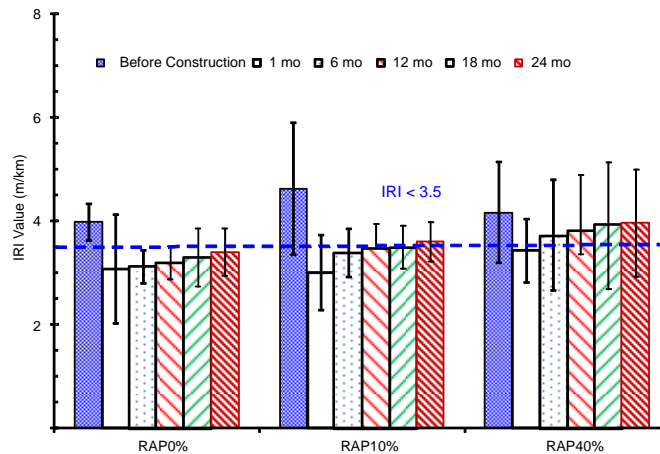


Figure 8. Ride quality of pavement surfaces.

8 CONCLUSIONS AND RECOMMENDATIONS

This study is to evaluate the design, production, construction and performance of hot-mix asphalt (HMA) pavements built with up to 40% reclaimed asphalt pavement (RAP). Three distress indicators, i.e., rutting, friction, and smoothness, are adopted to represent pavement performance. Based on the information and the limited test results, the following conclusions and recommendations could be made.

- For all RAP mixtures, the RAP materials were fractioned similar to virgin aggregates. The RAP material was properly characterized for mix design purposes. The laboratory mixture design was established using RAP as a component. Linear blending appeared to be applicable to the high RAP content mixes. Satisfactory inclusion of up to 40% RAP was achieved for well-designed mixtures with good gradation characteristics and with the use of recycling agents. RAP quality was shown to be associated with proper techniques for obtaining, stockpiling, and processing RAP.
- Plant production practices used in the production and placement strategies during the construction of HMA need to address concerns when using high RAP contents. The plant production was monitored and adjusted for moisture content. High-quality high RAP mixtures were achieved with processing and production practices, which resulted in mixes with good performance in rutting, friction and ride quality.
- Mixtures containing up to 40% RAP were successfully designed, produced and paved. Proper procedures were followed and attention to detail was paid during material selection, mix design, plant production, and field construction.
- The performance evaluation was conducted based on the distresses of the RAP sections and the virgin section, which served as the reference. Mixes with high RAP contents performed equivalent to virgin HMA. Overall, RAP and virgin sections showed satisfactory performance over two years of the service period.

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PAPER TITLE	SUSTAINABLE AIRPORT PAVEMENT MANAGEMENT SYSTEM: Effects of delay in maintenance approach based on life-cycle cost analysis and life-cycle assessment		
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ABSTRACT:

Airport pavements deteriorate with age due to environment influence and air traffic. The most challenging task in pavement management in developed countries is to maintain the performance of airport pavements through preventive maintenance while in developing countries maintenance is done on a reactive (require to do) basis. There is a dearth of studies on the scheduling strategy of airport pavement maintenance. Assessment of different scheduling strategies depends on the life-cycle cost analysis (LCCA). An analysis of the cost benefit study of delay in maintenance alternatives should be done to determine whether or not a one year delay is beneficial. It should be noted that the sustainability of airport pavement management system takes into consideration economics as well as environmental and social factors. This study reviews airport pavement management systems and the significance of delay in maintenance and preservation scenarios are demonstrated. The scenarios demonstrated are four preventive maintenance strategies, namely three methods of crack treatments, which are crack sealing, patching and slurry seal, and surface treatment in which the overlay method is done in two Pavement Condition Index (PCI 75 and PCI 65), and the resulting service life enhancement. This article discusses a one year delay in preventive maintenance from the life cycle cost and life cycle assessment perspectives. The results show that a one year delay in preventive maintenance increase the deterministic life cycle cost by 52% while the variance is between 50.5% - 53.6% in the probabilistic life cycle cost analysis. From the life cycle assessment point of view, a delay of only one year cause a 67.5% increase in NO_x, SO_x and PM.

SUSTAINABLE AIRPORT PAVEMENT MANAGEMENT SYSTEM:

Effects of delay in maintenance approach based on life-cycle cost analysis and life-cycle assessment

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1 INTRODUCTION

The construction and maintenance of airport pavements require a huge amount of public fund. In both industrialized and developing countries, airport pavements are the lifeline of fundamental economic activities through which goods and people are transported. The increase in air traffic is the main reason for the increase in load imposed on pavements with the passage of time, and this affects pavement structures which have either reached or depleted their service life. For this reason, airport management authorities have shifted their focus from constructing new pavements to maintenance and rehabilitation (M&R) activities. Planning future maintenance of fatigue pavement is becoming an increasingly difficult task due to the complex behaviour of aging pavement. At the same time the increasingly limited budget, which most agencies are currently enduring, has caused a decrease in maintenance activities. Most researches tend to focus on the advancement in airport pavement management systems (APMS) made by several agencies since the last couple of decades. The main role of APMS is to aid decision-makers in discovering M&R techniques for the maintenance of airport pavements which are in usable condition over a predefined period of time in the most cost-effective manner (Haas et al., 1994). Different APMS might adopt a different framework based on the preferences of agency requirements although there are several similar functions in all APMSs that are crucial in ensuring performance, such as pavement performance prediction, network inventory, pavement condition assessment, and planning strategies.

The first airport pavement management system was introduced in the 1970s as a project conducted by the Construction Engineering Research Laboratory (CERL) for the US Air Force and has led to the establishment of the PAVER and MicroPAVER systems. Harrison and McNerney (1995) conducted an assessment of different APMSs and concluded that the MicroPAVER system, which only looks at surface distress, is a time-consuming process which does not take into consideration the costs borne by airlines and airports, such as cost of user delay due to the closing of runways and expanded roughness which cause fatigue to airplane. A comprehensive economic impact on pavement network, which takes into consideration flight delay and operating costs, was addressed in the recently Enhanced APMS (EAMPS) (ARA, 2011). Broten and Wade (2004) performed a principled evaluation and assessment on airport pavement management activities by conducting a survey of all aviation agencies in every state in the US. The transportation agencies from several states, such as Michigan (MDOT, 1999), Minnesota (MNDOT, 2001), Ohio (ODOT, 2001) and California (CalTrans, 2008), prepared complete pavement maintenance manuals in their respective areas of authority. The Unified Facilities Criteria (UFC), published by the U.S. Department of Defence, contains related reports and technical guidelines on the management of pavement maintenance (USDOD, 2004). In addition to the above-mentioned researches, which are appropriate for both rigid and flexible airport pavements, other APMSs which are specific to other types of pavement are also available, such as the AirPACS which was published to answer various questions regarding to maintenance of jointed plain concrete pavement (JPCP) at airports (Ismail et al., 2009).

Airport pavement management simplifies the life cycle cost analysis (LCCA) for various alternatives and aids managers in the timing of framework and choosing the most effective alternative. It

provides a consistent, systematic method for choosing the maintenance required and identifies the preferred and the best maintenance schedule by predicting future condition of pavements (FAA, 2007; Santos and Ferreira, 2013). APMS can be an important tool which alert managers of the trigger condition of pavements and determine the proper preventive maintenance (PM) in a life cycle. The LCCA for a particular segment or project analyses for possible M&R are taken into consideration when determining the available alternatives for the pavement section and the most cost effective alternative for the life cycle of the pavement. The best progressive strategy for improving the allocation of available budget for several transportation agencies is the multi-year prioritization which utilizes incremental cost-effectiveness analysis; however this it is very rarely performed for airport pavement networks (Santos et al., 2015). The reason for the slow implementation include shorter airport pavement networks, other more significant performance issues at airports, and lack of airport pavement management programs (ACRP, 2011). A review of current studies on APMSs shows that only limited determination was made based on available data and information, with the objective of preparing a review of regular airport pavement maintenance activities and their current application.

As shown in Figure 1, maintenance cost might decrease significantly if maintenance was done during the early stages of deterioration. Preventive maintenance should be carried out before pavements reach their threshold performance level (base level of serviceability) and require reconstruction. The trigger value or threshold performance level is the base adequate execution level below which pavements are deemed unsuitable for its serviceable plan. This is frequently based on a particular agency’s implementation requirements or user point of view. Agencies usually use threshold values which are based on performance plan rather than time planning (Pinto et al., 2015). The effectiveness of short term prevention, which considers pavement performance jump, is invaluable in ensuring pavement performance and measures the viability of alternative maintenance. The effectiveness of long term prevention, which incorporates service life extension and average pavement performance and the area jumped by the performance curve, is suitable for preservation “strategies” or “schedules”, as shown in Figure1.

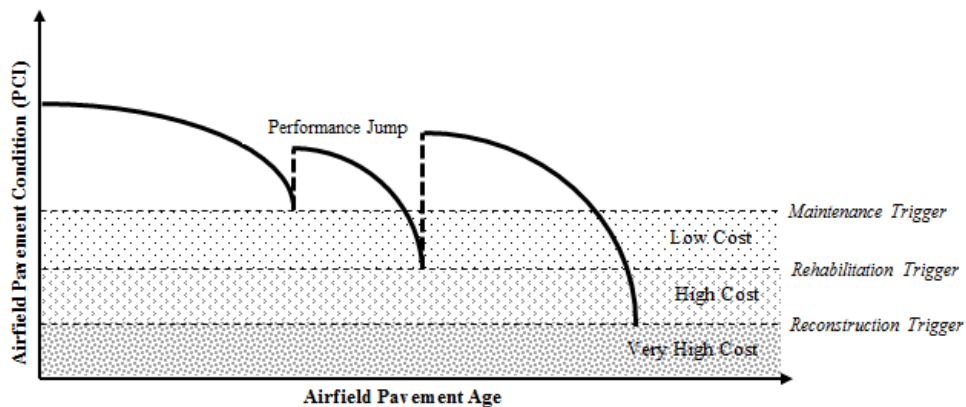


Figure 1- General life cycle of pavements performance curve (Babashamsi et al. 2016)

The principle components in all APMSs include network inventory, various methods of assessing and monitoring pavement condition, the framework used to predict the assessment of pavement condition, and management strategies. Conditions reports are presented in the form of quality index, such as pavement condition index (PCI) related to a single or a combination of various pavement characteristics and represent the deteriorating state of pavement during its service life (Baladi et al., 1992; Levy et al., 2014). The PCI process comprises of collecting broad data with regard to various distresses for the calculation of total index. The disadvantage of using the PCI method is the questionable repeatability of visual survey due to the subjective nature and distinction between severity of distress, which could be managed through broad and careful inspection rules or through the use of automated data gathering devices (Lee, 1992). Additionally, the functional assessments design and framework process of airport pavements depend on structural considerations such as constraining stresses, strains and deflections (Monismith, 1978).

In APMSs, several performance predicting methods play an important role in managing crucial decision-making process. Pavement performance serves as a tool in the planning of future M&R activities and can be anticipated through the deterministic and probabilistic methods (Lytton, 1987; Butt, 1991; Meneses and Ferreira, 2013). PCI prediction models are functional performance models that have generally been produced for the PAVER framework and are also used in other APMSs, such as Integrated APMS and AirPAVE (Rada et al., 1992). The methods created for performance prediction relate future PCI quality to a progression of informative or predictive variables such as age of pavement, deflection measurement, time since last overlay, and traffic. In order to determine the number of cycles of load applied before failure occurs, structural performance models relate the characteristics of pavement material structure to the load applied to it. These types of prediction models are widely used by pavement managers and have been expanded for pavements by different agencies, such as Asphalt Institute, U.S Army Corps of Engineers, Portland Cement Association (PCA), and Shell International Petroleum Company (Zaniewski, 1991; Meneses, 2013).

Currently, decisions on airport maintenance in developing countries depends on the historical experience of airport pavement experts and the judgment of management authorities without taking into consideration the LCCA and other management practices. Given the current situation of financial prudence, pavement infrastructure requires a more deliberate and systematic approach in determining maintenance requirements and preferences. Decision making agencies responsible for pavement maintenance often consider urgent requirement or experience instead of adopting long-term management and documenting data. This method does not allow management organizations to assess the cost effectiveness of various alternative maintenance strategies and this has led to imprudent spending. The most recent development in airport pavement management provides a way of managing pavement economically and in a timely manner (Liu et al., 2015). This paper aims to determine the most cost effective maintenance schedule method which would produce minimum life cycle treatment cost (Economic Benefit). It also takes into consideration environmental impacts by evaluating various scheduling strategies to determine which strategy has the least potential of contributing towards global warming (Environmental Benefit). This would empower airport management companies to determine pavement maintenance requirement through prudent spending of available funds.

2 METHODOLOGY

The surface distress of airport pavements is usually evaluated through PCI. The value of PCI could range from 0 to 100 and could be set as shown in Table 1. PCI distress data are obtained through a visual survey carried out by expert pavement inspectors. PCI depends on the assessment of the type, quantity and severity of distress (ACRP, 2011; Ferreira and Santos, 2013).

Table 1- Airport Pavement Treatments Based on PCI (ACRP, 2011)

PCI Rating	Description	Preservation Treatment
100-86	Good - minor distress	Routine maintenance
85-71	Satisfactory - medium/low distress	Preventive maintenance
70-56	Fair - Severe distress	Corrective maintenance/Rehabilitation
55-41	Poor - low operational problem	Rehabilitation
40-26	Very poor - high operational problem	Rehabilitation/Reconstruction
25-11	Serious - operational restriction	Reconstruction
10-0	Fail	Reconstruction

This paper considers two critical PCI. The first is the higher PCI limit (PCI 75) in which pavement is in satisfactory condition (medium/low distress). The second point is the lower limit of critical PCI, which is assumed to be the minimum acceptable PCI (PCI 65), and that pavement is in fair condition (severe distress) for all alternatives in the airport pavement management system models. Rahman and Tarefder (2012) stated that in high and moderate traffic airports these two critical PCI points could be reached within less than one year from one another. For a particular treatment alternative in this study, maintenance work was performed in the base year of 2015 and PCI 75 would be reached in 2017 and PCI 65 would occur in 2018, which means a one year gap within the same preventive maintenance activity. The following alternatives were considered for implementation during the 25 year pavement life cycle in the two different

PCI range for the current study. Crack treatments include crack sealing and spray patching, while surface treatment include slurry seal and thin overlay.

Crack sealing is a maintenance activity which seal cracks with rubberized bituminous material. It includes the routing of cracks, cleaning the routed surface, and applying sealant on them. Crack filling is similar to crack sealing, but without the routing. Crack filling is easily damaged by snow plows and hence is not cost-effective. Spray patching is a preventive maintenance measure which involves the application of a bituminous compound which is then covered with a layer of aggregate. It could be done manually or through the use special mechanical equipment to spray emulsion, apply the cover aggregate, and this if followed by preparation for basic compaction, all in a single pass. If spray patching is applied on the entire width of a pavement, it can be categorized as surface treatment. Slurry seal in itself is a surface treatment and uses unheated mixture of asphalt emulsion, graded fine aggregate, mineral filler, water, and other additives; these materials are mixed and spread over pavement surface as slurry. Slurry seal is applied to provide a protective layer of bitumen-rich mortar. Hot mix overlay of asphalt concrete pavement comprises of setting a layer of hot mix over existing pavement. Traditional asphalt concrete overlays are typically constructed with a base thickness of 2.5 inches (63.5mm). Overlays less than 2.5 inches are called thin overlay.

The assessment of various scheduling strategies depends on the LCCA. LCCA is an engineering economic analysis technique used to differentiate the relative monetary benefits of different development or rehabilitation design alternatives for a project. LCCA aids in the selection of the cheapest method which fulfil the objectives of a project. LCCA is used to determine the benefits achieved when all choices being considered are equal. The LCCA procedure starts with the development of alternatives to achieve the performance targets of a project. Initial and future activities required for executing each project design alternatives are then planned and the expenses for these exercises were assessed. In this study, only direct agency expenditures (maintenance activities) are considered while ignoring user costs that result from agency work zone operations. By utilizing a monetary technique known as discounting, these costs are converted into present value and are then added to each option. Two calculation approaches, namely deterministic and probabilistic, can be used in LCCA. The methods differ in the way they handle the variability associated with the LCCA input. Each LCCA input variable is assumed to have a fixed, discrete value in the deterministic method whereas in the probabilistic LCCA inputs are specified by probabilistic functions that convey both the range of likely inputs and the likelihood of them happening (Walls and Smith, 1998). In this study the integrated LCCA and life-cycle assessment (LCA) program, L2C3A2, is used for probabilistic LCCA. The [Triangular distribution](#) is taken to signify the variability of discount rate [as it was suggested by Walls and Smith \(1998\) in Interim Technical Bulletin. The discount rate can consider in Triangular or normal distribution the benefit of triangular is reducing the number of irritations and to achieve the results faster \(ARA, 2011\).](#) For all alternatives, 3%, 4% and 5% are chosen as minimum likely, most likely and maximum likely value of the discount rate respectively. The maintenance cost is assumed to have a normal distribution. [It would be more likely that the distribution of maintenance costs would be closer to a right skewed normal distribution and not uniform. This article followed ARA \(2011\) assumptions.](#)

The mean values of unit cost for crack sealing, patching, slurry seal, and overlay are assumed to be 1.79, 5.05, 3.57, and 8.93 \$/m², respectively. The standard deviation for the four alternatives were determined to be 0.6, 1.48, 1.20 and 2.23 \$/m² respectively.

The key objective of this study, namely to determine the maintenance schedule which would produce minimum life cycle cost, was achieved through the probabilistic life cycle cost analysis, and life cycle gas emissions by using a special tool named L2C3A2. [This software/program is comprehensive in considering LCCA and LCA as airport pavement management system toward sustainable development in Universiti Kebangsaan Malaysia \(UKM\) and will be available online soon. The novel integrated LCCA+LCA program is introduced by this research, namely, the L2C3A2 to compare alternatives not only in the economic aspect by considering direct costs, indirect costs and salvage values, but also in the environment aspect by various modules such as material, transportation, construction, usage, maintenance and rehabilitation, and end-of life, which covers every aspect of the pavement's life-cycle. This tool can be applied on both flexible and rigid airport pavements to focus on all areas of analysis. The results of this tool will be of assistance to engineers and airport management as it provides a fair, unbiased, defensible statistical method that evaluates the alternatives throughout the pavement life-cycle. This study attempts to determine the optimum pavement maintenance strategy. All benefits and costs were converted into monetary value. Information regarding pavements and their respective maintenance and their unit costs were obtained from the Airport Cooperative Research Program Synthesis \(ACRP, 2011\) and is shown in Table 2. PCI increase](#)

data was obtained from a previous study (Ningyuan et al., 2001). The unit cost of crack sealing is assumed to have a typical crack density of 0.25 m/m² (Hicks et al., 2000). In calculating total cost, all treatments were applied over the entire area as surface treatment, except for spray patching. Assumption is made that 50% of the area should be patched when patching is required.

Table 2 - Unit Cost and Life Extension of the Different Maintenance Strategy (Rahman and Tarefder, 2012)

Maintenance Activity	Life Extension (Years)	PCI Rise	Mean Unit Cost (\$/m ²)	Std. Dev. (%)
Crack Treatment	2-3	5	1.79	33.5%
Spray Patching	3-5	5	5.05	29.3%
Slurry Seal	4-7	10	3.57	33.5%
Thin Overlay	7-12	15-20	8.93	25.0%

In order to quantify the benefit of a pavement management in monetary terms, a guideline was obtained from a previous study (Smadi, 2004). The undiscounted expenditure timetable for minimum acceptable PCI 75 and 65 is shown in Figure 2.

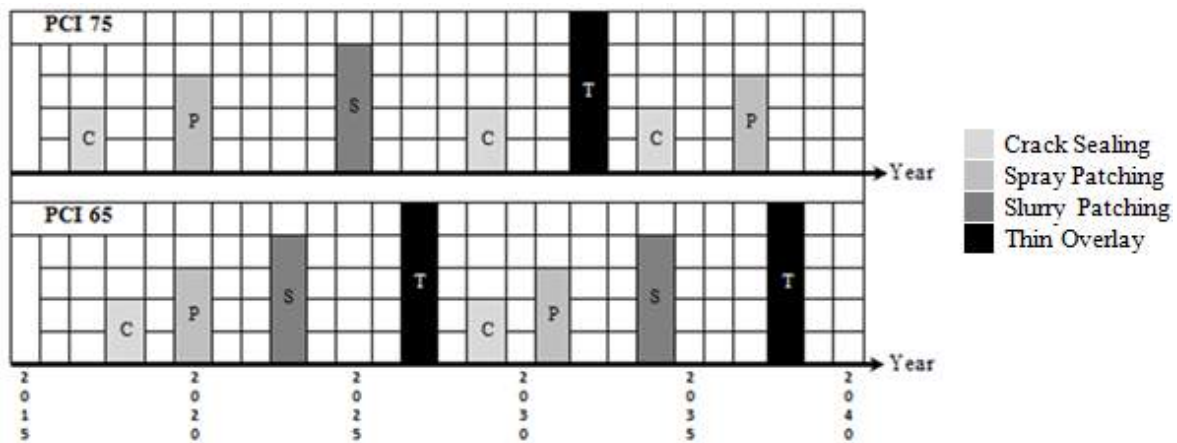


Figure 2- Timetable maintenance strategies on PCI 75 and PCI 65

As shown in Figure 2, when PCI=75 the pavement is in a satisfactory condition and therefore the maximum of life extension was considered for each activity, whereas when PCI=65 the pavement is in fair condition and, due to fatigue and severe distress, minimum life extension was selected.

3 RESULTS AND DISCUSSION

- Results for the Deterministic Method

The deterministic results are presented in Table 3 and illustrated in Figure 3. A discount rate of 4% was used in this deterministic study. Discount rate is the interest rate by which future costs (in dollars) is converted to present value. In other words, the *discount rate* is generally the distinction of interest and inflation rates, which show the real value of money over time, as shown in Equation 1. Real discount rates typically range between 3 to 5%.

$$i_{dis} = i_{int} - i_{inf} \text{ (Decimal)} \quad (1)$$

Present value (PW) is calculated using Equations 2 and 3, where “i” is the discount rate and n is the year of maintenance. NPW discounts all future costs to the present value using a discount factor:

$$PW = C \times \left[\frac{(1+i_{inf})}{(1+i_{int})} \right]^n \quad (2)$$

$$PW = \sum \text{future cost} \left(\frac{1}{(1+i_{dis})^n} \right) \quad (3)$$

Where

PW = Present-worth cost (\$);

C = Future cost in present-day terms (\$),

i_{inf} = Annual inflation rate, decimal;

i_{int} = Annual interest rate, decimal;

i_{dis} = Annual discount rate, decimal;

n = Time until cost C is incurred, years.

Table 3- NPW of Different Maintenance Scheduling Treatments

Minimum PCI	Maintenance	No. of application	Year of application	PCI before	PCI after	NPW (10000\$)
75	Crack Sealing	3	2017	80	>86	48.40
			2029	75	80	77.50
			2034	75	80	94.30
	Spray Patching	2	2020	80	>86	76.80
			2037	71-75	80	149.60
Slurry Seal	1	2025	70	>86	132.10	
Thin Overlay	1	2032	70	>86	434.90	
TOTAL COST						1013.60
65	Crack Sealing	2	2018	75	>80	50.30
			2029	70	75	77.50
	Spray Patching	2	2020	75	>80	76.80
			2031	70	75	118.20
	Slurry Seal	2	2023	71-75	80	122.10
2034			70	80	188.00	
Thin Overlay	2	2027	65	>75	357.40	
		2038	60	>75	550.20	
TOTAL COST						1540.50

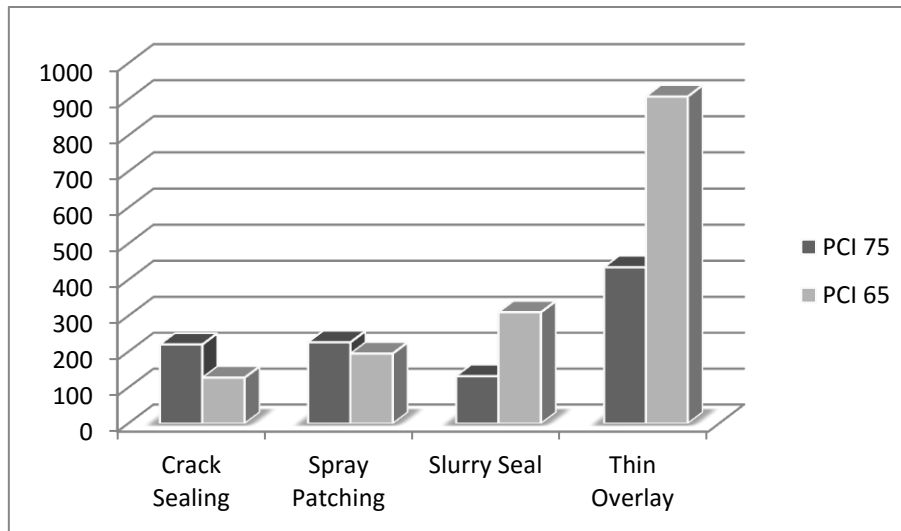


Figure 3- Deterministic results of different maintenance schedule activities

The sensitivity analysis method is used to determine the variables which influence result at the highest level. By utilizing sensitivity analysis, the variables of the model were identified and the ranking of the considered options can be rearranged by determining the breakeven points. If an adjustment in the variables of a model is the same as the discount rate, it could influence the position of the achievable design options but there will be no development of dominant alternative design options (Ferreira and Santos, 2013). In addition, the impact of a single model variable on the results of analysis can be judged through sensitivity analysis; however it is not practical for managers to accomplish a simultaneous and consolidated impact of

the several variables model on LCC outcomes and rankings. Finally, no tendency of specific values was detected as probability distribution was not assigned to the variables. As shown in Table 4, the sensitivity analysis of discount rate causes an increase in total cost. This increase is 41%, 43.2% and 44.8% for 3, 4 and 5 percent discount rate respectively.

Table 4- Discount rate sensitivity analysis results

Minimum PCI	Maintenance	NPW 3% (10000\$)	NPW 4% (10000\$)	NPW 5% (10000\$)
75	Crack Sealing	47.50	48.40	49.30
		67.70	77.50	88.60
		78.50	94.30	113.10
	Spray Patching	73.20	76.80	80.60
		121.00	149.60	184.70
Slurry Seal	119.90	132.10	145.40	
Thin Overlay	369.00	434.90	511.70	
TOTAL COST		876.80	1013.60	1173.40
65	Crack Sealing	48.90	50.30	51.80
		67.70	77.50	88.60
	Spray Patching	73.20	76.80	80.60
		101.30	118.20	137.80
	Slurry Seal	113.10	122.10	131.90
Thin Overlay	156.50	188.00	225.50	
	318.30	357.40	400.90	
440.60	550.20	685.70		
TOTAL COST		1319.60	1540.50	1802.80

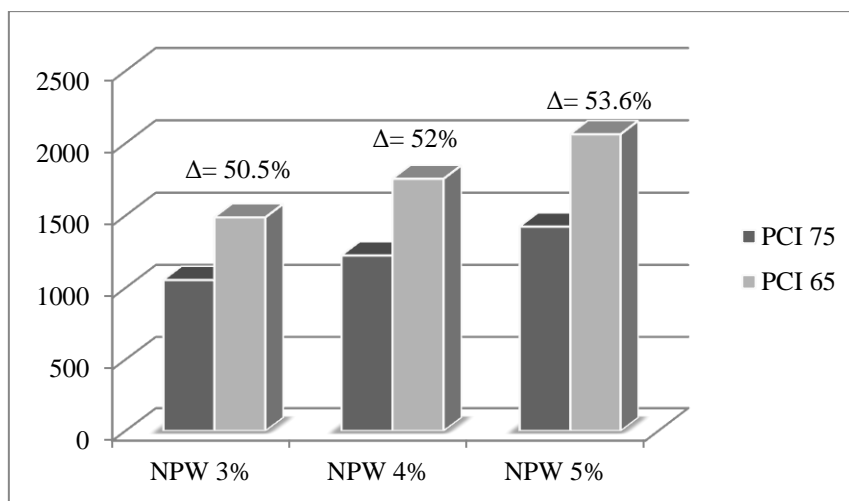


Figure 4- Differentiation of maintenance delay in various discount rate (10000\$)

- Results for the Probabilistic Method

Probabilistic LCCA was performed using L2C3A2 with 5,000 iterations. Figure 5 shows the risk assessment of NPV for two PCI alternatives at a minimum acceptable level, namely PCI 75 and PCI 65, where the probability is the area below the curve. The whole range of possible results is displayed with the evaluated probability of every result actually happening. There is no assumption that a specific option is better. The fundamental advantage of histogram is that it demonstrates variability about the mean. The wider distribution shows a larger variability. As can be seen in Figure 5, the result for Alternative II (PCI 65) is more uncertain than that of alternative I. The cumulative risk assessment for maintenance cost activities is given in Figure 6. As can be seen, there is a 40 percent probability that the maintenance cost for Alternative I (PCI 75) will be less than \$1,000,000 and a 40 percent probability that Alternative II will be less than

\$1,500,000. This means that for the 5,000 iterations processed, 40 percent of the computed values for NPV is less than \$1,000,000 when maintenance activities are carried out one year earlier (PCI 75). A steeper cumulative slope means that variability is lower while flatter slope indicates greater variability. Since the slope for Alternative II is steeper than that for Alternative I, Alternative II has a higher variability than Alternative I.

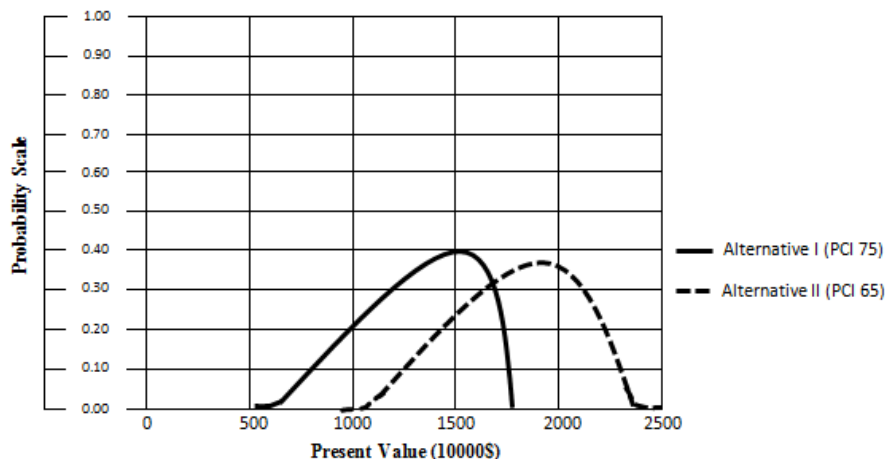


Figure 5- NPW histogram for PCI 75 and PCI 65

With regard to the consequence of making a decision based on risk assessment outcomes, decision makers need to characterize the level of risk the agency could endure. Stakeholders who could afford lower risk prefer a small spread in conceivable outcomes, with a greater probability associated with favourable results. Decision makers who are risk-takers would accept a wider spread, or acceptable variation, in the distribution of the result (Walls and Smith, 1998).

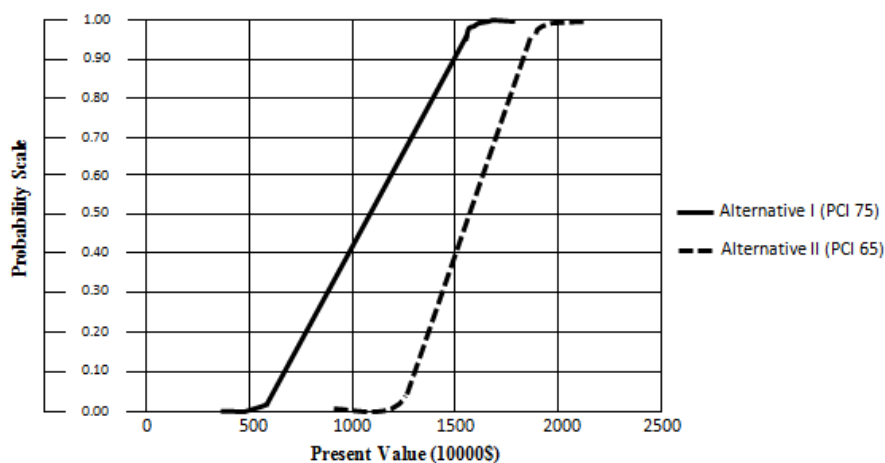


Figure 6- Cumulative risk assessment for PCI 75 and PCI 65

- Environmental Assessment

Air pollution associated with the various maintenance alternatives were calculated using L2C3A2. The input needed to run the environmental assessment at different acceptable PCI, namely NO_x , SO_x , and particulate matter (PM), is presented in Table 5. All the data in this part is borrowed from Rahman and Tarefder (2012). As shown in Table 6 the result of postponing maintenance activities for only one year (PCI 75 to PCI 65) is an increase of 67.5% in emission.

Table 5- Environmental analysis inputs for different schedule of maintenance activities

Minimum PCI	Maintenance Activity	Area Applied (m ²)	Average Thickness (mm)	Density (ton/m ³)	No. of Application	Volume of work (m ³)
75	Crack Sealing	250,000	3	1.09	3	2250
	Spray Patching	125,000	6	1.60	2	1500
	Slurry Seal	250,000	15	1.60	1	3750
	Thin Overlay	250,000	25	2.38	1	6250
65	Crack Sealing	250,000	3	1.09	2	1500
	Spray Patching	125,000	6	1.60	2	1500
	Slurry Seal	250,000	15	1.60	2	7500
	Thin Overlay	250,000	25	2.38	2	12500

Table 6- Environmental assessment of different maintenance schedule activities
(Rahman and Tarefder, 2012)

Minimum PCI	Maintenance Activity	SO _x (ton)	NO _x (ton)	PM (ton)
75	Crack Sealing	74.70	19.16	3.21
	Spray Patching	49.77	12.78	2.13
	Slurry Seal	124.47	31.95	5.34
	Thin Overlay	210.56	54.08	9.03
Overall		459.50	117.97	19.71
65	Crack Sealing	49.80	12.77	2.14
	Spray Patching	49.77	12.78	2.13
	Slurry Seal	248.95	63.91	10.68
	Thin Overlay	421.13	108.16	18.06
Overall		769.65	197.62	33.01

4 CONCLUSION

One of the most important assets of a country is its airport network, which should be managed and maintained with satisfactory schedule and condition. The advanced pavement management framework provides a precise and systematic process for keeping track of the inventory of pavement infrastructure, monitoring pavement construction, determining the right maintenance for each pavement alternative with a suitable schedule, planning and funding of airport pavement preservation exercises, and assessing the cost-effectiveness of previous pavement preservation activities.

Delaying preventive maintenance for only one year leads to an increase of approximately 52% in life cycle cost and 67.5% in life cycle assessment of airport pavements. However, it should be noted that preventive maintenance in developing countries still is being based on required to do (reactive) basis.

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ABSTRACT:

In a metropolitan region in Australia, 136 signalised intersections were identified to have been subjected to surface treatment/resurfacing over the period 2005-2010. In this study the safety effectiveness of treatment was evaluated using Naïve (simple) before-after study and Empirical Bayes (EB) approach. The results of Naïve before-after study indicated that there was a statistically significant reduction in crash rate for all crash severity levels (casualty, high severity and other injury) at 95% confidence level. For conducting EB method a reference group was used to apply Negative Binomial regression and develop safety performance (SPF) function for three different crash severity levels and predicting the expected number of crashes at treated sites. The results showed that there was a reduction in total casualty crashes by 21.3%, high severity crashes (fatality and serious injury) by 15.3% and other injury crashes by 21.4%.

Safety effectiveness of surface treatment: A comparison of Empirical Bayes and Naïve before and after study

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1 INTRODUCTION

To evaluate the change in safety performance on a roadway facility after implementation of a safety treatment several methods can be used. The most commonly used method is before and after study (Hauer, 1997, Elvik, 2002). Harwood et al. (2002) stated that there are three common approaches for evaluating safety effectiveness: Naïve or simple before-after study, before-after study with comparison group and before-after study with Empirical Bayes (EB) method. The primary goal for all these methods is to compare observed crash data for before period with crash data in the after period for same roadway facility (Hauer and Persaud, 1983). The simplest technique for conducting such comparison is Naïve or simple before-after study, where crash rate in the before period is compared with crash rate in the after period (Hauer, 1997). However, this method does not account for regression to the mean (RTM) bias and traffic volume change. RTM which is the most serious problem is the tendency of sites with an unusually high or low crash counts to regress to the mean frequency of crashes in the year following (Gross et al. 2010). This phenomenon arises when sites are selected based on their historical crash records (Hauer, 1997, Shen and Gan, 2003, Gross et al. 2010). The EB method for before and after study has been developed to control for RTM problem (Hauer, 1997, Hauer et al. 2002). The idea behind this method is to evaluate the effect of a treatment i.e. the change in safety performance at a treated site. This can be done by comparing the expected number of crashes for the after period had the treatment not been implemented with the reported number of crashes during the after period. This is based on using reference sites with similar traffic and other site characteristics (Hauer, 1997, Persaud and Lyon, 2007, Gross et al. 2010).

A recent study by Sacchi and Sayed (2015) has been conducted to evaluate before and after safety effectiveness using common Bayesian approaches and to compare the accuracy of these methods. The authors found that all approaches including Naïve before and after study resulted in small statistically non-significant change in crashes in the case of random selection of sites, with the Fully Bayesian (FB) method providing more precise estimates. However, when sites were selected according to high crash occurrence, the safety effectiveness index can be biased by 10%, which can have effect on the evaluation (Sacchi and Sayed, 2015).

The objective of this study was to evaluate safety effectiveness of the improvements in pavement surface condition of signalised intersections. The evaluation was conducted using both Naïve and simple before-after study and EB approaches to estimate the reductions in total casualty crashes, high severity (fatality and serious injury) crashes and other injury crashes. The targeted facility type was 136 treated intersections and their approaches (the immediate 200 m approach) along which the crashes occurred. These intersections were subjected to surface treatment, which involved resurfacing with asphalt. The study objective was accomplished using traffic volume data and crash data of treated sites and reference sites during the period 2000-2013.

2 LITERATURE REVIEW

Several studies have been conducted to evaluate safety effectiveness of pavement surface treatment by using different methodologies. However, Hauer et al. (2002) suggested that for addressing problems of safety estimation, EB method provides a more precise estimate of the number of crashes and corrects for the RTM problem. Provided below is a review of a number of before and after studies for evaluating the effectiveness of pavement surface treatments in improving safety performance.

Giles and Sabey (1959) evaluated safety improvement of skid resistance before and after resurfacing in the United Kingdom. The authors utilised data for 128 resurfaced sites with skid related crashes and 100 control sites of similar traffic volumes and site characteristics. The crashes were classified into weather related crashes (dry/wet) and skidding related crashes. The authors reported that resurfacing demonstrated a reduction in crashes after applying a surface treatment which confirms an improvement in skid resistance. They found that the average number of total crashes that occurred in wet pavement conditions decreased from eight to two and the average number of crashes related to skidding decreased from six to two (Giles and Sabey 1959). In another before and after study, reported by Kinnear et al. (1984), the authors assessed safety improvement associated with three high skid resistance treatments at 39 intersections in Sydney, Australia. Their study included assessing the cost, wet sideways frictional coefficient and crash occurrence at these sites in comparison with 12 control sites i.e. untreated. Analysis results of crash data three years before and 18 months after treatment indicated that there was an efficient reduction in crashes of 25% at 99% confidence level for all surfaces after treatment. The authors suggested that providing an adequate level of skid resistance for surfaces at intersections is associated with a very high benefit cost ratio (Kinnear et al. 1984).

Cleveland (1987) examined safety effect of improving pavement roughness and skid resistance after resurfacing. The study revealed that resurfacing resulted in a small increase in total crash frequency in rural areas and a reduction in wet pavement crashes of about 20%. The author related the negative effect of resurfacing on safety to increased vehicle speed on smooth surfaces after resurfacing. Hauer et al. (1994) applied EB approach to evaluate the effect of resurfacing on the safety of two-lane rural roads in New York State. The results indicated that non intersection crashes increased by 21% for projects involving resurfacing only. However, safety was improved for projects where resurfacing was accompanied with additional surface improvements. Lyon and Persaud (2008) conducted a study to evaluate safety effect of improving pavement skid resistance in New York, USA. The authors used data from intersections (256 treated intersections) and roadway segments (118 treated segments) that had high proportions of wet-road crashes and low friction numbers. With an equivalent number of reference sites, they evaluated safety improvement using EB method. The results indicated that the improvements in skid resistance produced considerable safety benefits for both intersections and road segments.

Abdel-Aty et al. (2009) studied the safety effect of 136 resurfacing projects on multilane arterials in Florida using EB method. With the application of Negative Binomial (NB) regression, 27 Safety Performance Functions (SPFs) were developed for three different crash groups (total, rear-end and severe) and for three land uses (urban, suburban and rural) using different length groups of 0.5-1.5, 1.5-3 and > 3 miles. For all developed SPFs, speed limit was found to be not significant for severe crashes except for urban sections less than 3 miles. The results of EB method indicated that total number of crashes increased by 62% at the treated sites. However, the rear-end crashes reduced by 0.83% and severe crashes reduced by 4.63%. The authors also reported that although there was a slight reduction in wet-pavement related crashes from 13.88% in the before period to 11.14% in the after period, the reduction could not be directly related to resurfacing and that it could be related to lower rain fall in the after period.

In a before and after study with a comparison group conducted by Pardillo Mayora and Jurado Pina (2009) on 419 resurfaced pavement segments there was a significant reduction (average of 68%) in wet-pavement crash rate. Zeng et al. (2014) applied EB approach to evaluate the effect of pavement condition improvements on the safety of rural two lane undivided highways in Virginia. The results indicated that the treatment was significant in reducing fatal and injury crashes by 26%. However, it was not effective in reducing total crash frequency. According to government reports by Candappa et al. (2007), the application of different types of intersection treatments such as line marking, geometry, signage, and resurfacing to reduce the number of crashes and fatalities on Victorian roads (Australia) has been extremely effective over the years. Many of the applied surface treatments were related to improving surface skid resistance and texture loss.

3 SITE SELECTION

Two groups of sites were selected for this study namely; treated sites and reference sites. All these sites consist of granular pavements (crushed rock bases and sub bases) with thin asphalt surfacing or double coats of sprayed seal.

- Treated sites: The evaluation was done through a before and after study where only intersections that were subjected to surface treatment over the period 2005-2010 were considered. This period was selected as it was the period over which relevant condition data were available for reasonable time periods before and after treatment years. This study could be considered fully controlled as the same intersections were used before and after treatment hence, the only variation was related to surface condition and all other road related factors that may affect crash occurrence remained unchanged. It is important to note that the applied treatments do not cover the whole intersection. They may cover the intersection centre, intersection centre and approaches or any of its approaches. The lengths of treated sections used in the analysis range between 100 m and 500 m. A large number of signalised intersections that were treated (with a thin layer of asphalt) over the nominated period was identified from maintenance records. Out of these sites, only those that had crash data for at least 5 years before and a minimum of 3 years after treatment years (NZTA, 2012) were included. The final set of treated sites had a sample size of 136.
- Reference Sites: A group of 66 reference sites were selected to match approximately with their pairs of treated sites in terms of traffic volumes and site characteristics but were not treated during the study period.

4 DATA PREPARATION

The data collected for studying safety performance of the identified signalised intersections include the following:

- Crash data at the treated approach of each site for 5 years before and 3 to 5 years after treatment year were considered for the analysis. The collected data included all casualty crashes i.e. covering all severity levels (fatal, serious injuries and other injuries), type of crash (head on, rear etc.), light condition when they occurred (day or night), road surface moisture condition (wet or dry) and speed limit. Crash data of treatment year was not included. Crash data was collected from VicRoads (2014) which contains information on crashes that occurred during the 13-year period of interest (2000-2013) as provided by the Victorian police. This data was filtered to select only crashes that occurred along the treated section which could include the centre of intersection only, the intersection centre and its approaches or the approaches only i.e. immediate 200 m. In other words, those that occurred within a 500 m long section as shown in Figure 1. This was achieved by using the chainages where crashes occurred. Locating crashes to their exact sites helps to achieve more precise results in crash analysis. This would also help in relating approach variables to crash injury so their effects can be investigated in the analysis (Wang and Abdel-Aty, 2008). Crash data for all intersections was available for 5 years before treatment years but for after treatment years, data was available for 3, 4 or 5 years. Crash frequency has been used for safety evaluation using EB method. However, evaluating safety effectiveness of surface treatment using simple (Naïve) before and after study is based on crash rate analysis.
- Traffic volume data for the years before and after treatment covering the same number of years of available crash data. Traffic volume data used in the analysis was in terms of Annual Average Daily Traffic (AADT) that used the section of the road where the intersection site was located i.e. not peak traffic at the intersection. Traffic volume data for the relevant years before and after treatment was used for developing Safety Performance Functions (SPFs) and estimating expected number of crashes at the treated sites.
- The evaluation period of 5 years before and (3-5) years after treatment year was selected as a multiple of 12 months in length to control for seasonal bias in the evaluation (Highway Safety Manual, 2010).

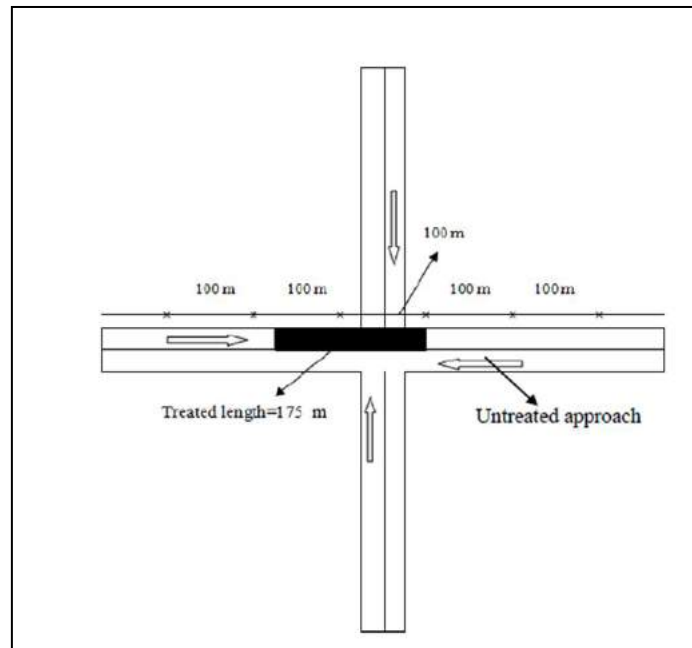


Figure 1. Sketch of an intersection and approaches of immediate of 200 m illustrating the treated length

5 METHODOLOGY

In this study both Naïve or simple before-after study and EB approaches have been used to evaluate safety performance at 136 intersections (or the immediate 200 m approach) that were subjected to surface treatment. Both approaches were used to evaluate treatment effectiveness in terms of reducing or changing crash rate and frequency. Accordingly, three sets of crash data were considered in the analysis and they include casualty crashes, high severity crashes (fatality and serious injury) and other injury crashes.

To compare the rate of a measurement under different conditions with the same sample and identify the effect of treatment using simple before and after study, a paired t-test is suggested by Austroads (2009). However, when the assumptions of normality are not met for paired groups, a nonparametric test is a powerful alternative test to paired t-test. In Wilcoxon test, intersection crash rate value under the before treatment condition is paired with the same intersection crash rate value under the after treatment condition. The differences between crash rate for the before treatment period and after treatment period are computed and tested using Wilcoxon signed-rank test. The test ranks these differences and assigns a negative sign when the crash rate before treatment is less than crash rate after treatment and a positive sign when the crash rate before treatment is higher than the crash rate after treatment. If the positive ranks are much higher than negative ranks, then crash rates are higher before treatment than after (Gibbons and Chakraborti, 2011).

EB before-after study has been conducted for safety evaluation to account for regression to the mean bias and time trend. Annual crash data and Annual Average Daily Traffic (AADT) of the reference sites were used to develop Safety Performance Functions (SPFs) to account for changes in traffic volume. SPFs were developed for different crash severity levels including total casualty crashes, high severity crashes and other injury crashes separately using Generalised Linear Model (GLM) (Gross et al. 2010). One of the characteristics of crash frequency data is that the variance of crash counts is greater than the mean (over dispersion). Milton and Mannering (1998) suggested that the negative binomial regression is an appropriate predictive model for applying in crash frequency studies to overcome the over-dispersion problem. Several criteria were used to choose the appropriate probability model for crash data analysis. These criteria are examining whether the dispersion statistic is greater than one and testing whether the dispersion parameter (α) > 0 using the Lagrange multiplier test. The values of the dispersion statistic (Pearson chi square/degree of freedom) for the Poisson model with no covariates (intercept only) of 1.77 for casualty crashes, 1.27 for high

severity crashes and 1.6 for other injury crashes are greater than 1 which suggest that the dependent variable (crash frequency) is over dispersed. In addition, the dispersion parameter (alpha, α) refers to the parameter used in NB2 model to take account of over-dispersion which is assumed to be zero in the Poisson model. However, the negative binomial model allows it to be greater than zero. The dispersion parameters for NB2 model with no covariates (intercept only) of 0.41 for casualty crashes, 0.434 for high severity crashes and 0.471 for other injury crashes are greater than zero which again indicate that the dependent variable (crash frequency) is over dispersed. Results of the Lagrange multiplier test for crash data illustrate that the Z-score test is 4.75 for casualty crashes, 2.22 for high severity crashes and 4.36 for other injury crashes with a t-probability of $P < 0.001$ for casualty and other injury crashes and t-probability of $P = 0.026$ for high severity crashes. A significant Lagrange test for all crash severity levels indicates that the model dispersion parameter is different from zero. These results prove that the hypothesis of no over dispersion is rejected and the real over dispersion exists in all data sets. Therefore, casualty, high severity and other injury crash data should be modeled as negative binomial which is preferred over Poisson model.

Model coefficients were estimated assuming negative binomial error distribution. SPFs are mathematical models that predict crash frequency for each year at sites with similar traffic volumes and other site characteristics to treatment sites. This SPF is then used to derive the number of crashes predicted at treated sites. Traffic volume is considered the most important factor in crash occurrence (Abdel-Aty and Radwan, 2000 and Chin and Quddus, 2003) and is used as an explanatory variable in crash modification factor (Lord and Persaud, 2000 and Lord and Persaud, 2004). Accordingly, the SPF for this study is based on total traffic volume entering the nominated approach (where crashes occurred) of each intersection per year in a reference group. By using relevant information (crash frequency and traffic volume) from both treated and reference sites, the expected number of crashes at treated sites can be estimated. The detailed procedure of EB method described in Highway Safety Manual (2010) has been followed to evaluate safety effectiveness of pavement surface treatment expressed in percentage change in crashes. The steps followed are summarised in the Appendix.

6 EVALUATION RESULTS AND DISCUSSION

The evaluation results of treatment effect on safety improvement at signalised intersections for both Naïve and EB before and after study are presented in the following sections.

6.1 Naïve or simple before-after study results

The Wilcoxon signed-rank test is used to evaluate statistical significance of the changes in crash rates for all the three crashes categories namely total casualty, high severity and other injury crashes. Descriptive statistics of crash rate for all crash severity levels before and after treatment periods are shown in Table 1. The test results are summarised below for each crash category.

Casualty crashes: Table 2 of Wilcoxon ranks shows that in 89 out of 136 cases, positive ranks are much higher than negative, which indicates that the rates for crashes were higher before treatment than after (Gibbons and Chakraborti, 2011). Also, the table provides the exact two-tailed p-values for the statistic W (Wilcoxon). Clearly the test shows that the difference between casualty crash rate before treatment (Mean= 5.34, SD= 5.19) and crash rate after treatment (Mean= 3.77, SD= 3.68) is statistically significant ($p=0.000$).

High severity crashes: Table 2 also shows that for high severity crashes, in 68 out of 136 cases positive ranks are again higher than negative. This indicates that the rates are higher before treatment than after. Wilcoxon test results show that the difference between crash rate before treatment (Mean= 1.64, SD= 2.18) and crash rate after treatment (Mean= 1.27, SD= 1.59) is statistically significant ($p=0.026$).

Other injury crashes: the same test was applied to other injury crashes only and showed that in 88 out of 136 cases, positive ranks have greater rank than negative, which again indicates that the rates were higher before treatment than after. The results of Wilcoxon test show that the difference between crash rate before treatment (Mean= 3.84, SD= 4.17) and crash rate after treatment (Mean= 2.47, SD= 2.51) is statistically significant ($p=0.000$).

Table 1. Descriptive statistics for crash rates in before and after treatment period

Crash Severity	N	Mean	Min	Max	SD
Casualty crash rate before	136	5.3379	0.00	32.48	5.19277
Casualty crash rate after	136	3.7699	0.00	22.83	3.67628
High severity (F&SI) crash rate before	136	1.6391	0.00	17.41	2.18239
High severity (F&SI) crash rate after	136	1.2738	0.00	11.42	1.59181
Other injury crash rate before	136	3.8371	0.00	28.54	4.17299
Other injury crash rate after	136	2.4724	0.00	11.42	2.51007

Table 2. Positive and negative ranks

Crash Severity		N	Mean Rank	Sum of Ranks	Wilcoxon test, Sig. (2-tailed)
Casualty crash rate before Casualty crash rate after	Negative ranks	30 ^a	45.25	1357.50	.000
	Positive ranks	89 ^b	64.97	5782.50	
	Ties	17 ^c			
	Total	136			
High severity (F&SI) crash rate before High severity (F&SI) crash rate after	Negative ranks	43 ^a	54.72	2353.00	.026
	Positive ranks	68 ^b	56.81	3863.00	
	Ties	25 ^c			
	Total	136			
Other injury crash rate before Other injury crash rate after	Negative ranks	29 ^a	42.64	1236.50	.000
	Positive ranks	88 ^b	64.39	5666.50	
	Ties	19 ^c			
	Total	136			

a. Crash rate before treatment < Crash rate after treatment

b. Crash rate before treatment > Crash rate after treatment

c. Crash rate before treatment = Crash rate after treatment

6.2 Empirical Bayes before-after study

Summary statistics for treated sites (136 sites) and reference sites (66 sites) used in this study are provided in Table 3. The table includes descriptions for crashes in the three categories namely total casualty, high severity and other injury crashes for both treated and reference sites. It is important to note that the mean value of AADT entering each intersection approach of treated sites is approximately similar to that of the reference sites.

6.3 Safety performance function development

Data from reference sites were utilized to develop the SPF for different crash severity levels including total casualty crashes, high severity crashes (fatality and serious injury) and other injury crashes separately using Generalised Linear Model (GLM). Model coefficients were estimated assuming negative binomial error distribution. The SPF for this study is based on total traffic volume entering the nominated approach (where crashes occurred) of each intersection per year in a reference group. By using relevant

information (crash frequency and traffic volume) from both treated and reference sites, the expected number of crashes at treated sites can be estimated.

Table 3. Summary statistics for treated and reference sites

	Crash Frequency	Mean	Min	Max	SD
Treated sites	Casualty before	9.96	0.00	44.00	8.63
	Casualty after	7.43	0.00	32.00	6.90
	High Severity before	2.90	0.00	12.00	2.99
	High Severity after	2.51	0.00	18.00	2.86
	Other Injury before	7.05	0.00	36.00	6.76
	Other Injury after	4.90	0.00	22.00	4.81
	AADT _{entering} before	12864.90	1600	27136	5537.16
	AADT _{entering} after	13485.33	1600	28250	5686.13
Reference Sites	Casualty before	9.02	0.00	31.00	6.37
	Casualty after	8.21	0.00	28.00	6.25
	High Severity before	2.64	0.00	11.00	2.26
	High Severity after	2.56	0.00	10.00	2.25
	Other Injury before	6.38	0.00	21.00	4.81
	Other Injury after	5.65	0.00	22.00	4.69
	AADT _{entering} before	13395.3	3420	29135	6038.62
	AADT _{entering} after	13864.8	4495	30250	6175.75

The primary model form for all SPFs used in this study is:

$$N_{spf} = \text{Exp}(\alpha + \beta * \ln(\text{AADT entering})) \quad (1)$$

where:

N_{spf} = Estimated crash frequency per year

α, β = Regression coefficients

AADT = Annual Average Daily Traffic in veh/day

Results of NB regression for developing SPF for all three crash severity levels are provided in Table 4. The results indicate that the parameters are significant at 95% confidence level. Overall the results indicate that NB model fits the data well and is a suitable predictive model for applying in SPF development. The dispersion parameter of 0.374, 0.42 and 0.419 for total casualty, high severity and other injury crashes, respectively are greater than zero which indicate that the dependent variable (crash frequency) is over dispersed in all three models. Furthermore, the value of deviance of 1.149, 0.996 and 1.125 and Pearson chi square of 0.951, 0.995 and 0.963 for total casualty, high severity and other injury crashes, respectively obtained from the goodness-of-fit criteria are close to the ideal value of 1 which supports the use of NB model.

Necessary yearly factors as SPFs multiplier are applied to Equation (1) to account for time trend in EB approach as shown in Equation (2).

$$N_{predicted} = N_{spf} * \text{Yearly Factor} \quad (2)$$

where:

$N_{predicted}$ = Predicted crash frequency per year for each site

Yearly Factor = Yearly correction factors to account for time trend

Table 4. Results of SPF model development for all crash severity levels

Crash Severity	Total Casualty	High Severity (F & SI)	Other Injury
Parameters	Estimated value (standard error)	Estimated value (standard error)	Estimated value (standard error)
Intercept	0.303 (0.128)*	-0.748 (0.196)*	-0.123 (0.148)*
AADT _{entering}	0.000027 (0.000008)*	0.000016 (0.000013)*	0.000033 (0.00001)*
Dispersion Parameter	0.374 (0.076)	0.420(0.186)	0.419 (0.101)
Deviance	1.149	0.966	1.125
Pearson chi square	0.951	0.995	0.963

* Significant at 95% confidence level

These SPFs multiplier are derived for different crash severity levels for each year before and after treatment. They are estimated as the total number of observed crashes divided by total number of predicted crashes for a given year (Highway Safety Manual, 2010) as shown in Tables 5.

Table 5. Applied yearly factors for all crash severity levels

Casualty crashes										
Treatment	Years Before treatment					Years After treatment				
year	1st Year	2nd Year	3rd Year	4th Year	5th Year	1st Year	2nd Year	3rd Year	4th Year	5th Year
2005*	2.83	1.84	2.09	1.48	2.09	2.46	1.84	1.97	1.23	1.23
2006	0.95	1.05	0.63	0.95	1.23	1.69	2.11	0.86	1.48	0.98
2007	1.52	1.69	1.22	1.09	1.30	1.82	0.96	1.00	1.00	1.17
2008	0.68	0.91	1.59	1.59	1.25	1.36	0.91	0.97	0.80	1.14
2009	2.17	1.85	1.39	2.00	1.60	2.21	1.89	1.52	0.62	
2010	0.6	1.92	1.18	1.03	1.48	1.43	1.43	1.43		

High severity crashes (F & SI)										
Treatment	Years Before treatment					Years After treatment				
year	1st Year	2nd Year	3rd Year	4th Year	5th Year	1st Year	2nd Year	3rd Year	4th Year	5th Year
2005	2.46	2.46	0.70	1.06	1.76	2.58	2.58	2.58	1.06	1.06
2006	1.21	1.21	0.60	0.60	0.60	1.50	2.11	0.91	0.91	0.60
2007	1.86	1.12	0.87	0.99	1.12	1.24	0.99	1.73	0.74	0.99
2008	0.89	0.89	1.30	0.81	1.14	0.65	0.65	0.64	0.65	0.65
2009	1.64	1.76	1.29	0.82	1.06	1.76	1.76	0.94	0.94	
2010	1.48	1.48	2.11	2.53	1.27	2.10	0.63	0.63		

Other injury crashes										
Treatment	Years Before treatment					Years After treatment				
year	1st Year	2nd Year	3rd Year	4th Year	5th Year	1st Year	2nd Year	3rd Year	4th Year	5th Year
2005	2.64	1.88	2.64	2.07	2.07	1.88	1.51	2.07	1.32	1.32
2006	0.81	0.81	0.97	0.97	1.62	1.62	2.10	0.97	0.97	0.97
2007	1.4	1.99	1.40	1.13	1.40	1.99	0.84	0.73	1.06	1.40
2008	0.61	1.04	1.65	2.00	1.30	1.57	1.22	1.13	0.70	1.57
2009	2.13	2.01	1.44	2.64	1.88	2.46	1.93	1.66	0.80	
2010	0.9	1.02	1.01	0.90	0.91	2.04	2.03	0.90		

*Example: For a treatment that is applied in 2005, the years before treatment are: 2004 (1st year), 2003 (2nd year), 2002 (3rd year), etc. For after treatment years, they are: 2006 (1st year), 2007 (2nd year) and so on.

To account for the difference in duration between before (5 years) and after (3-5 years) periods, the adjustment factor r_i is recommended by Highway Safety Manual (2010) and is calculated as given by Equation 5 in the Appendix. This factor is the sum of the annual SPF predictions for the after period divided by the sum of these predictions for the before period. The result, after using this factor, is the expected number of crashes for the after period had the treatment not been implemented. The overall results obtained from EB approach to evaluate safety effectiveness of pavement condition improvement following treatment are given in Table 6.

Table 6. Results of safety effect evaluation by EB before-after study

Parameters	EB Results		
	Total Casualty	High Severity (F & SI)	Other Injury
Number of intersections	136	136	136
Total number of crashes observed in the after period (Nobserved,A)	1010	341	667
Total number of crashes expected in the after period had the treatment not been implemented. (Nexpected,A)	1282.94	402.21	847.82
Treatment effectiveness in the form of odd ratio for all sites, OR'	0.787	0.848	0.787
Var (Nexpected, A)	983.34	239.57	559.25
Unbiased estimate of treatment effectiveness, OR (Crash Modification Factor, CMF)	0.787	0.847	0.786
Safety Effectiveness = $100 \times (1 - OR)$	21.3%	15.3%	21.4%
Var (OR)	0.00098	0.00316	0.0014
SE (OR) = $\sqrt{\text{Var}(OR)}$	0.0313	0.056	0.037
95% Confidence Interval= $OR \pm 1.96 \times SE(OR)$	0.72~0.85*	0.74~0.96*	0.71~0.86*
SE (Safety Effectiveness)= $100 \times SE(OR)$	3.13	5.56	3.75
Abs[Safety Effectiveness/SE(Safety Effectiveness)]	6.8*>2.0	2.75* > 2.0	5.71*>2.0

* Significant at 95% confidence level

For total casualty crashes evaluated in this study, the value of OR (Highway Safety Manual, 2010) which is also referred to as Crash Modification Factor (CMF) is 0.787 with a standard error of 0.0313. For high severity crashes, the CMF is 0.847 with a standard error of 0.056 and for other injury crashes, CMF is 0.786 with a standard error of 0.037. The CMF is significant (less than 1) at 95% confidence level since the 95% confidence interval of (0.72~0.85) for total casualty and (0.74~0.96) for high severity (fatality and serious injury) crashes and (0.71~0.86) for other injury crashes does not include 1. This means that it is at least 95% certain there is a reduction in crashes after pavement surface treatment (Gross et al. 2010). The absolute value of the measure, Abs [Safety Effectiveness/SE (Safety Effectiveness)] for all total casualty, high severity and other injury crash categories is greater than 2.0 which confirms that the treatment effect is significant at the (approximate) 95% confidence level (Highway Safety Manual, 2010). These evaluation

results indicate that there is a reduction in total casualty crashes by 21.3% with a standard error of 3.13%, and 15.3% with a standard error of 5.56% for high severity crashes. The evaluation results also showed that other injury crashes were reduced by 21.4% with a standard error of 3.75%.

7 SUMMARY OF FINDINGS AND CONCLUSION

This paper evaluates the safety effectiveness of pavement surface treatment for 136 signalised intersections. The authors acknowledge that many studies have been performed on the relationship of surface condition and safety and the beneficial effects of resurfacing. The reported studies used different approaches in data collection, different condition parameters and adopted different statistical techniques for analysis. This is in addition to using appropriate distribution function (NB), following a number of relevant tests. In this study, a different approach is used in that using existing data collected for other purposes i.e. it is an observational study as it did not include collecting field data for a fully controlled study. It is acknowledged that the changes in driver behaviour and vehicle characteristics over time are impossible to control. However, during the study period for each site used herein, the variations in these factors are expected to be limited. Additionally, a systematic approach has been adopted in selecting the sites to ensure that limited variations in possible contributing factors have occurred. Most importantly, the geometry of intersections, speed limits and signal phasing and timing stayed constant. This means that some of the results could be affected by the uncontrolled factors. Since it is not a fully controlled study, the results can be considered indicative and not exact. Results indicated that for the types of crashes assessed/considered herein, resurfacing has a positive effect on reducing them. The results are discussed in detail in section 6. The findings from the safety evaluation of resurfacing described in this paper can be summarised as follows:

- 1- Results of Naïve or simple before-after study using nonparametric Wilcoxon Signed Rank test indicated that the difference between crash rate before and after treatment for all crash severity levels (casualty, high severity and other injury) were statistically significant at 95% confidence level. However, it is not obvious what is the percentage of reduction due to the treatment and what percentage is due to the contribution of other factors that changed during the time.
- 2- The Empirical Bayes (EB) is a state of the art evaluation method in before and after study to find the crash modification factor and account for regression to the mean bias and traffic volume change through developing SPFs. Results of EB approach revealed that the treatment effect in improving safety at the selected sites is significant at the (approximate) 95% confidence level in reducing total casualty crashes by 21.3%, high severity crashes (fatality and serious injury) by 15.3% and other injury crashes by 21.4%.

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APPENDIX

Empirical Bayes (EB) before and after safety effectiveness evaluation steps:

1. Select a group of reference sites with approximately similar traffic and site characteristics to treated sites which have not been treated during the study period.
2. Develop Safety Performance Function (SPF) using Negative Binomial (NB) regression based on data from reference sites.
3. Estimate the predicted number of crashes for treated sites based on SPF for each year in the before period.

$$N_{\text{predicted}, B} = N_{\text{spf}} * \text{Yearly Factor} \quad (1)$$

$$N_{\text{spf}} = \text{Exp}(\alpha + \beta * \ln(\text{AADT entering})) \quad (2)$$

Where: N_{spf} = Predicted number of crashes estimated by SPF.

AADT= Annual Average Daily Traffic Entering Intersection

Yearly Factor = Yearly correction factors to account for time trend

4. Estimate the expected number of crashes for the entire before period for treated sites.

$$N_{expected, B} = W * N_{predicted, B} + (1 - W) * N_{observed, B} \quad (3)$$

where

W = the weighted adjustment for each site for the before period

$$W = \frac{1}{1 + k \sum N_{predicted, B}} \quad (4)$$

and

$N_{expected, B}$ = Expected number of crashes for treated sites for before period

k= Over dispersion parameter from NB regression for the SPF model

5- Estimate the predicted number of crashes for treated sites in the after period based on SPF developed.

6- Calculate the adjusted factor r_i to account for the difference in duration between before and after periods at each site.

$$r_i = \frac{\sum_{Year} N_{predicted, A}}{\sum_{Year} N_{predicted, B}} \quad (5)$$

7. Estimate the expected number of crashes for the after period had the treatment not been implemented.

$$N_{expected, A} = N_{expected, B} * r_i \quad (6)$$

8. Estimate the treatment effectiveness for each site

$$Safety\ Effectiveness = 100 (1 - OR_i) \quad (7)$$

$$OR_i = \frac{N_{observed, A}}{N_{expected, A}} \quad (8)$$

9. Estimate the treatment effectiveness in the form of odd ratio for all sites, OR'

$$OR' = \frac{\sum_{All\ sites} N_{observed, A}}{\sum_{All\ sites} N_{expected, A}} \quad (9)$$

10. Calculate the adjustment for an unbiased estimate of treatment effectiveness, OR

$$OR = \frac{OR'}{1 + \frac{Var(\sum_{All\ sites} N_{expected, A})}{(\sum_{All\ sites} N_{expected, A})^2}} \quad (10)$$

11. Estimate overall unbiased safety effectiveness

$$Safety\ Effectiveness = 100 (1 - OR) \quad (11)$$

12. Calculate the variance and standard error of odd ratio, $Var(OR)$ and $SE(OR)$

$$Var(OR) = \frac{(OR)^2 \left[\frac{1}{N_{observed, A}} + \frac{Var(\sum_{All\ sites} N_{expected, A})}{(\sum_{All\ sites} N_{expected, A})^2} \right]}{\left[1 + \frac{Var(\sum_{All\ sites} N_{expected, A})}{(\sum_{All\ sites} N_{expected, A})^2} \right]} \quad (12)$$

$$SE(OR) = \sqrt{Var(OR)} \quad (13)$$

13. Estimate the standard error of safety effectiveness

$$SE(Safety\ Effectiveness) = 100 * SE(OR) \quad (14)$$

14. Evaluate the statistical Significance of the estimated safety effectiveness by determining whether the Abs [Safety Effectiveness / SE (Safety Effectiveness)] is greater than 2.0. If this measure is greater than 2.0 it means that the treatment effect is significant at 95% confidence interval.

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PAPER TITLE	Active Traffic Management with Congestion mitigation Tools in Japan		
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KEYWORDS:

Active traffic management, Traffic congestion, Congestion mitigation tool, VMS, Traffic squad

ABSTRACT:

NEXCO-Central (Japan) has introduced “Active Traffic Management” (ATM) with the leading-edge technology that is well structured to actively manage traffic flow and then carry out all the process automatically, reduce congestion, and secure safety. Through our traffic control centers, on-site information such as traffic accidents, congestions, weather conditions, road defects, fallen objects, and lane closures are collected real time and 24/7. Then, this information are appropriately processed and effectively delivered using wide range of mediums.

As one of ATM, we have effectively developed congestion mitigation tools for increasing traffic capacity and distributing traffic demand. Regarding tools for increasing in traffic capacity, we have introduced “variable message signs” (VMSs) for alerting reduced speed as well as network expansion, ETC installment, and road widening. The result showed that approximately 5% of traffic congestion was reduced in a certain section after appropriate deployment of mobile VMS with their display indicating cautions. Regarding distribution of traffic demand, proactive congestion prediction and real time information provision have been introduced to encourage drivers to change their choices of travel time and route. The result brought by proactive congestion prediction applied during a holiday season, with which congestion prediction is provided on a time-zone basis so that drivers can easily learn the rough travel time to their destinations, showed that approximately 5% of traffic congestion was reduced in a certain section, using the road pricing, the giveaway campaign, and the issue of guidebook notifying a peak period. As a real time information provision which is a reactive approach, various VMSs have been prepared between Interchanges, at every exit, toll gate entrance and general road connecting to an expressway. Also graphic travel time signs are available when drivers have choices of the route. Furthermore, in order to collect accurate road information promptly, we own and operate well-trained traffic squads for expressway patrol by ourselves, who play an essential part in collecting information and securing smooth traffic.

In this paper, our “Active Traffic Management” (ATM) is described focusing on our traffic control center, congestion mitigation tools applied and traffic squads.

Active Traffic Management with Congestion Mitigation Tools in Japan

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1. Overview

NEXCO-Central operates approximately 2,000km of expressways with Japan's heaviest traffic volume roadway connecting three major Japanese metropolitan regions that together produce more than 50 percent of Japan's gross domestic product (GDP). In order to operate these expressways safely and reliably with high-quality service and ensure the vitality of Japan's economy, leading-edge construction, operations and maintenance technologies are essential under severe regional conditions of central Japan such as the diverse climate and geography, dense population and high traffic volume. Due to these unique regional conditions, continuous efforts have been made in traffic management field not only to offer safety to drivers but also to alleviate traffic congestion.

In this paper, one of our efforts, which is "Active Traffic Management" (ATM) is described focusing on our traffic control center, congestion mitigation tools applied and traffic squads.

2. ATM's purpose

There have been three primary purposes in introducing ATM. 1st is providing accurate information to "every single driver", 2nd is encouraging drivers to use expressways wisely by offering choice of alternative routes, and 3rd is leading positive social effects such as declining accidents, alleviating congestion, reducing CO₂ emission, and improving customer satisfaction. To reach these objectives, we have made use of ATM with the leading-edge technology, that is well structured to actively manage traffic flow and then carry out all the process automatically, reduce congestion, and secure safety. Through our traffic control centers, on-site firsthand information such as traffic accidents, congestions, weather conditions, road defects, fallen objects, and lane closures are collected real time and 24/7. Then, information is appropriately processed and effectively delivered through wide range of mediums such as variable message signs (VMSs), mobile VMSs, advisory radio. Furthermore, as one of ATM, we have effectively developed a variety of congestion mitigation tools, making use of this information.

3. Traffic control center

Our traffic control centers, which are located in four regional areas operating approximately 500km respectively, play a key role in making ATM work effectively. In close conjunction with the Maintenance/Customer Service Centers, the functions of the traffic control center include monitoring all road conditions and equipment operation, dealing with the incidents immediately, and providing real-time expressway-related information, and as a result minimizing the negative effects on the smooth traffic and recovering the expressway to the ordinary condition as soon as possible. It also coordinates with the Expressway Traffic Police Unit, Fire Department, and other safety and support agencies, and functions around-the-clock to ensure that expressways are constantly safe and reliable by dispatching tow truck, maintenance teams, ambulances and fire trucks. To minimize the negative effects such as secondary accidents and further expansion of traffic congestion and then recover expressways as soon as possible, the traffic control centers' systems feature automatic and prompt information dissemination. Figure 1 illustrates the road information flow centered on the traffic control center.

Although mobile devices are becoming popular as ways to obtain and disseminate incident information, from the view point of higher reliability and accuracy of information, utilizing the traffic control center is more indispensable. Furthermore, considering the importance of traffic control centers for securing smooth traffic and safety, we have had a recovery system in case of emergency, meaning that if one of the centers break down due to earthquake and inclement weather, another traffic control center which is integrated with others complements the disabled center.

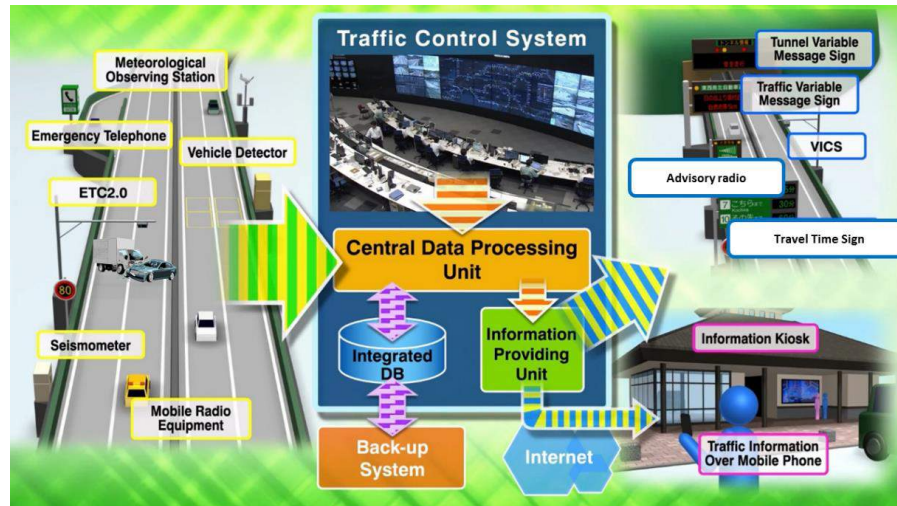


Figure 1. Road Information Flow

As for equipment for information collection and transmission, we actively collect the road information with multiple devices such as vehicle detector (traffic counter), meteorological observatories monitoring the temperature, the wind and the rainfall, and emergency telephones. Recently the latest equipment such as CCTV cameras and image processing traffic counters have been installed in certain major expressways. CCTV cameras automatically detect vehicles' speed, parked vehicles and wrong-way driving vehicles on expressways and image processing traffic counters automatically detect traffic congestions and calculate the number of vehicles in traffic. Figure 2 shows the pictures of the applied equipment. The collected information is transmitted to the center and provided to the drivers with various systems, including VMS, advisory radio, and Information Kiosk.



Figure 2. Road equipment for information collection

4. Traffic congestion in Japan

In order to build suitable congestion mitigation tools, we need to identify the causes of congestion in advance. The causes of congestion in Japanese expressways are described in Figure 3. 60% of causes occur at uphill and sag, 18% at tunnel entrances and 13% at merging sections. Therefore, the ratio of bottleneck in Japan occupies 90% of causes, meaning that most of congestions are caused by bottleneck. To solve this problem, we have used primarily two types of congestion mitigation tools, the increase in traffic capacity and the distribution of traffic demands.

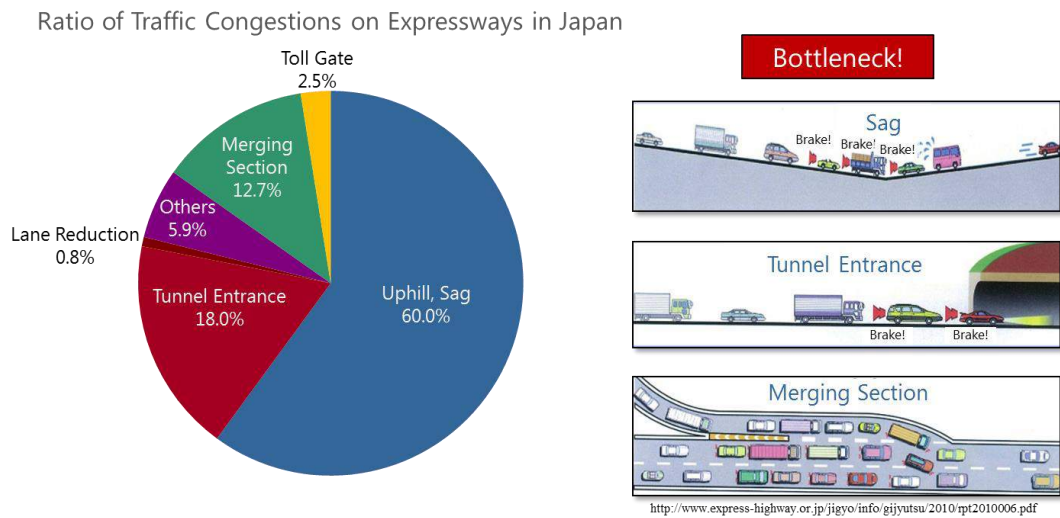


Figure 3. The cause of traffic congestion in Japanese expressways

5. Congestion mitigation tools

5.1 Increase in traffic capacity

First mitigation tools aim to increase in traffic volume, and they consist of two types, hardware and software. As for a hardware type of changing road length and structures, four mitigation tools including the network expansion, the ETC installation, the road widening, and the shoulder use, have been implemented. As for a software type of deploying equipment on expressways, VMSs for alerting reduced speed have been used at uphill, sag or non-recurring congestion point. Especially in urban areas with limited right-of-way, the hardware type with which road length and structures have to be changed according to the design could be difficult to introduce. Therefore, the needs in the software type such as VMSs for road operators operating urban or narrow areas are expected to be increased. Figure 4 shows congestion mitigation tools for increase in traffic capacity applied by NEXCO-Central.

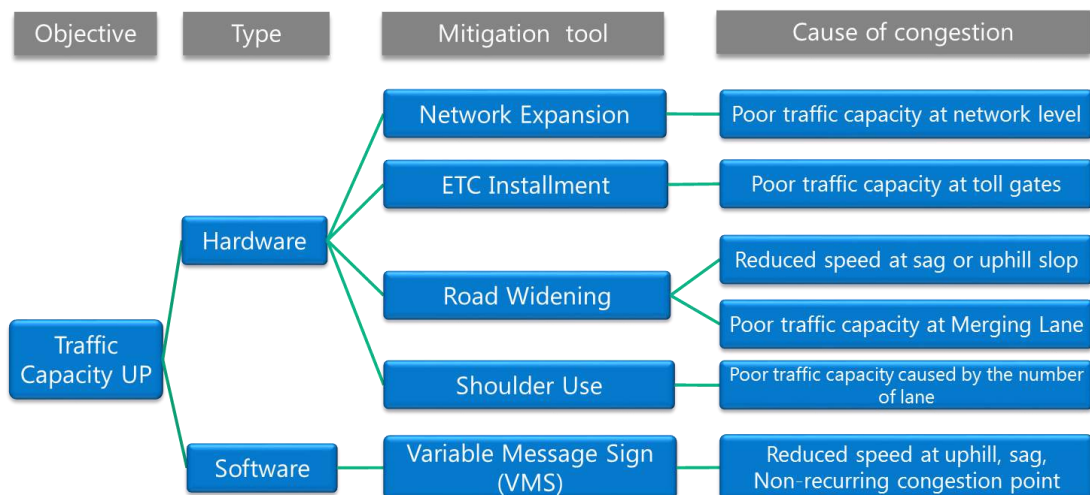


Figure 4. Congestion mitigation tools for increase in traffic capacity

5.1.1 Shoulder use

In order to increase the traffic capacity, we have considered a tool to effectively use a road shoulder as a traffic lane. We have temporarily operated the shoulder use in a certain section with heavy traffic volume

of over 100,000 vehicles per day on the Tomei Expressway. Originally this traffic volume was the largest among our 4-lane expressway and the congestion was constantly observed during rush hours. After the shoulder use was conducted, we obtained the incredible outcome that the congestion was reduced by 94% and the number of accidents was reduced by 30% (Figure 5). Moreover, since the shoulder has become narrowed from 3m to 0.75m after operation, the traffic safety measures were simultaneously applied to this section such as limiting the driving speed and installing the emergency parking space.

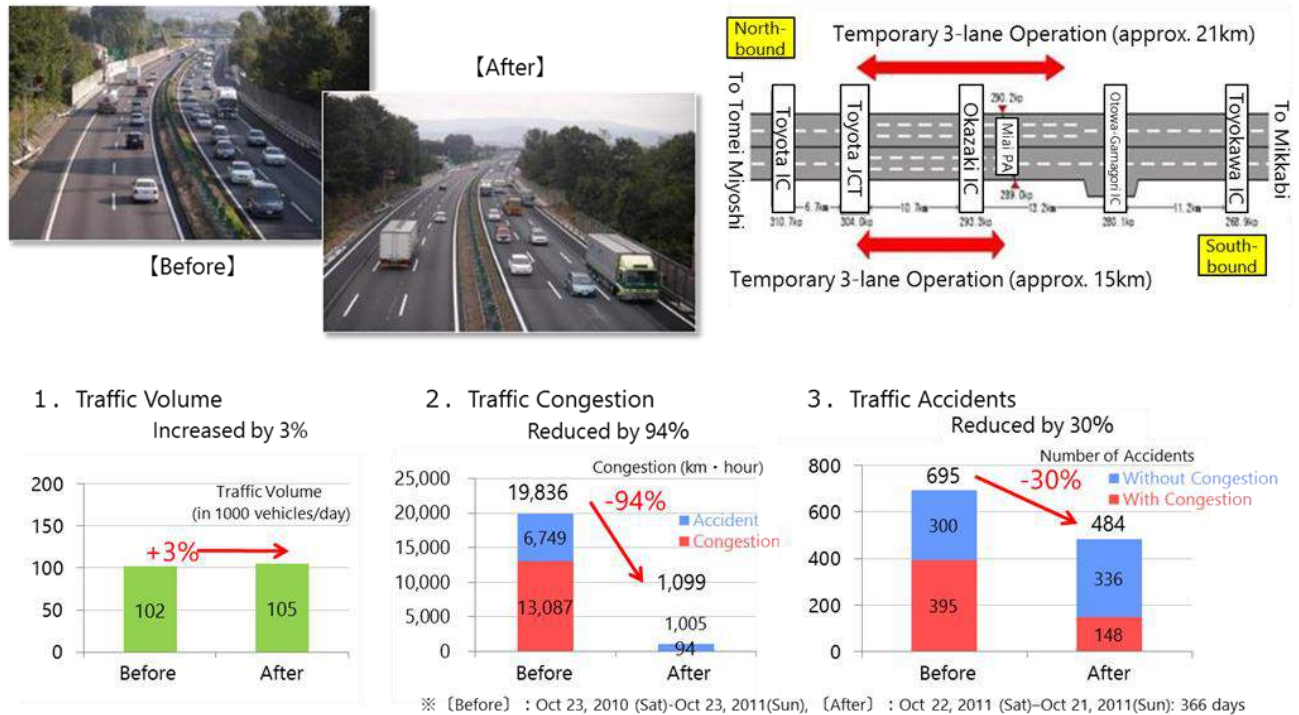


Figure 5. Example of temporary shoulder use and its effect

5.1.2 Variable Message Sign (VMS)

Take a look at an example of the bottleneck notification using VMSs at a tunnel. These purposes are to alert speed reduction caused by distracted driving and to encourage quick restoration of driving speed. Figure 6 shows how to place the VMSs and to prevent reduced speed of vehicles around the bottleneck of the tunnel. The displays of VMSs show messages of “Congestion End 1 or 2km Ahead” at 1 and 2km before the end of congestion, followed by blinking messages of “Congestion Ends at Tunnel Exit” and “Restore Speed” at the tunnel entrance. Finally, blinking messages of “Congestion Ends” and “Restore Speed” are displayed at the end of congestion again to remind the drivers. These displays help drivers realize the accurate location of the congestion end. By applying this method, the annual traffic congestion volume (km/h) has been reduced by up to 5% (Figure 7).

Although both fixed and mobile VMSs have been used, one of our features is mobile VMS. When we are not able to place the fixed sign or non-recurring congestions are found, this mobile VMS will be temporarily applied at any point on -site. VMSs become a helpful equipment for any road operator to take an action for congestion mitigation tools. Figure 8 shows the example of fixed VMS displaying “Restore Speed”, and Figure 9 shows the example of mobile VMS which is a vehicle with LED information to display “Restore Speed”.

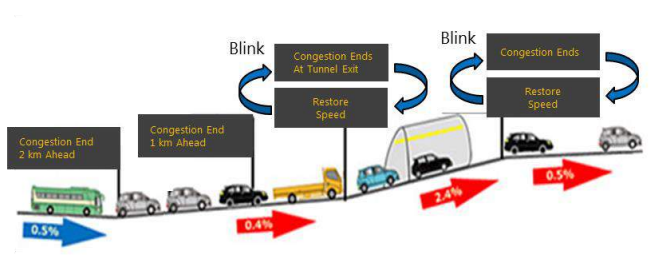


Figure 6. Use of VMS at a tunnel

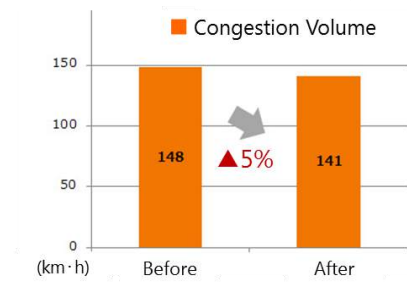


Figure 7. Effect of VMSs



Figure 8. Fixed VMS



Figure 9. Mobile VMS

5.2 Distribution of traffic demand

Second tools aim to distribute traffic demand, and they consist of a software type. This software type is classified into three mitigation tools including the proactive congestion prediction, the real time information provision, and the reduction in traffic regulation. By using these three tools selected according to each cause of congestion, it is expected to encourage drivers to change their choices of travel time and route. Figure 10 shows congestion mitigation tools for distribution of traffic demand applied by NEXCO-Central.

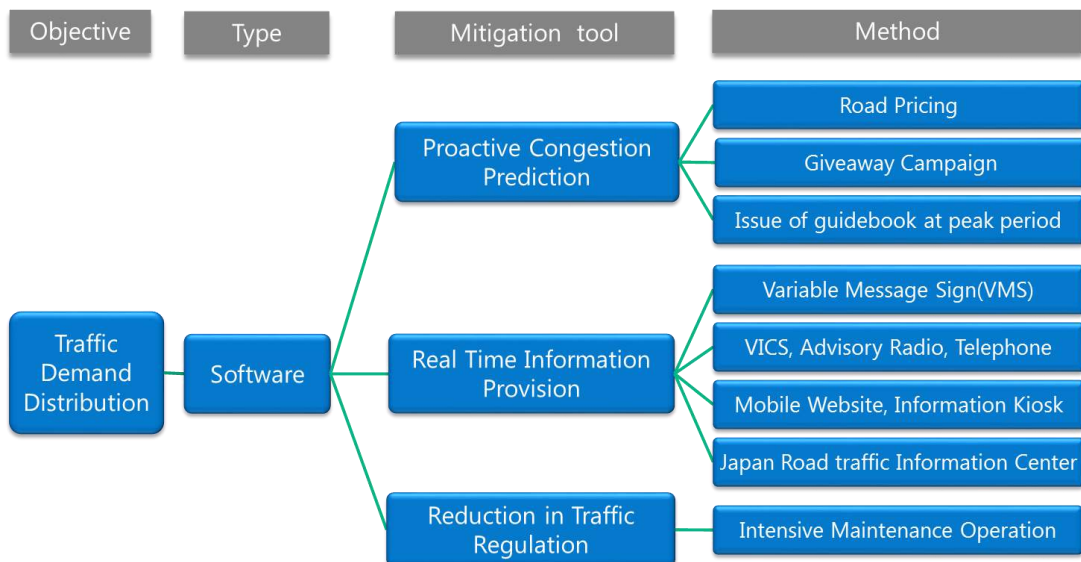


Figure 10. Congestion mitigation tools for distribution of traffic demand

5.2.1 Proactive congestion prediction

As a proactive congestion prediction, where congestion prediction is provided on a time-zone basis so that drivers can easily learn the rough travel time to their destinations, we employ the road pricing that changes the toll by peak time and off-peak time (Figure 11), the giveaway campaign, meaning that we offer giveaway to drivers passing during expected off-peak time, and the issue of a special guidebook at a peak period during a holiday season with predicted congestion information calculated based on the past data (Figure 12). The drivers are able to use this information to avoid the rush hours. With these approaches, we have successfully reduced the congestion by 5% in a certain section (Figure 13).

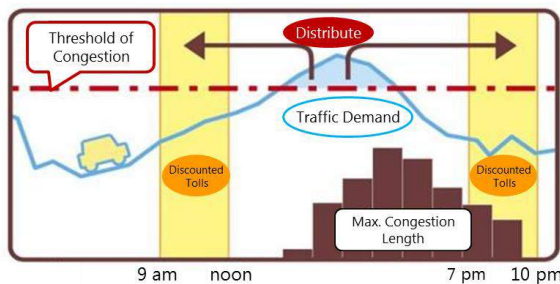


Figure 11. Concept of road pricing

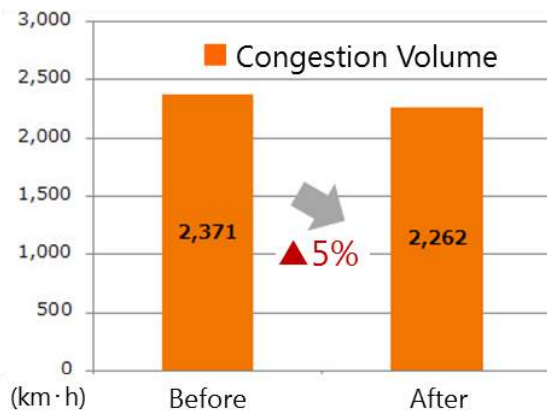


Figure 13. Effect of proactive congestion prediction



Figure 12. Giveaway Campaign

5.2.2 Real time information provision

5.2.2.1 Variable message sign (VMS)

As for real time information provision, we employ a variety of VMSs to change the traffic flow by notifying real time information to every driver and then making every driver avoid coming closer to a traffic congestion area. One of the features is travel time signs, with which recently the travel time, which adds general travel time into the travel time passing current events based on the traffic counter data or the past data can be displayed. Also, map-based graphic travel time signs have been adopted to achieve better user-friendliness especially for elderly people and enables drivers to select the most appropriate route, lead to the destinations in a short time and eventually balance the traffic volume. At present, these information provided by road operators are still more accurate, reliable, and efficient to control the traffic behavior than mobile apps, which can be one of the popular notification tools. Figure 14 shows examples of VMSs applied by NEXCO-Central.



Figure 14. Examples of VMSs

Table 1 shows the locations and objectives by VMS types in Shin-Tomei Expressway which was newly opened to traffic in 2012. The key thing is that all drivers are able to receive the information which enables them to select the most appropriate route and pay careful attention to incidents on the way whenever they need. Therefore, locations of VMSs and contents of displays have to be determined so that the maximum effects can be produced.

Items	Locations	Objectives
✓ Travel Time Sign	After the toll gate at all Interchanges (ICs)	To determine whether to use expressway To pay careful attention to incidents on the way
✓ Map-based Graphic Travel Time Sign	Before all Junctions where drivers have a choice of routes to one destination.	To determine which expressway to go To pay careful attention to incidents on the way
✓ VMS	Before entrance at all ICs	To determine whether to use expressway To pay careful attention to incidents on the way
✓ VMS for Wide-area Info	Before exit at the ICs with good alternate route	To determine whether to use expressway
✓ Mobile VMS	Before congestion area	To pay careful attention to congestion (Secondary accident prevention)

Table 1. Location of VMSs

5.2.2.2 Other Information provision

A variety of information dissemination media provide every single driver with the traffic information. For example, “Vehicle Information and Communication Systems (VICS)” provides the graphical traffic information with the drivers through the car monitors. “Highway advisory radio” is broadcasted along expressways, which provides the information before Interchanges. Traffic information is provided through “Our Mobile Website” and “Information Kiosk” at rest areas. Furthermore, “Japan Road Traffic Information Center” collects the traffic information nationwide and provides them through radio, telephone and the internet, updating them every 5 minutes. Figure 15 shows a variety of information dissemination media used by NEXCO-Central.

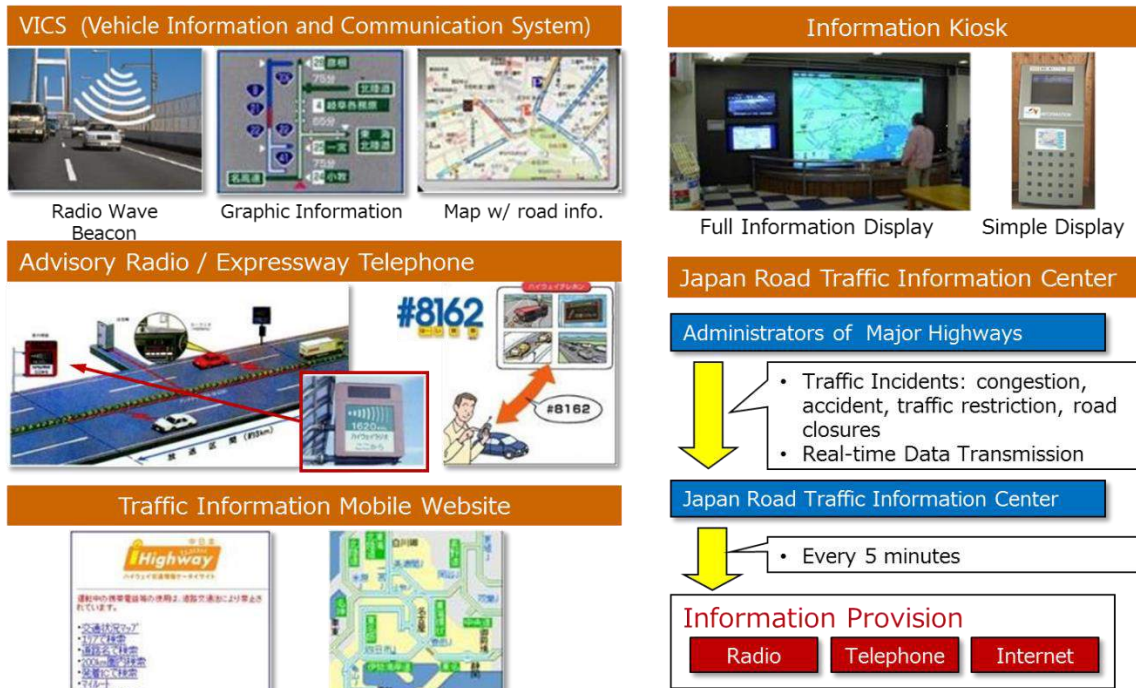


Figure 15. Information dissemination media

5.3 Reduction in traffic regulation

5.3.1 Intensive maintenance operation

Since regulating traffic flows on heavy traffic expressways can cause congestions, we have adopted the intensive maintenance operation to avoid traffic regulations, which control the movement of vehicles for safe traffic flow by restricting usable lanes, and to reduce congestions. That is to say, we have conducted short-term concentration of improvement and maintenance works instead of long-term separated improvement and maintenance works. As a result of this operation on the Tomei Expressway during two weeks in 2013, the number of traffic regulations was reduced by 40% from approximately 3,400 to 2,100, and then the number of traffic congestion was reduced by 70% from approximately 1,400 (assumed) to 450 (Figure 16, 17).

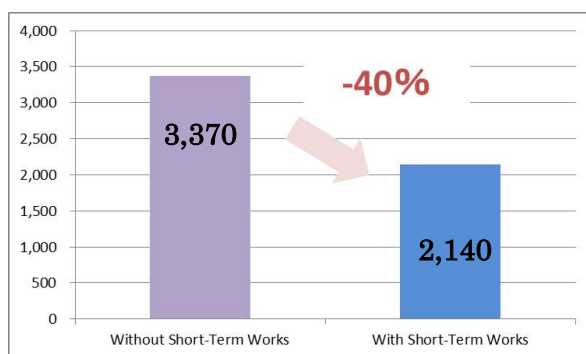


Figure 16. Effect on the number of traffic regulation

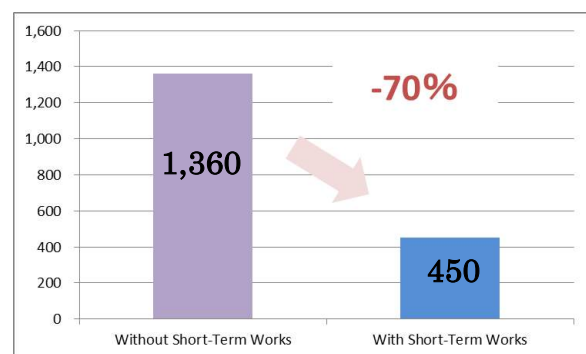


Figure 17. Effect on the number of traffic congestion

6. Traffic squad

In order to collect accurate road information promptly, we own and operate well-trained traffic squads by ourselves, which play an essential part in information collection along with other equipment. However, our traffic squads' role is not only to collect road information, but also to control traffic flow and provide roadside assistance. The key role of traffic squads is to patrol on the expressway 24/7 to deal with the incidents such as traffic accidents, disabled cars, falling obstacles as well as to check pavement and road

equipment. When traffic congestion occurs due to incidents, traffic squads notify the caution to drivers behind with an LED display mounted on a patrol car. Figure 18 shows traffic squads with patrol cars and equipped items they have on-site. Their role of keeping the expressway in a safe, secure and smooth condition is different from police officers, who enforce over-speed and illegal vehicles based on the law.



Figure 18. Traffic Squads with patrol cars and equipped items

When receiving a call from a driver in an accident, we notify the nearest traffic squads with dedicated digital radio and make an urgent request on rushing to the scene. They make an instant decision on required emergency vehicles and materials, and then request them to the traffic control center. Spontaneously, they regulate the lanes to alleviate the negative effects on the traffic. Furthermore, they guide the injured drivers and passengers into the safety area to avoid the secondary accident. Figure 19 shows traffic squads' activity against an accident on-site.



Figure 19. Traffic Squads' activity against an accident on-site

Another important role of traffic squads is to pick up fallen obstacles on expressways. For example, the big tool box and tires dropped from vehicles are extremely dangerous. Traffic squads run and pick up the obstacles before severe accidents occur. More than 24,000 fallen obstacles were annually removed under the direction of one traffic control center. Figure 20 shows traffic squads' activity of picking up fallen objects.

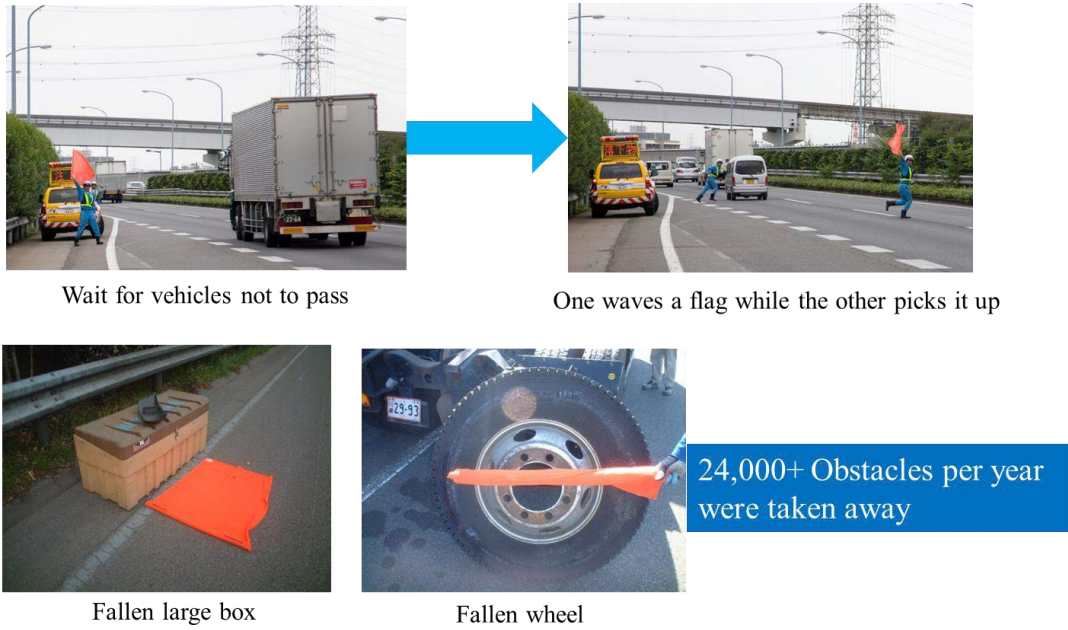


Figure 20. Traffic Squads’ activity of picking up fallen objects

7. Conclusion

In our active traffic management (ATM), the traffic control center with state-of-the-art technology has been established and promptly and accurately collect and provide road information for all drivers. Using this information, a variety of congestion mitigation tools have been carefully considered and implemented on-site. As a result of utilization of these tools at the congestion areas, these tools have brought the positive outcomes of reducing traffic congestion by increasing traffic capacity and distributing traffic demand. Besides, traffic squads also play an important role in working ATM effectively by collecting on-site information and securing smooth traffic.

We strongly believe that our ATM which are well organized with a variety of technologies and resources helps road operators keep safe, reliable and comfortable expressways by collecting and offering accurate information of the incidents and congestion ahead to every driver and as a result leading the reduction on traffic congestions and smooth traffic.

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PAPER TITLE	DEVELOPMENT OF THRESHOLDS FOR TRAVEL QUALITY ASSESSMENT IN COLOMBIAN URBAN ROADS		
TRACK	Asset Management		
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Travel quality, International Roughness Index, comfort, urban roads.

ABSTRACT:

Users' perception of travel quality constitutes a relevant parameter for determining the quality of road infrastructure. Therefore, establishing numerical values to quantify this parameter is imperative for the assessment and management of road networks. The International Roughness Index (IRI) has been widely used for this purpose. However, some limitations in using the IRI in urban roads have been identified as a result of the speed at which is modeled (80 km/h), which does not represent typical urban conditions. Subsequently, the main objective of this research study was to establish thresholds for IRI based upon the Weighted Vertical Acceleration (a_{wz}) parameter as an aid to assess the users' perception. The proposed methodology allows one to establish maximum allowable IRI values for a given road based on its operational speed. The results indicated that IRI thresholds are in agreement with international and local Colombian standards.

Development of Thresholds for Travel Quality Assessment in Colombian Urban Roads

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1 INTRODUCTION

The quality of a country's road network has a relevant impact on the economic development, as these provide communication links that allow the displacement of both citizens and essential commercial products. In this sense, the road network represents one of the most important aspects of infrastructure investments which represent a large share of the city budget, hence, decisions associated with its maintenance and rehabilitation must be taken under well-defined technical parameters.

Currently, management decisions taken for the maintenance and rehabilitation of urban roads are mostly based on experience rather than well-defined on the basis of technical information and economic projections over time. Over the last two decades, a conceptual change has been applied in Colombia with respect to: the quality of roads, maintenance criteria of road infrastructure and considerations on the role of safety and comfort of users of the network. Therefore, roadways must not only provide an adequate initial service, but also guarantee a suitable service condition during the projected design period, which requires a balance between the technical and economic components. Thus, both administrators and engineers should exercise caution when road investments are planned, and these must be supported by the functional and structural conditions of existing roads and projected traffic requirements.

Moreover, a pavement should provide a stable and secure surface for users, which will have a perception about the quality of travel, typically quantified by means of the serviceability index, which is defined as the ability of a pavement to provide a service to users, ensuring comfort and safety conditions for the traffic for which it was designed. Research has concluded that serviceability is affected to a great extent by the surface irregularities of the pavement, namely the roughness (Hass, Hudson, & Zaniewski 1994; Moore, Clark, & Plumb 1987). Moreover, there are different parameters to quantify the roughness of a pavement, among those, the most important and widely used is the International Roughness Index (IRI), introduced in 1986 by the World Bank, after previous efforts from a project funded by the National Cooperative Highway Research Program (NCHRP). IRI relates the accumulation of the relative vertical displacement in absolute value, from the upper to lower mass of a quarter car vehicle type called "Golden Car", divided by the total distance traveled at a constant speed of 80 km/h. The Golden Car is typically named Quarter car model, which is characterized by five constants: C_s (suspension damping rate), K_s (suspension spring rate), K_t (tire spring rate), M_s (sprung mass) and M_u (unsprung mass). The model accounts only for displacements in the vertical direction (Cantisani & Loprencipe 2010).

In spite of the various advantages and applicability of IRI, some investigations have shown that IRI has some limitations when used in assessing travel quality in urban roads. One of these is due to the speed of dynamic simulation (Quarter car model at 80 km/h), which does not represent the operating condition on urban roads, in which typically lower speeds are used (25 to 50 km/h). In addition, urban roads have singularities related to: 1) short sections, in general no longer than 100 meters; 2) intersections; 3) signage and, 4) vehicle traffic. All these variables have an impact over IRI measurements made by laser type high performance vehicles. Additional references regarding limitations of IRI on urban roads are available elsewhere (ISO 1997; ASTM 2009; Papagiannakis 1997; Papagiannakis 1998; Papagiannakis & Raveedran 1998; La Torre et al. 2002; Rayya & Kerry 2003; Nemmers et al. 2006).

In addition to restrictions of IRI for urban roads, previous research has identified some drawbacks associated with the use of laser devices for measuring IRI at vehicle operating speeds in urban roads. In this regard, high-performance lasers are widely used worldwide and have been used for more than 30 years;

however, there are still deficiencies in the systems that remain a research topic (Nemmers et al. 2006). Research has shown that changes in the longitudinal gradients adversely affect the accuracy in measuring the profile and the accuracy of the high performance devices when operating at low speeds (Choubane, McNamara, & Page 2002; Gagarin, Mekemson, & Lineman 2003; Karamihas & Sayers 1998; La Torre et al. 2002; Reggin, Shalaby, Emanuels, & Michel 2008). According to Sayers and Karamihas (1998), this inertial equipment requires specific speeds for proper operation. For example, these do not operate well at speeds lower than 15km/h. Furthermore, Gagarin, Mekemson and Lineman (2003) recommend that these type of equipments must operate at constant speeds near 40 mph (64 km/h). In practical terms, ensuring these optimum conditions in urban road becomes a challenge in terms of keeping constant and high speeds due to the presence of intersections and traffic interactions. Moreover, Karamihas (2005) stated that certain high-performance devices have difficulties in calculating IRI in textured concrete pavement, which lead to repeatability and reproducibility issues. In response to this issue, improved laser sensors have recently been used for improving the accuracy, reproducibility and repeatability.

Based on the limitations for IRI for assessment of travel quality in urban roads and the need for defining alternative parameters for its evaluation, this investigation aimed at developing specific thresholds that reflect the quality of urban road pavements for different operating speeds and provide indication on the travel quality experienced by the users of a local road network in Colombia.

2 REFERENCE METHODS FOR CALCULATING IRI THRESHOLDS

As presented in previous sections, the current concept of a standardized IRI does not adequately represent the operating conditions of urban roads since it is based on a dynamic model (Quarter Car Model) that uses 80 km/h as reference speed which, in turn, is higher than the speeds at which vehicles circulate in a city. As a result, this section focuses on previous research performed to account for this and other limitations of the use of the IRI in urban roads.

Yu, Chou and Yau (2006) proposed threshold levels of IRI for different operating speeds. These thresholds were defined from the changes of vertical acceleration of a vehicle named “Jolt”. 102 profiles were selected in rural roads from the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) database from 1989 to 2002. The authors also stated that different values of “Jolt” are presented for the same level of roughness IRI, determining the values of acceptance of IRI from a normal distribution. Table 1 shows the thresholds proposed by the authors.

Table 1. Suggested IRI thresholds for various operational speeds (Yu, Chou and Yau 2006).

Travel Quality	IRI thresholds for various operational speeds (m/km)				
	120 km/h	100 km/h	80 km/h	70 km/h	60 km/h
<i>Very Good</i>	<0.95	<1.14	<1.43	<1.63	<1.9
<i>Good</i>	0.95 – 1.49	1.14 – 1.79	1.43 – 2.24	1.63 – 2.57	1.90 – 2.99
<i>Fair</i>	1.50 – 1.89	1.80 – 2.27	2.25 – 2.84	2.58 – 3.25	3.00 – 3.79
<i>Mediocre</i>	1.90 – 2.70	2.28 – 3.24	2.85 – 4.05	3.26 – 4.63	3.79 – 5.40
<i>Poor</i>	>2.70	>3.24	>4.05	>4.63	>5.50
	50 km/h	40 km/h	30 km/h	20 km/h	10 km/h
<i>Very Good</i>	<2.28	<2.86	<3.8	<5.72	<11.44
<i>Good</i>	2.28 – 3.59	2.86 – 4.49	3.80 – 5.99	5.72 – 8.99	11.44 – 17.99
<i>Fair</i>	3.60 – 4.54	4.50 – 5.69	6.00 – 7.59	9.00 – 11.39	18.00 – 22.79
<i>Mediocre</i>	4.55 – 6.25	5.70 – 8.08	7.60 – 10.80	11.40 – 16.16	22.80 – 32.32
<i>Poor</i>	>6.25	>8.08	>10.80	>16.16	>32.32

Similarly, Ahlin and Granlund (2001) stated that, under some circumstances, IRI does not adequately describe the roughness of a road under vehicle-specific operational conditions. The authors also indicated that under some conditions (normal operating conditions of urban roads), IRI does not adequately

describe the users' ride quality and comfort. As a result, a methodology established by the ISO 2631 standard was proposed as an alternative evaluation. In particular, the ISO 2631 standard defines a procedure for measuring the vibrations perceived by a passengers' body during the journey on a vehicle: some vibrations are generated by the irregularities of the road, which are translated into vertical accelerations perceived by the passenger (Ahlin & Granlund 2002).

In the ISO 2631 standard: *Evaluation of human exposure to whole-body vibration* (ISO 1997), a widely used methodology for evaluating the effects that occur on the human body due to vibration exposure is presented: *basic evaluation method using quadratic weighted average acceleration* a_{wz} . This acceleration is also known as vertical weighted RMS acceleration (frequency-weighted root mean square acceleration). This parameter is mainly used in occupational health and risk exposure assessments to the effect of vibration. Table 2 shows threshold values for a_{wz} for public transportation according to the ISO 2631 standard.

Table 2. Threshold Values for a_{wz} for public transportation in accordance to ISO 2631 standard

a_{wz} (m/s ²)	Users' comfort level
< 0.315	Not uncomfortable
0.315 – 0.63	A little uncomfortable
0.5 – 1.0	Fairly uncomfortable
0.8 – 1.6	Uncomfortable
1.25 – 2.5	Very uncomfortable
> 2.0	Extremely uncomfortable

ISO 2631 standard was developed by the International Organization for Standardization (ISO) and is regarded to as a standard model adopted by several countries in the globe. For instance, some adaptations are presented as follows:

- Spain (UNE-EN ISO 5349-1): *Mechanical Vibrations: Measurement and evaluation of human exposure to vibration.*
- Portugal (NP ISO 2631-1): *Vibrações mecânicas e choque - Avaliação da exposição do corpo inteiro a vibrações.*
- Ecuador (NTE INEN ISO 2631): *Mechanical vibration and shock. Assessment of human exposure to vibration throughout the body.*
- United Kingdom (BS 6841): *Measurement and evaluation of human exposure to whole-body mechanical vibration.*
- Colombia (NTC 5435-1): *Mechanical vibration and shock. Assessment of exposure of humans to vibration throughout the body.*

Furthermore, a recent research focused on rural roads developed statistical correlations between IRI and a_{wz} , allowing for assessments of comfort when travelling on a pavement (vertical vibrations due to acceleration) (Ahlin & Granlund 2001; Ahlin & Granlund 2002; Cantisani & Loprencipe 2010; Hou Liang, Ma, & Wanghua 2009; Zhang & Yang 2010). More specifically, Ahlin and Granlund (2002), proposed a model defined on Equation (1) to determine the comfortable vehicle speed -CVS- from the limit values of acceptance of a_{wz} .

$$CVS = 80 \left(\frac{IRI}{5} \right)^{\frac{2}{1-n}} \quad (1)$$

Where:

CVS: comfortable vehicle speed, km/h.

IRI: International Roughness Index, mm/m.

n: amplitude parameter for the terrain roughness.

The n parameter values typically ranges between 1,1 and 2,5. The n value is high for roads where the wave amplitude of the prevailing pavement is long and conversely, the n value is low for a given road when the dominant wave amplitude pavement is short, as in the case of a deteriorated road.

According to Zhang and Yang (2010), RMS weighted vertical acceleration is directly related to traveling comfort perceived by the drivers and users of a road network. In accordance to ISO 2631, a_{wz} is calculated from vertical accelerations obtained in the Quarter Car simulation. The subsequent steps to calculate the parameters are:

1. The RMS acceleration of the mass of the passenger (a_{iz}^{RMS}) is calculated in terms of the Power Spectral Density for each i th octave thirds band.
2. a_{iz}^{RMS} values are multiplied by its weight factor (W_k), for each frequency band.
3. After these values are obtained (a_{iz}^{RMS} and W_k), the RMS can be determined from the square root of the sum of squares of these values as shown in Equation (2):

$$a_{wz} = \sqrt{\sum_{i=1}^{23} (W_{k,i} * a_{iz}^{RMS})^2} \quad (2)$$

Where:

$W_{k,i}$: factor for each frequency band weight.

a_{iz}^{RMS} : RMS Vertical acceleration of the mass corresponding to the passenger.

Similarly, Hou Liang, Ma and Wanghua (2009) evaluated the correlation between the International Roughness Index and Body Ride Comfort, concluding that the road displacement frequency power spectrum densities (evaluated in terms of a_{wz}) are not necessarily correlated to the IRI. Nevertheless, this research study attempted at developing additional correlations to validate the authors' findings as further shown in the paper. More recently, Cantisani and Loprencipe (2010) provided direct correlations between IRI and a_{wz} for different vehicle operating speeds focusing on rural roads. 124 pairs of profiles of rural roads were used (124 footprint left and 124 footprint right), which were collected by the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) of the Transportation Research Institute at the University of Michigan (1998). Initially, IRI was evaluated according to the standard procedure described by ASTM and calculated the average of each pair of profiles as indicated on Equation (3).

$$IRI = \frac{IRI_{left} + IRI_{right}}{2} \quad (3)$$

Then, the a_{wz} parameter was determined from the model simulations with Quarter Car, where the accelerations in the front passenger seat were taken. This calculation for acceleration was performed using different simulation speeds from 30 km/h to 90 km/h in increments of 10 km/h.

Additional research has been conducted in the Colombian context by the Urban Development Institute (IDU) in Bogota, 2007. The study included an inventory of the surface quality of 842 kilometers of pavements in different sectors of the city using a high-performance laser device, which measured IRI without changes in traffic real conditions. The results showed excessively high IRI values (average of 7.63 m/km) that are consistent with those for pavements with significant surface irregularities.

3. FIELD EVALUATIONS OF IRI IN COLOMBIAN URBAN ROADS

Measurements on several road sections constituted by rigid pavements were conducted in Barranquilla (Colombia) as shown in Figure 1. In these sections, the surface condition was evaluated by using IRI on the right wheel path of the right lane.

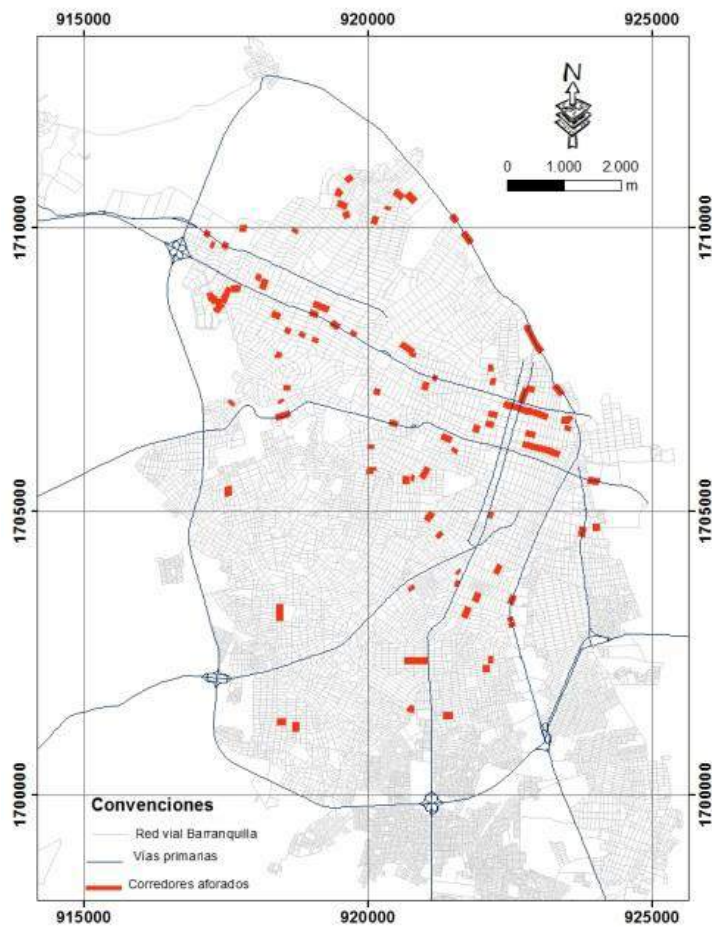


Figure 1 Distribution of the pavement sections evaluated in Barranquilla, Colombia.

For collecting the data two (2) devices were used: SurPRO (Figure 2a) and a high performance vehicle (Figure 2b). The SurPRO device is used for measurement of pavement profiles with high accuracy with a limiting maximum operating speed of 4 km/h (Nazef, Mraz, Scott, & Whitaker 2008). The high performance Laser Profilometer was used to measure pavement roughness at different operational velocities.



(a)

(b)

Figure 2. (a) SurPRO device and, (b) Laser Profilometer.

4. COMPARISON OF IRI MEASUREMENTS FOR SurPRO AND LASER PERFILOMETER

Figure 3 depicts that IRI values evaluated using the SurPRO are consistently lower than those obtained using the laser profilometer for all the evaluated pavement sections. The data shown in Figure suggests that, as the operating speed of the laser profilometer increases, roughness levels approach those obtained with the SurPRO. This results highlights the limitations encountered when attempting to evaluate roughness condition in urban roads using a laser profilometers, since these cannot always operate at high speeds due, among other factors, to: traffic signs, singularities, traffic lights and interaction with other vehicles.

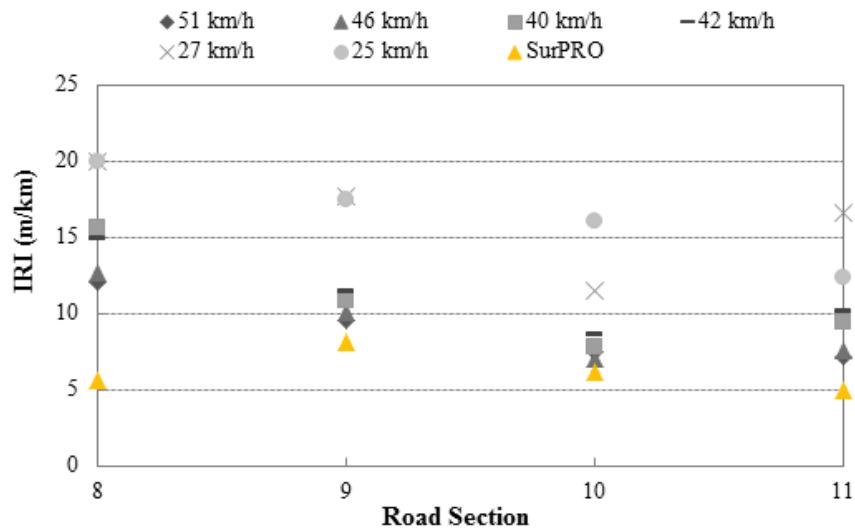


Figure 3. Comparison of SurPRO and Profilometer IRI-measurements at different speeds.

5. PROPOSED THRESHOLDS FOR TRAVEL QUALITY ASSESSMENT BASED ON AWZ PARAMETER

As a contribution of this research study, thresholds for the International Roughness Index are proposed. For this purpose, RMS weighted vertical acceleration (a_{wz}) is used to estimate the users' perception of accelerations. In this respect, this study-included measurements of 102 urban profiles constituted by rigid pavements were conducted in Barranquilla using the SurPRO (Figure 2a). Due to the nature of urban roads, variations occurred in the lengths of the sections evaluated. As depicted in Figure 4a, there is a mathematical relationship between a_{wz} evaluated at 80 km/h and the IRI, which was also calculated using the reference standard speed of 80 km/h. Nevertheless, as shown in Figure 4a, high dispersion in the dataset was found as indicated by the low R^2 (0.51). Based on the same procedure, trends for each of the a_{wz} evaluated using different simulation speeds are shown in Figure 4b.

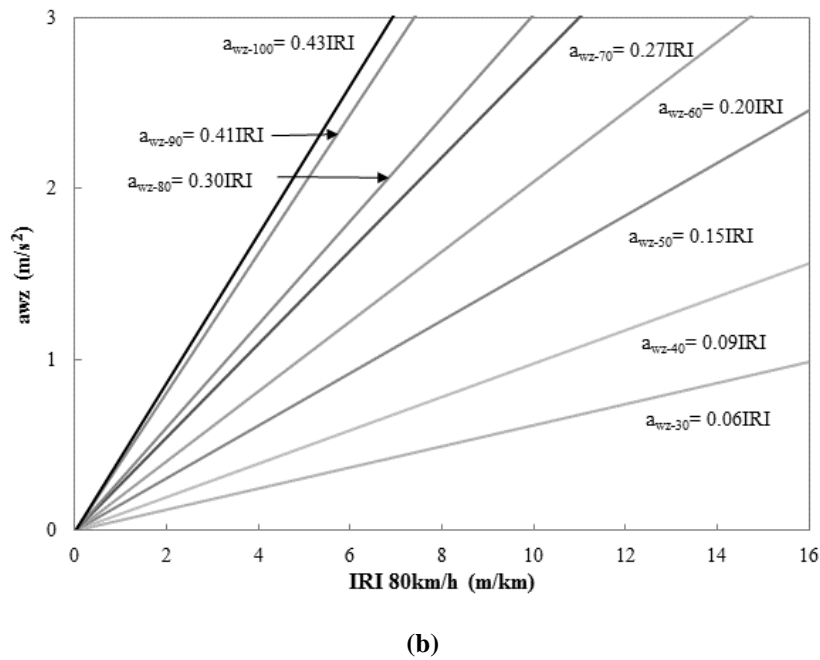
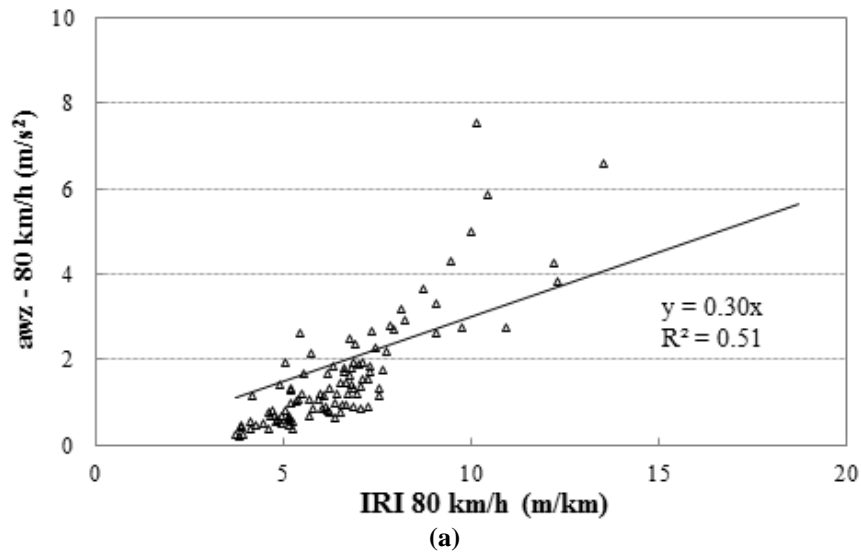


Figure 4. (a) Correlation between IRI and a_{wz} parameter at 80 km/h and, (b) correlation between IRI and a_{wz} parameter at evaluated at different speeds.

On the basis of the a_{wz} thresholds proposed by Cantisani and Loprencipe (2010), we proposed new thresholds of International Roughness Index using the linear regressions presented above (Figure 4b). Figure 5 depicts the location of the thresholds for 80 km/h. This same procedure was performed for each of the remaining velocities, giving as result thresholds of International Roughness Index - IRI to different operating speeds (Table 3).

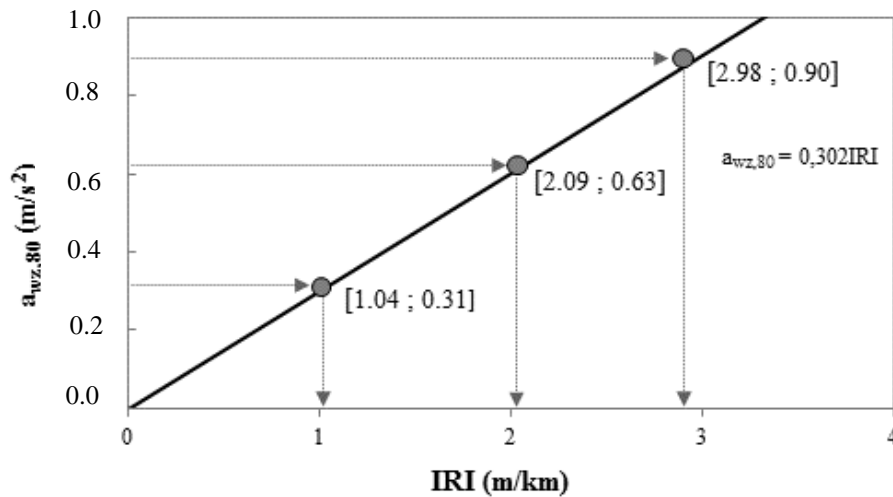


Figure 5. a_{wz} parameter limits for the assessment thresholds International Roughness Index - IRI.

Table 3. Proposed IRI thresholds for various operating speeds.

Road operating speed (km/h)	Proposed values for IRI (m/km)				Slope (Equation)	R^2
	Very Good	Good	Regular	Bad		
100 km/h	< 0.73	0.73 - 1.45	1.45 - 2.07	> 2.07	0.43	0.49
90 km/h	< 0.78	0.78 - 1.55	1.55 - 2.22	> 2.22	0.41	0.42
80 km/h	< 1.04	1.04 - 2.09	2.09 - 2.98	> 2.98	0.30	0.52
70 km/h	< 1.15	1.15 - 2.31	2.31 - 3.29	> 3.29	0.27	0.41
60 km/h	< 1.54	1.54 - 3.09	3.09 - 4.41	> 4.41	0.20	0.49
50 km/h	< 2.05	2.05 - 4.1	4.1 - 5.86	> 5.86	0.15	0.47
40 km/h	< 3.23	3.23 - 6.47	6.47 - 9.24	> 9.24	0.10	0.52
30 km/h	< 5.1	5.1 - 10.19	10.19 - 14.56	> 14.56	0.06	0.48
a_{wz} Parameter Limit Values(m/s²)	< 0.315	0.315 - 0.63	0.63 - 0.90	> 0.90		

As presented in Table 3 and Figure 6, higher levels of roughness are permitted in low-speed roads due to the fact that users will perceive less discomfort as a result of lower vertical accelerations experienced on the same road while traveling at lower speeds. Conversely, the limits become more demanding as the operating speed of the road increases. The results shown in Figure 6 may be different from those obtained by Cantisani and Loprencipe (2010) considering that different road types were used to define the thresholds. Hence, the discrepancies could be related to the nature of the roads considered for this research (urban roads), whereas those included in their study, which were highways. In addition, the previous results allow for evaluating roads at different speeds with tighter criteria that provide indication of the comfort level experienced by the users at different speeds while circulating on the road.

Furthermore, in order to validate the findings of this research, a comparison of the proposed threshold is made with other regional and local regulation and standards. For this purpose, using the information from Table 3 and Figure 6 and selecting 80 km/h as reference speed, an IRI value of 2.98 m/km is found, which would correspond to a bad road in terms of rideability and users' comfort. Then, when comparing this threshold with those proposed in Portugal and Spain by similar research, the values are in close agreement ($2.98 \text{ m/km} \approx 3.0 \text{ m/km}$). More specifically, the Colombian standard establishes a maximum

allowable value of 3.0 m/km for 100% of a given road under a specific traffic level classified as NT2, which once again, is very close to that found in this research study.

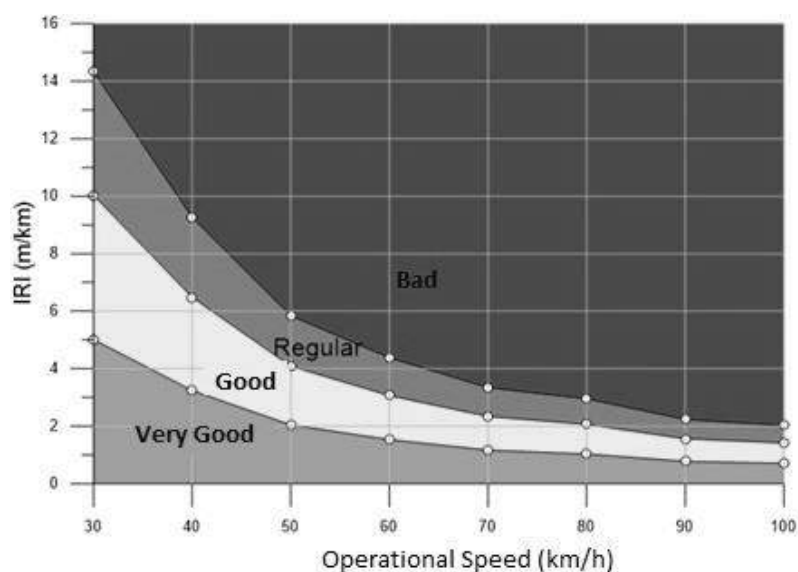


Figure 6. Thresholds for International Roughness Index at different operational speeds.

6. CONCLUSIONS AND RECOMMENDATIONS

This research is aimed at developing thresholds for assessing travel quality in urban roads. An a_{wz} parameter that accounts for users perception of road quality was used to complement the roughness assessment which is traditionally based solely on the International Roughness Index (IRI). The main conclusions of this research are presented below:

- Travel speed has a significant impact on the perception of road quality that users have as they travel in a specific pavement section. In particular, it was found that the reference speed used to calculate the IRI (modelling speed of 80 km/h) is not representative of the actual operating speed in which users normally travel on urban roads (30 to 60 km/h). Subsequently, evaluating surface roughness and users' comfort should be conducted at specific speeds for specific road types, which in turn, could provide more realistic data to be used in pavement management systems.
- Some deficiencies associated with laser profilers on urban roads were identified as a result of traffic signs, singularities, traffic lights and interaction with other vehicles that prevent accurate and realistic measurements of IRI and might induce inappropriate characterization of the surface roughness. Therefore, the use of high-performance laser equipment on urban roads is recommended only when very low traffic flow or partial closure of the road is available in order to take measurements at the recommended speeds.
- Considering that speed is rarely considered as a key parameter in evaluating road quality and users' perception of ride quality in the local context, an additional contribution of this research is proposed. Thus, thresholds for International Roughness Index for evaluation of users' comfort at different speeds were determined. These thresholds have been developed according to the nature of urban roads and its wide range of travel speed, which influences different perceptions of pavement surface quality.

Finally, the proposed thresholds for IRI are congruent when compared with the maximum allowable values by international standards and Colombian standards (INVIAS). The thresholds obtained allow not

only the definition of permissible maximum values for a road section, but also provide insight into the evolution of user comfort throughout the life of the pavement (relationship between IRI and user comfort); so to determine the ability of a road to increase the operating speed according to IRI values on the road.

7. ACKNOWLEDGEMENTS

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PAPER TITLE	SAFETY IMPROVEMENTS TO NATIONAL ROUTE TWO SECTION 27 IN KWAZULU-NATAL, SOUTH AFRICA		
TRACK	Road Safety		
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<p>KEYWORDS:</p> <p>South African National Roads Agency</p> <p>Decade of Action</p> <p>Road Safety</p>

<p>ABSTRACT:</p> <p>Road safety is of paramount importance to South Africa. Therefore the South African National Roads Agency has made it one of its nine strategic objectives. South Africa is regarded to be amongst the highest road fatality countries in the world. Due to the high number of accidents, the Civil Engineering industry is challenged to improve and retrofit highways in order to maintain a safe South African road network.</p> <p>This paper will discuss the strategy adopted to undertake the identification of hazardous curves and improvements made in order to eliminate motor vehicle accidents along the northern part of National Route N2 in the province of KwaZulu-Natal, South Africa in the most economical manner. Design proposals will be compared and discussed which will aid in illustrating why the most favourable option was selected. This paper will also discuss the implementation of a road safety audit which ensured an optimum design solution covering all aspects of road safety.</p>
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INTRODUCTION

South Africa is regarded to be among the highest road fatality countries in the world. Due to the increasingly high number of road fatalities, in 2015, South Africa was ranked 6th internationally for fatalities on roads. This is shown below in Figure 1. (World Atlas, December 2015)

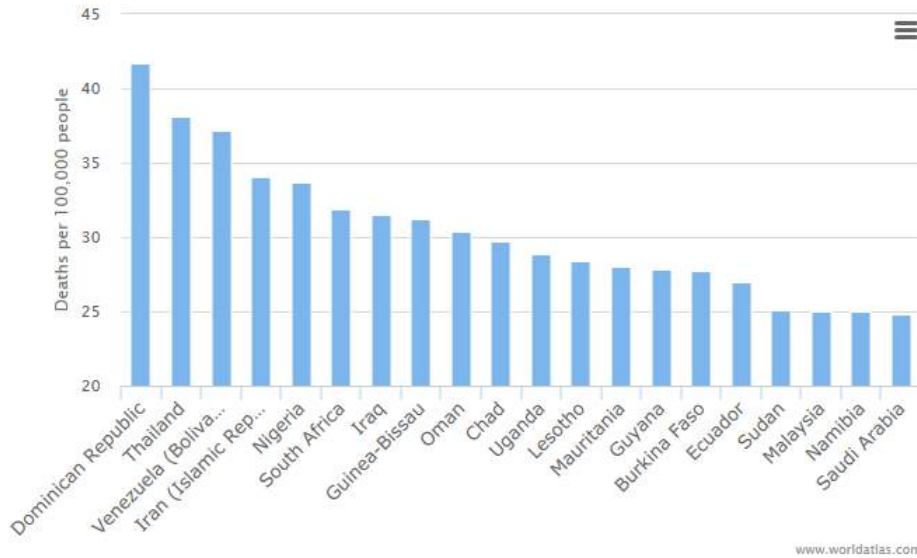


Figure 1. Number of fatalities per country across the globe due to road accidents in 2015

Figure 1 above shows that South Africa is not the only country that experiences a high number of road fatalities and this has become a global problem. In an attempt to remedy this global problem the General Assembly Resolution 64/255 adopted on 2 March 2010, requested that the World Health Organization and the United Nations regional commissions in cooperation with the United Nations Road Safety Collaboration and various stake holders, prepare a Plan of Action as a guideline document to support and assist the drive to reduce road fatalities. This Decade Plan of Action was launched in May 2011 to assist and attempt to reduce the number of road fatalities globally 2020. In 2011, the global road fatality toll was approximately 1.3 million according to the Global Plan for the Decade of Action for Road safety 2011-2020.

Road safety is of paramount importance to South Africa. In February 2007 the Minister of Transport and the Health of African States adopted the Accura Declaration that noted the deteriorating condition of transport infrastructure in Africa which encouraged member states to use the World Health Organization/World Bank World Report on Road Traffic Injury Prevention (*Geneva, 2004*) as a framework or guideline for road safety and implement its recommendations to reduce the causes and risks associated with road crashes (*SARSAM, 2012*).

In Figure 2 below the number of road fatalities with South Africa from 2011 to 2015 are shown.

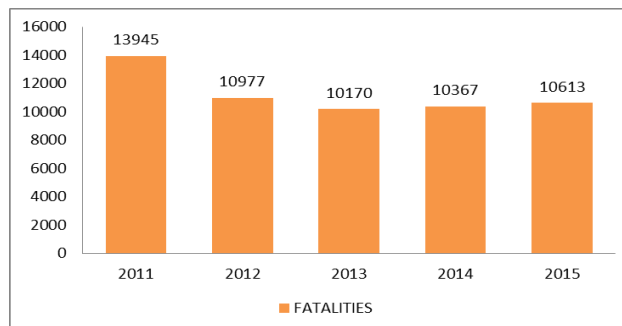


Figure 2. Number of Fatal accidents in South Africa from 2011 to 2015 (*Road Traffic management Corporation, Department of Transport, 2011-2015*)

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From Figure 2 above it can be seen that in 2011 there were 13945 fatalities and in 2015 there were 10613 fatalities on South African roads. This is approximately a 24% reduction in the number of road fatalities between 2011 and 2015. It is evident from the statistics provided above that South Africa has reduced road fatalities from 2011 to 2015 which coincides with the objectives of the Decade of Action to reduce the number of crashes by 50% in 2020. Every accident has an effect to the economy of the country. Table 1 below indicates the average cost of a fatal accident in South Africa.

Table 1. Average Fatality Cost in South Africa

Average Fatality Cost in South Africa (2015)			
City of Cape town report (2005)	Tshwane- Road Safety Master Plan Manual (2008)	National Department of Transport (2004)	Average
R 1 160 654	R 1 756 342	R 1 213 493	R 1 376 830

The average cost of a fatality is related to the total value of a person that could have benefited the economy of South Africa over his/her life time. Based on the average cost of R 1 376 830 per fatality in South Africa, the total loss to the economy in 2011 was approximately R 19 billion and in 2015 was approximately R 14.5 billion due to road fatalities. From this it can be seen that there has been a saving of approximately R4.5 billion to the economy due to the reduction in the number of fatal crashes in South Africa. This money can be utilised in various other aspects such as, infrastructure development, health services, housing, education etc. Hence the reduction in road fatalities is something that will definitely strengthen the economy of the country and provide an opportunity for the country to enhance other social aspects that requires improvement.

South Africa developed a Road Safety Manual (*SARSM, 1999*) to assist road authorities with the evaluation of traffic operations and assessment of road safety aspects along the road network nationally. The SARSM was published in 1999 as a draft document and consisted of the following volumes:

- Volume 1: Principles and Policies
- Volume 2: Road Safety Engineering Assessment on Rural Roads
- Volume 3: Road Safety Engineering Assessment on Urban Roads
- Volume 4: Road Safety Audits
- Volume 5: Remedial Measures and Evaluation
- Volume 6: Roadside Hazard management
- Volume 7: Design for Safety

Based on the recommendations of the World Health Organisation’s world report on road traffic injury prevention, countries were encouraged to focus on the following five road safety pillars, namely (*Implementing the 2011-2020 Decade of Action in Sub-Saharan Africa, 2010*) :

- Pillar 1: Road Safety Management
- Pillar 2: Safer Roads and Mobility
- Pillar 3: Safer Vehicles
- Pillar 4: Safer Road Users
- Pillar 5: Post-Crash Response

In 1998 the South African National Road Agency SOC Ltd (SANRAL) was established as an independent, statutory company with a distinct mandate to finance, improve, manage and maintain the national road network within South Africa. SANRAL currently manages 21 403 km (2015) of national roads in South Africa. SANRAL’s direct sphere of influence relates specifically to the Pillars of Safer Roads and Mobility, Safer Road Users, and Post-Crash Response. For Safer Roads and Mobility reference to road infrastructure interventions relates directly on the road network, for Safer Road Users reference is made to Road Safety Education and Awareness Programmes, and for Post-Crash Responses reference is made to the rollout and operations of Road Incident Management Systems on the entire national road network. These are the main focus areas where SANRAL will continue to promote and improve road safety for all road users.

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In 2005 the Road Traffic Monitoring Corporation (RTMC) was established with the objective of pooling powers and resources to eliminate the fragmentation of responsibilities for all aspects of road traffic management across various levels of government in South Africa. The RTMC has embarked on a project that focuses on infrastructure safety audits to promote safer roads which is one of the pillars of the Decade of Action, through the revision of the South African Road Safety Audit Manual (Volume 4) into a stand-alone South African Road Safety Audit Manual (SARSAM). The intention for this manual is to be used by all road authorities nationally to conduct road safety audits in order to identify potentially hazardous locations and implement remedial measures to reduce the number of accidents and fatalities.

For the purpose of this paper, the strategy adopted to undertake the identification of hazardous curves and improvements made in order to eliminate motor vehicle accidents along the northern part of National Route 2 (N2) in the province of KwaZulu-Natal will be used as a case study. The location of this case study is shown in figure 3 below.



Figure 3. Locality Map

This section of the N2 has been a notorious area for accidents and fatalities. The section of road has an Average Daily Traffic (ADT) count of approximately 10,600 vehicles per carriageway of which 12% are heavy vehicles (trucks). According to accident records, between February 2011 and February 2012, on this section of the N2, there have been 87 vehicles involved in accidents of which 5 were fatalities. This project was therefore identified as a road safety improvement project to identify hazardous locations and to reduce the accident rate along this section of the N2. SANRAL appointed a Consulting Engineering Firm to undertake investigations to determine the possible cause of the accidents and propose possible remedial measures.

METHODOLOGY

The approach and methodology adopted to obtain an optimum solution to the ongoing road accidents will be discussed under this section of the paper. It should be noted that even though SANRAL was the client and appointed service providers to undertake the investigations, SANRAL played a fundamental and key role in the management of the service providers, interpreting the findings and decision making. In this instance SANRAL played the role of the client including client technical input as well as the overall project manager on this contract.

As part of the investigation SANRAL adopted the following methodology:

- a) A full Road Safety Audit of the existing road,
 - o Interaction with national and local traffic authorities to obtain accident statistics and information.
 - o Visual Inspection of the Road
 - o Determine operating speeds/speed profiles
 - o Undertake the Road Safety Audit as per SARSAM

- b) A preliminary design of the proposed safety improvements,
 - o Analyse the existing geometry of the road
 - o Check compliance in terms of design speed
 - o Check compliance in terms of operating speed
 - o Prepare proposals for improvements

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- c) A detailed design of the selected proposal
- o Proceed with selected design proposal
 - o Optimise the solution in terms of costs
 - o Finalize design

It should also be noted that as all contracts are governed by a financial budget, it is the responsibility of the Engineer to produce a financially viable solution. Notwithstanding, this statement should not be interpreted that South Africa and SANRAL will compromise road safety because of financial constraints. All that is required is that all proposals and solutions are optimized to eliminate wasteful expenditure.

From the methodological approach as indicated above, it was envisaged that the outcome of such an approach would enable one to determine the exact locality of the accidents and the potential causes. This approach would also allow SANRAL to remedy any potential technical factors that may have contributed to accidents.

RESULTS AND ANALYSIS

Road Safety Audit and Visual Assessment of the Road

As part of the road safety audit the initial investigation was to obtain any accident records from national and local traffic authorities and analyse the existing accident records. Accident records were obtained from three (3) sources, namely:

- South African Police Services (SAPS)
- Road Traffic Inspectorate (RTI) – (Road Traffic Police)
- Intertoll – (Operator of the Toll on this section of the N2)

The SAPS report indicated the number of accidents within the section of road over a length of approximately 15km but however did not indicate the actual location of the accidents. Table 2 below, shows the statistics from SAPS.

Table 2. Accident Statistics obtained from SAPS

YEAR	NUMBER OF ACCIDENTS
2010	72
2011	120
2012	69
2013	42 (for the 1 st half of 2013)

From Table 2 an average of 87 accidents per annum can be expected on this section of the N2. However these records were not of much help as they were unable to indicate the exact locations of the accidents in order to streamline the investigation.

The accident records from the RTI grouped accidents into two (2) locations, namely, uMhlali River Bridge and Mvoti River Bridge. Accident records for the period 2011 and first half of 2013 are summarised in Table 3 below.

Table 3. Accident Statistics obtained from RTI

LOCATION	YEAR	TOTAL	FATAL	MINOR	MAJOR
uMhlali River Bridge	2011	7	1	5	2
	2012	16	3	7	9
	2013	5	0	2	3
Mvoti River Bridge	2011	5	2	3	3
	2012	6	0	1	5
	2013	3	1	2	1

The records obtained from the RTI clearly indicated that these two (2) locations where the problematic areas within this section of the N2.

The records obtained from Intertoll were for the period between 2009 and 2013. These records were in great detail which allowed for a more-in depth analysis. This assisted the analysis of information with more historical data which resulted in a more accurate position of the locations of the accidents. Fortunately, the records from Intertoll included the exact locations as well. The two (2) locations that were prevalent after analysis was again the Umhlali River Bridge and

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the Mvoti River Bridge. Therefore it could be concluded from this initial accident record investigations that these two locations were the high accident risk zones.

The locations of these two (2) accident zones can be seen in Figure 4 below.



Figure 4. Locations of the Two (2) Accident Zones

Further investigations with regards to the speed profiles along this section of the N2 were undertaken. The results of this can be seen in Table 4 below.

Table 4. Indicating Speed Profiles

TEL STATION	SECTION	KM	AVERAGE SPEED [Km]		85% SPEED	
			LIGHT	HEAVY		
1279	Salt Rock Piezo	N2/27	9.8	107.5	82.0	121.9
799	Umhlali i/c	N2/27	11.7	111.0	83.2	125.9
2231	Mvoti Plaza 2	N2/27	21.6	107.6	80.3	123.9
1069	Groutville i/c	N2/27	23.4	105.7	74.9	122.0
801	Stanger i/c	N2/27	30.0	111.1	77.0	125.9
802	Zinkwazi	N2/27	41.0	107.1	74.2	123.9

It should be noted that all the Stations in the Table 4 provide the 85th percentile speed on the section of the N2 shown in Table 4 above.

According to the Geometric Design Guidelines of SANRAL, the selected design speed should accommodate the 85th percentile desired speed that is likely to materialize. According to the results from the speed profiles these two (2) curves ideally should have a design speed of 120km/h. Detailed surveys were requested to be undertaken in order to determine the actual horizontal and vertical geometry of the curves. From the survey and further analysis of the curves in geometric design software one was able to create the exiting geometry of the curves. This clearly showed that the curves are adequate for a 100km/h design speed but insufficient for 120km/h. It should be noted that this section of the N2 is sign posted at a recommended speed of 100km/h for motorists. So legally the design speed is complaint with the sign posted speed. However, it is evident from the speed profile investigations in Table 4 above that motorists are exceeding this sign posted speed of 100km/h.

In addition, further investigations were undertaken as indicated in the South African Road Safety Audit Manual (SARSAM). The investigations can be summarised in Table 5 below.

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Table 5. Investigation Summary (SARSM,2012)

ITEM	SUB-ITEM	COMMENTS
General Topics	Landscaping and Natural Vegetation	OK
	Headlight Glare	OK
	Parking	OK
	Temporary Works	N/A
Cross-section and Alignment	Visibility and Sight Distance	OK for 100km/h but Not OK for 120km/h
	Design Speed and 85th percentile operating speed	NEEDS ATTENTION
	Overtaking	OK
	Readability by Drivers	OK
	Widths	OK
	Batter slopes	OK
	Drainage structures	OK
Auxiliary lanes and Exclusive Turning Lanes		N/A
Intersections	-	N/A
Special Road Users	-	N/A
Road Signs, Markings, Delineation and Lighting	Road Signs	OK
	Road Marking and delineation	OK
	Road Lighting	N/A
Traffic signals	-	N/A
Driver Perception		
Pavement	Loose gravel	OK
	Pavement Defects	OK
	Skid Resistance	OK
	Ponding	NEEDS ATTENTION

Table 5 above shows that from the investigations undertaken that two (2) items can potentially be the contributing factors to the cause of the accidents, namely the *Design Speed* (85th percentile operating speed) and the *Ponding issues* on the pavement that have been identified during site inspections. Ponding was identified within the development of the superelevation on both horizontal curves at Umhlali and Mvoti River Bridge

The Road Safety Audit investigation highlighted the following findings:

- Umhlali and Mvoti River Bridge are the 2 locations where accidents occur within this section of the N2
- The exiting curves at these locations were designed with a design speed of 100km/h, and the sign posted speed is 100km/h
- The speed profile investigation indicated that the operating speeds at both these locations are in excess of 120km/h.
- The curves at Umhlali and Mvoti River Bridge require geometric improvements for 120km/h operating speeds.
- Ponding was evident at these curves.

These findings will be discussed and remedied under the preliminary and detailed design phase.

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Preliminary and Detailed Design

For the purpose of the preliminary and detailed design, the SANRAL Geometric Design Guidelines and the G2 Geometric Planning Manual were used. The designs for the safety improvements will concentrate on the curves at Umhlali River Bridge and the Mvoti River Bridge, as these were identified as the most critical curves where safety improvements were required. The situational analysis of both curves can be summarised as follows:

- The speed limit on this section of the road is 100km/hr and is clearly signposted.
- The existing horizontal curve conforms to the design criteria of a 100km/hr design speed.
- The vertical alignments on both carriageways conform to the design criteria for a 100km/hr design speed.
- This section of road has been identified during the initial visual inspections and road safety audit to be an area of high accident rate and therefore is potentially a safety risk to the road users.

The images of the 2 Curves are shown in Figure 5, 6 and 7 below.

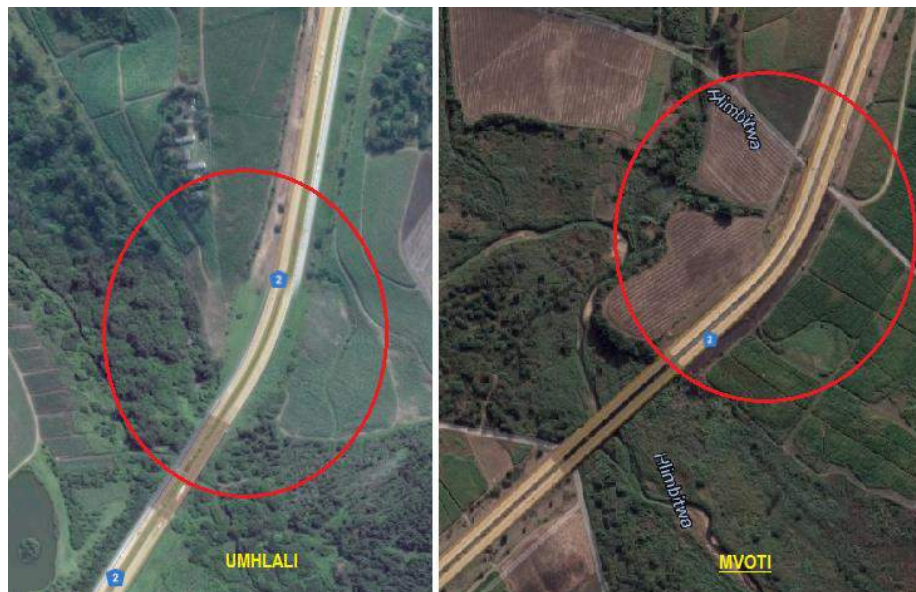


Figure 5. Aerial Photograph of Curves Umhlali and Mvoti



Figure 6. Umhlali River Bridge (South Facing)



Figure 7. Mvoti River Bridge (North facing)

From the investigations undertaken by the geometrics specialist it was discovered that the curve lengths of both curves were below 190m. According to the SANRAL Geometrics Guideline Manual a desired curve length on freeways should be 3 times the design speed. The existing curves were designed to a design speed of 100km/h. The curve length should have been to a minimum length of 300m. From the information discussed above it is evident that these curves can be regarded as kinks which is not ideal on a freeway. Hence it was agreed that the curves be redesigned according to acceptable standards as set out in Geometrics Guideline Manual.

The initial proposals for the safety improvements of both these curves were to realign the curves using circular curves at a design speed of 120km/h. The proposed solution required an excess of 200mm of asphalt to be placed on the bridge decks in order to accommodate the super elevation development. The super elevation development in the case of a circular is preferable to allow for two thirds of the superelevation development to occur at the tangents (i.e outside the circular curve) and one third of the development to occur within the curve (TRH 17, 1988). This development will occur at entry and exit of the curve. This was of concern as this development will have to occur on the bridge decks which ultimately would require an excess of 200 mm of asphalt in order to accommodate the superelevation development. This excess 200mm of asphalt will not only affect the structural integrity of the bridge but also the safety standards of the bridge parapets. The parapets have a vertical face of 140mm as shown in Figure 6 below.

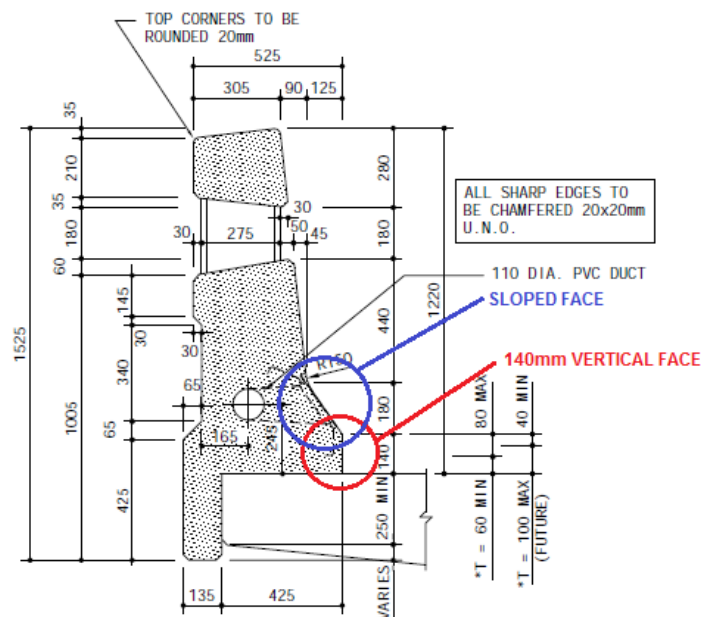


Figure 6. Parapet Details

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The purpose of the 140mm vertical face is to allow for a surface overlay on the bridge deck of up to 100mm, generally an asphalt wearing course. The sloped face is very important in that this face is sloped in order to minimize the head on impact of a vehicle onto the parapet. The angle of the slope allows the impacting tyre of the vehicle to climb up the parapet to a certain point to reduce the impact damage and then reflects the vehicle back onto the lane. In essence it has important safety advantages and serves an important role in the design of the parapet. Many studies have been undertaken in this regard and will not be discussed in this paper. Hence allowing the proposal an additional surfacing in excess of 200mm will definitely compromise the safety of the parapet design as this would reduce the length of the sloped surface which will result in an inadequate parapet design on site.

However in order to accept this proposal, it required a complete structural analysis to determine whether the current structure is capable of handling the additional dead load of asphalt, which would result in strengthening of the structure if possible or complete reconstruction. Over and above the dead load issue, all the parapets will have to be demolished and re-constructed. This proposal initiated a lot of additional costs to the project. SANRAL was of the opinion that another solution should be explored.

SANRAL advised the service provider to prepare another solution using transition curves as opposed to only using circular curves as initially proposed. The use of the transition curves will reduce the super elevation development length as the transition curve allows one to develop the super elevation completely within the curve (Geometric Planning (G2) Manual, 1984). This ultimately allows one to develop the superelevation within the curve and eliminates the problem of having an excess of 200m asphalt over the bridge decks. With the transition curve option, the overlay on the bridge decks was reduced to a maximum of 60mm which is acceptable and does not compromise the safety design of the parapets as well as does not impose any additional dead loads that the existing bridge cannot handle. This was possible as much less of the superelevation development occurs over the bridge.

Ponding was also of great concern on these curves as it was identified during the road safety audit and visual inspections. The actual locations of ponding were observed to be from the centre of the road to the inner side of the curves i.e. the lowest side of the road through a curve. According to the SANRAL Drainage Manual 6th Edition published in 2013 the flow depth during a 1:5 year storm should not exceed 6mm. In this case the minimum flow depth of 6mm was exceeded. In order to eliminate this problem it was agreed to increase the superelevation of the curves to 8%, and also to increase the tinning groove depth of the concrete pavement to reduce the depth of flow. Furthermore it was agreed to also increase the frequency of drainage chutes.

Increasing the superelevation will allow for the surface run-off to flow quicker off the road surface and the increase in the frequency of drainage chutes will aid in dissipating water of the road prism. In addition the increase in superelevation will also allow vehicles to take the curve with much more ease as this allows vehicles to track through the curve with less centripetal force being exerted.

The ideology behind increasing the tinning depth is to allow for more water to flow within the tinning groove before reaching the top of the driving surface. This method would reduce the risk of aquaplaning. The disadvantage with increasing the tinning groove depth is that this will increase the road noise on dry days. This was not of serious concern as the localities of both these curves are generally remote and away from residential dwellings. However, safety is of paramount importance and should be highest of priority when compared to the disadvantages of deep tinning grooves in concrete surfacing. Alternatively, sound barriers could have been considered had it been in an urban area. The final layout of the curve realignment for each of the areas is shown in Figure 7 and Figure 8 below.

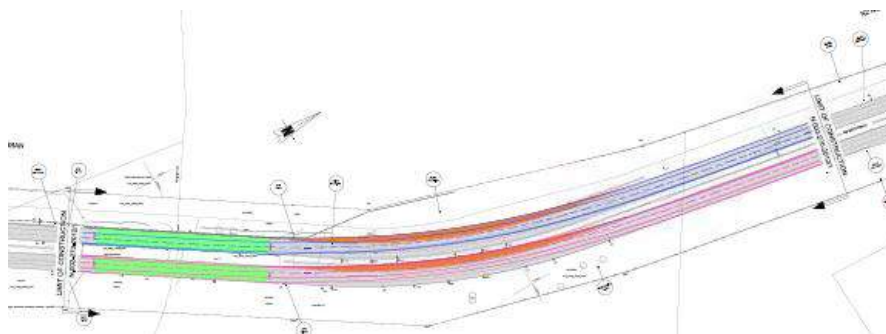


Figure 7. Final Layout of Realignment at Umhlali River Bridge

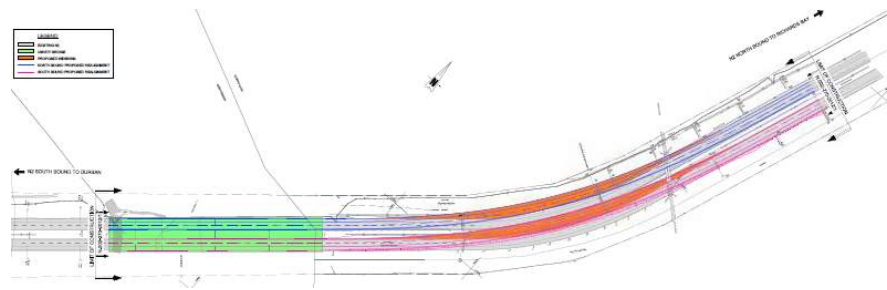


Figure 8. Final Layout of Realignment at Mvoti River Bridge

CONCLUSION

As highlighted throughout the paper, safety is of paramount importance to South Africa as a country and as well as the South African National Roads Agency. With the implementation of the decade of Action and South Africa aligning its outcomes and objectives to the Decade of Action, it can be seen from the fatality statistics shown in Figure 2 that South Africa has been successful since 2011 to 2015 in reducing the number of fatalities on their road network by 24%.

Improving the geometric alignment using the transition curve approach to meet the design requirements for a 120km/hr speed will also provide for a more safe, reliable, effective and efficient road. This obviously was the most suitable and economical solution for the safety improvements. In addition this proposal also reduced the construction duration because of the elimination of structural work to the bridges and the reconstruction of all parapets.

The N2 safety improvement as discussed in the contents of this paper merely showcases one of many safety improvement projects that are currently being undertaken in South Africa by SANRAL. SANRAL has currently adopted the policy that all improvement and new construction projects require a road safety audit to be undertaken. Furthermore, close interaction with the Road Traffic Inspectorate as well as the Road Traffic Management Corporation is ongoing as the Agency in the process of creating a live database that will allow one to identify high accident zones early and implement remedial measures at a quicker rate.

It should be noted that the improvements adopted at the Umhlali and Mvoti River Bridge were of a technical nature and that is all that can be done from an engineering point of view. However, many accidents occur due to human error, such as fatigue, intoxication etc. To target these issues SANRAL and South Africa have implemented awareness through media streams, sign boards along highways to make the motorists aware of the danger.

In conclusion South Africa has placed road safety at the top of its priority and will endeavour to achieve the goal of reducing the number of accidents by 50% by the year 2020 as set out in the Decade of Action Plan, and various roads authorities have placed road safety as a strategic objective.

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PAPER TITLE	Calibration of HDM-4 Road Deterioration Models for Rural Arterials in Victoria/Australia		
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ABSTRACT:

The highway Development Management (HDM-4) is a powerful pavement management tool capable of performing technical and economical appraisal and has been adopted by many countries. The purpose of this paper is to report the approach used in calibrating HDM-4 road deterioration models for both rutting and roughness to suit the conditions of Victoria's rural arterial network.

Time series pavement condition data (covering 3 to 5 years) were collected for 29 sections of rural arterials with different lengths covering different road classes (M, A, B and C), traffic volumes, climate zones and subgrade soil types. The sections were divided into 100 m length segments and only segments that had positive rate of deterioration were considered in the analysis. These segments were grouped into different combinations of road class, soil type and climatic conditions, resulting in 31 groups. The results indicate that, the rutting calibration factors are higher and roughness calibration factors are lower than the default values of one for class M and variable for classes A, B and C. The calibrated models have been validated by comparing the observed performance with the predicted performance obtained from using average group calibration factors and average class calibration factors. It was found that there is good to very good correlations between the predicted and observed values, when using average group calibration factors. This indicates that the criterion used for grouping of segments is successful. Also, there is good to very good correlation between the predicted and observed data, when using average class calibration factors.

Calibration of HDM-4 Road Deterioration Models for Rural Arterials in Victoria/Australia

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1 ABSTRACT

The highway Development Management (HDM-4) is a powerful pavement management tool capable of performing technical and economical appraisal and has been adopted by many countries. The purpose of this paper is to report the approach used in calibrating HDM-4 road deterioration models for both rutting and roughness to suit the conditions of Victoria's rural arterial network.

Time series pavement condition data (covering 3 to 5 years) were collected for 29 sections of rural arterials with different lengths covering different road classes (M, A, B and C), traffic volumes, climate zones and subgrade soil types. The sections were divided into 100 m length segments and only segments that had positive rate of deterioration were considered in the analysis. These segments were grouped into different combinations of road class, soil type and climatic conditions, resulting in 31 groups. The results indicate that, the rutting calibration factors are higher and roughness calibration factors are lower than the default values of one for class M and variable for classes A, B and C. The calibrated models have been validated by comparing the observed performance with the predicted performance obtained from using average group calibration factors and average class calibration factors. It was found that there is good to very good correlations between the predicted and observed values, when using average group calibration factors. This indicates that the criterion used for grouping of segments is successful. Also, there is good to very good correlation between the predicted and observed data, when using average class calibration factors.

2 INTRODUCTION

The highway network provides access and mobility to the industry and communities; therefore, it is one of the most important infrastructure elements and a major economic asset of any country. A more efficient network reflects better mobility service and better accessibility. The State of Victoria has the highest concentration of roads in Australia, and major activity centres are distributed all over the state. These centres are connected by high quality rural highways categorised into four main classes namely; M, A, B and C which vary by function, quality and level of service (Vicroads, 2013); a brief description of each is presented below. Network management requires reliable data and accurate performance models to use for predicting pavement condition over time. The latter is used with specific intervention criteria to perform gap analysis, develop short and long term maintenance programs and estimate the budgetary requirements.

- Class M: rural freeways; the main purpose of these roads are connecting Melbourne with other capital cities and major provincial centres. They have high standard of design and construction, and divided four lane roads with sealed shoulders.
- Class A: rural arterial highways; they have a similar role to M roads, but carry less traffic. Generally, they are undivided with two lanes with sealed shoulders.
- Class B: rural arterial highways; their main purpose is connecting the major regions which are not served by A roads and they are considered to be highly significant tourist roads. They are undivided two lane single carriageway roads.
- Class C: rural arterial highways; provide important links between Victoria's rural towns and primary transport network. They are generally two lanes undivided sealed roads with unsealed shoulders.

The highway Development Management (HDM-4) is a powerful pavement management tool capable of performing technical and economical appraisal. It has been used in more than 100 developed and developing countries that have different technological, climate and economic environments (Bennett and Paterson, 2000). It includes built-in road deterioration models that require calibration to local conditions for accurate predictions.

The purpose of this study is to calibrate HDM-4 road deterioration models for both rutting and roughness to suit the conditions of Victoria's rural arterial network. The study is focused on calibrating roughness and rutting road deterioration models, as they are considered the most important parameters for the

reasons highlighted below. Calibration of cracking models is not considered as results from a previous calibration study, (Toole et al, 2004) are considered sufficient and have been used in this study.

- Road surface roughness affects users' comfort and cost (travel time and delay), vehicle operating costs and network operation due to its effect on travel speed (Gillespie and Sayers, 1985, Foley, 1999, Loizos and Plati, 2002, Hunt and Bunker, 2004, and Moffatt, 2007, Loizos and Plati, 2008). Further, the interaction between surface roughness and heavy vehicles results in dynamic wheel loads that increase pavement damage (Paterson 1987, Gillespie et al, 1993, and Mikhail and Mamlouk, 1997). Roughness is accepted in Australia as the most relevant measure for long term functional behaviour of pavement network (Haas et al., 1994, Martin, 1996, Foley, 1999 and Moffatt, 2007).
- Rutting influences vehicle operation (vehicle tracking), users' safety (aquaplaning) and is usually associated with other distresses that lead to loss of pavement shape and integrity. These distresses include transverse shoving, depression, heaving and cracking (Foley, 1999, Morosuk et al, 2004, and Moffatt and Hassan, 2007).

For Victoria, calibration of these models has been done previously but was not supported by relevant road agency as the outputs did not match expected performance based on local experience and knowledge of network performance (Giummarra et al., 2007). Two HDM-4 calibration studies have been done for Victoria and reported in Toole et al. (2004) and Hoque et al. (2008). In these studies, the HDM-4 models for roughness, rutting and cracking have been calibrated using historical performance data. In Toole et al. (2004) study, 82 separate sections had been chosen from Victorian network covering different road classes, including class M, A, B and C, and pavement types. Hoque et al (2008) recalibrated the roughness and rutting models using the same sections, but after excluding the immediate effect of maintenance treatment using a new approach developed by Martin and Hoque (2006). From 82 sections only 55 sections showed positive rate of deterioration and had been used in the calibration exercise. Presented in Table 1 are the calibration factors for roughness and rutting models from both studies. They are applicable to classes M, A, B and C roads with granular pavement and surface treatment (spray seal).

In this study, a new approach for calibrating these models to Victoria's conditions is used. The approach includes the following:

- Development of a representative network of heavy freight routes in rural Victoria with a large sample size covering different road classes, traffic loading, subgrade soil types and climate conditions.
- Using roughness data from adjusted profile measurements so that same sections of road are actually compared over time. Also extracting corresponding rutting and cracking data of these sections from the database of relevant authority.
- Using only sections that have positive rate of deterioration for both roughness and rutting to remove the effect of maintenance.
- Using vehicle fleet composition and loading data that match actual usage of the road classes considered.

Table 1. Calibration results from previous studies

Road type	Pavement type	Traffic group	Climate group	Rutting Factors (K_{rst})		Roughness Factors (K_{gm})	
				Toole et al, 2004	Hoque et al, 2008	Toole et al, 2004	Hoque et al, 2008
A	STGB	Low	Dry	2.63	2.13	1.25	0.53
			Wet	3.00	2.13	0.30	0.26
		Medium	Temperate	1.38	0.66	0.30	0.28
B	STGB	Low	Dry	0.75	1.40	2.25	1.34
			Temperate	1.53	1.80	0.41	0.44
		Medium	Dry	3.35	3.58	0.95	0.84
			Temperate	3.00	1.58	0.30	0.19
C	STGB	Low	Temperate	2.37	2.82	0.54	0.33
			Wet	3.35	1.83	0.55	0.33
	STGB	Low	Dry	0.50	0.48	0.35	0.15
		Medium	Temperate	2.67	1.17	0.25	0.36
		Heavy	Temperate	2.17	1.72	0.42	0.39

1. Traffic group: low (AADT < 5000), medium (5000 < AADT < 15000), heavy (AADT > 15000)
2. Climate group: dry (TMI < 0), temperate (0 < TMI < 50), wet (TMI > 50)
3. STGB: surface treatment on granular base

It is believed that such approach would result in more accurate results. In previous studies a lower sample size was used and the condition data was extracted from database of relevant road agency. This time series data may not represent the actual sections over time as it was extracted from the database of relevant road agency. It is the result of processing raw longitudinal profiles for different years but without the application of proper adjustment to match the actual start and end of each section and ensure that the same sections are being compared over time.

The study also involved calibrating roughness deterioration model using roughness data extracted from the database of relevant road agency. They were then compared with those developed using roughness data from adjusted surface profiles. In addition, the study covers comparison between the study calibration factors and calibration factors obtained in previous study (Hoque et al., 2008).

3 DATA COLLECTION AND PREPARATION

This section describes the required data parameters for the calibration study, which include pavement condition data, traffic data, general network information, and climate data. It also covers data preparation such as adjustment of longitudinal profile data, matching rutting and cracking data to roughness data chainages, determining subgrade soil type and extracting climate data. That is in addition to data cleaning and filtering and grouping of pavement segments for calibration.

3.1 Network Selection

The availability of relevant data is a major criterion for the network selection. For this study (29) freeways and rural arterials with available historical condition data were selected to represent Victoria's rural arterial network conditions. These highways are distributed all over the State with variable lengths ranging from 10 to 128 km. The selected road network consists of 7, 10, 7 and 5 sections from road classes M, A, B and C, with total lengths of 170.6, 710.8, 571.4, 265.6 km respectively. The total length of the selected network is approximately 1718 km. These roads are selected to cover approximately all climate zones and soil types in rural Victoria.

3.2 Data Collection

HDM-4 is a comprehensive and complex system and requires data assembled in an efficient manner. Data quality and correct interpretation is considered as one of the important key parameters in the application of HDM-4, which is necessary for desired reliability of the results. For this study the data required can be described in the following categories or sets:

- General network information: road type, pavement type, pavement width, shoulder type and width, road side, topography and sub-grade soil type.
- Road condition data: Roughness, rutting, cracking, strength and drainage quality
- Climate data: TMI (Thornthwaite moisture index), rainfall and temperature
- Historical data: pavement age, maintenance history
- Traffic data: AADT and vehicle fleet composition
- Other data relating to HDM-4 input including calibration factors, maintenance standards, and default coefficient values

All the above data were extracted from relevant road authority databases, except for the following:

- Roughness data was determined from adjusted raw longitudinal profile data
- Subgrade soil type was identified from Victoria's expansive soils map
- Climate data (TMI) was extracted from the climate tool as described later
- Some calibration factors, maintenance standards and default coefficient values were obtained from a previous study (Toole et al, 2004) and HDM-4 documentation.

3.3 Data Preparation

The required data for the analysis is prepared as described in the following steps.

3.3.1 Roughness

Roughness is a pavement condition parameter and is defined as a measure of irregularities in the pavement surface (Morosiuk et al, 2004). It affects user's comfort and perception of ride quality, user's cost and vehicle operating cost and used to predict future network condition (Foley, 1999, Hunt and Bunker, 2004 and Moffat, 2007). Roughness is reported by two units in Victoria, namely, NAASRA Roughness Meter (NRM) in counts/km and the International Roughness Index (IRI) in m/km (Jameson and Shackleton, 2009).

The data is used by relevant road agency to trigger investigation into pavement rehabilitation, using certain intervention levels, together with other performance measures or criteria.

For the sample sections, raw longitudinal profile data were obtained for a number of years, between 1998 and 2010. Each section had available raw longitudinal profile data for 3 or 5 years which were found to vary in their start and end chainages between the different years. This variation may affect the results when comparing the profile characteristic of the same road over time. Therefore, it was essential to adjust the profile data for the different years before calculating the roughness data to ensure that the same section of the road is being compared over time.

The adjustment is done by selecting one year as a reference (usually latest available year) and making the required shifting by adjusting the sample interval of the profile data. An in-house Excel-based tool developed by Evans and Arulrajah (2011) is used for this purpose. The ProVal (Profile Viewing and Analysis) software (Proval, 2013) was used for viewing the profile data for the different years and deciding on the required shifts. This software has been used in many studies such as (Chang et al, 2009, Lee and Chou, 2010 and Islam et al. 2014). After ensuring that all profiles are matched, the roughness (IRI) values for both left and right wheel paths are determined using ProVal software for each 100 m segment. This is the recommended reporting interval in Australia and matches the reporting interval in relevant road authority database for rutting and cracking data.

3.3.2 Rutting and Cracking

Rutting and cracking data of the sample sections are extracted for each 100 m segment from the relevant database, provided by the road agency responsible for managing these roads. This database also includes corresponding IRI values at different chainages. To match the rutting and cracking data set with the adjusted roughness data, roughness profile from the database for each sample section is matched with its adjusted roughness profile for each year. Once the two profiles are matched, rutting and cracking values corresponding to the correct chainages of the adjusted roughness data are selected.

3.3.3 Subgrade Soil Type

The subgrade provides support to pavement layers and its support depends on the soil type, material density and moisture content during construction and in-service (Vicroads, 2003, Mann, 2003 and sharp, 2009). Over half of the roads in Victoria are built on expansive soils with different degrees of swell potential (Vicroads, 1995). The Majority of Victoria's rural highway network includes thin granular flexible pavements built on expansive subgrade soils with different swelling potential levels, and located in different climatic zones (Hassan et al., 2006). The integrated colour coded map of expansive soils in Victoria produced by Mann, (2003) was used to establish the soil type underlying each section. The different colours represent different soil types with different swelling potential levels. Reactivity of subgrade soils for the selected roads were determined by plotting the selected road sections on Victoria's soils map using AutoCAD for precise location of sections using their start and end chainages. The sample road sections pass through seven different soil types that have different levels of swell potential from non to high expansive.

3.3.4 Climate Data

Climate is one of the main factors affecting pavement performance. The variation in the moisture content which is caused by dry and wet periods has a significant effect on the pavement life (Zue et al, 2007). In expansive soils, seasonal moisture variation changes the volume of expansive soil and makes it become continually active (Mann, 2003). In this study TMI has been used as an indicator of the climatic condition. TMI is a number that indicates the relative wetness or dryness of a particular soil climate system (Thorntwaite, 1948). Positive TMI value indicates wet area and negative value indicates dry area.

In this study TMI values for the sample sections are determined using the climate extraction tool developed by Byrne and Aguiar (2010). The tool is an Excel database that requires inputting a GPS location in order to access a wide range of historical climate data between 1960 and 2007. In addition, this tool supplies a range of simulated climate data from 2008 to 2099. This tool has been also used by Sen (2012) for determining TMI values. The TMI for each 100 m segment over the different years are determined first then average TMI value for each segment is determined as a mean value for the period over which condition data is available i.e. analysis period. HDM-4 allows inputting one climate zone per section. Furthermore, the data of mean daily temperature, rainfall and maximum and minimum daily temperature was extracted from Bureau of Meteorology website. This data is required as input in HDM-4.

3.3.5 Traffic Data

The average annual daily traffic (AADT) data for the selected sections was extracted from the relevant database for a number of years. The latter includes the years for which the condition data is available, where possible. The average growth factors for the selected sections were determined from all available data and have been used for determining traffic volumes of the missing years in the analysis period.

Typical vehicle fleet for each road class was also determined from the traffic data supplied by the relevant road agency. This data was collected using traffic counters at some locations of the sections in the sample. It should be noted that HDM-4 does not allow input of more than one vehicle fleet for all sections in a network. Hence, a representative fleet composition for each road class was developed from all available data using the following approach.

1. For each site the proportion of each vehicle class shown in Table 2 is determined as the average value over available years.
2. The average of the proportions of that vehicle class over all sites in relevant road class is then determined and used as a representative value for the road class.

The vehicle fleets consist of 15 to 20 vehicle types (Toole et al, 2004) as shown in Table 2. The Equivalent Standard Axles (ESAs) for each vehicle type was determined by assuming 25% of each vehicle type operate at the tare weight i.e. empty and 75% of them are assumed fully loaded with legal loads under General Mass Limits scheme (NHVR, 2014).

Table 2. Vehicle fleet compositions

Vehicle type	Austroad Class	Axle/Vehicle	Tare Load, tonne	Fully loaded, tonne	ESAs/Vehicle	Percentage			
						M	A	B	C
Courtesy of ARRB Group									
Light Vehicles	1	2				82.05%	81.6%	86.2%	88.65%
Light Rigid	3	2	1.60	3.75	0.01	2.13%	2.16%	2.40%	1.84%
Medium Rigid	3	2	3.20	10.40	0.53	2.13%	2.15%	2.39%	1.84%
Heavy Rigid	4	3	12.0	19.00	2.72	0.72%	0.79%	0.69%	0.56%
Heavy Bus	4	3	9.00	22.50	1.81	0.72%	0.79%	0.69%	0.56%
Artic 4 Axle	7	4	11.0	31.50	3.96	1.13%	1.02%	0.82%	0.67%
Artic 5 Axle	8	5	13.0	39.00	4.40	1.21%	1.05%	0.73%	0.65%
Artic 6 Axle	9	6	15.0	42.50	3.89	6.15%	6.06%	3.80%	3.17%
Rigid +5 Ax Dog	10	8	19.0	59.00	5.44	1.16%	1.46%	0.76%	0.68%
B-Double	10	9	22.0	62.50	4.93	1.16%	1.46%	0.76%	0.69%
TS + 5 Ax Dog	10	9	21.0	64.00	5.90	1.16%	1.46%	0.76%	0.69%
A-Double	11	11	31.0	79.00	6.50	0.08%			
B Triple	11	12	34.0	82.50	5.99	0.08%			
AB Combination	11	14	39.0	99.00	7.54	0.08%			
A-Triple	12	16	44.0	115.5	9.10	0.02%			
Double B-Double	12	17	47.0	119.0	8.59	0.02%			

3.4 Data Cleaning and Filtering

Originally 1705, 7108, 5713 and 2655, 100 m segments, for class M, A, B and C, respectively with their data were available. These numbers were reduced through cleaning and filtering processes described below.

3.4.1 Data Cleaning

The cleaning process involves removing all 100 m segments that have anomalous roughness values from all relevant years, as a result of the different events during the automated condition survey. Such events include bridge joints, lane change, stopping at intersections and rail crossings, which are reported with condition data in the database together with their locations (Hassan, 2011). Their locations are further confirmed by viewing the longitudinal profile of each road section and observing the chainages of unusual spikes. The remaining number of segments after this cleaning process is 1624, 6950, 5679 and 2650 segments for class M, A, B and C, respectively.

3.4.2 Data Filtering

The calibration approach of HDM-4 road deterioration model applied herein is based on using the segments with continuous deterioration i.e. with positive rate of deterioration, where the effects of maintenance and rehabilitation are removed. This has been achieved by determining the rate of deterioration over time for roughness and rutting of each segment using the Linear Rate of Progression (LRP) Excel-based tool developed by Martin and Hoque (2006). In this tool, the deterioration rate of a segment is determined from at least three or more valid time series data points. This tool has been used in other studies such as (Hoque et al, 2008, and Sen, 2012). All segments that show negative or zero rate of deterioration are excluded. This means that segments subjected to the periodic maintenance and rehabilitation activities are excluded. However, the effects of routine maintenance activities, such as crack sealing, are already taken into account in the progression rates of roughness and rutting. The remaining numbers of 100 m segments with positive rate of deterioration for both roughness and rutting are 461, 1453, 1211 and 572 (46, 145, 121 and 57 km) for class M, A, B and C, respectively.

3.5 Grouping of Segments

The segments with positive rates of deterioration are grouped according to four soil types by swell potential level (high, moderate, low and non-expansive) and three climate categories. The climate categories (semi-arid, subhumid and humid) are based on the HDM-4 climate zones classification (Bennett and Paterson, 2000) shown below. These grouping criteria resulted in (6, 10, 8 and 7) groups for class M, A, B and C respectively as shown in Table 3.

- Semi-arid: TMI range = -60 to -21
- Subhumid: TMI range = -20 to 19
- Humid: TMI range = 20 to 100

The calibration process of HDM-4 road deterioration models is repetitive and time consuming therefore; the 461, 1453, 1211 and 572 segments are merged into 45, 110, 73 and 43 new segments for class M, A, B, and C respectively, ranging between 0.2 and 3.9 km as shown in Table 3. This is based on a set of criteria including starting analysis year, road name, pavement width, pavement strength, drainage condition, terrain and traffic volume. The conditions and other parameters for these new segments are reported as a mean value of the corresponding combined 100 m segments.

4 CALIBRATION OF HDM-4 ROAD DETERIORATION MODELS

The calibration process was focussed on rutting and roughness road deterioration models. According to the structure of HDM-4 models, there is an interaction between the models. This means that the predicted roughness is influenced by the predicted rutting; therefore, the rutting model was calibrated before the roughness model.

The calibration process contained a number of preparatory, analysis and interpretation steps, which comprised establishing an organised filing structure. This process generates a large number of files that are all related to each other and dependent on one another, which are required for the calibration process implementation.

The following assumptions have been made for conducting the calibration exercise:

- The performance data represents pavement condition some year after the construction or rehabilitation as the performance data for the whole life of pavement is not available
- Only segments with positive deterioration rates are included to remove the effect of periodic maintenance and rehabilitation.
- Deterioration of each segment is modelled for the same period over which its condition data is available. The segments have different starting condition years namely; 1998, 1999, 2001, 2004, 2005 or 2006 and have condition data for 3 – 5 years (between start year and 2010). Therefore, they have different analysis periods, which range between 4 and 12 years.
- The rutting structural deterioration calibration factor (K_{rst}) and roughness environmental component calibration factor (K_{gm}) are calibrated. The roughness structural component K_{gs} is set equal to K_{gm} to reduce the gap between predicted and measured roughness (Hoque et al, 2008).
- All calibration factors other than the ones being calibrated in this study are assumed to be at the default values recommended in HDM documents except for all structural cracking initiation (K_{cia}), all structural cracking progression (K_{cpa}), and ravelling progression ($K_{vp}=0.5$), which are taken from a previous study reported in (Toole et al, 2004) and shown in Table 4.
- All segments are subjected to routine maintenance to take into account the routine maintenance practice during the analysis period (Toole et al, 2004). That includes patching (> 1 pothole/km), edge repairs (> 1 m²/km) and removal and patching of cracked areas (> 2 percent wide cracking).

Table 3. Number of segments and relevant range of parameters for all groups

Road Class	Climate/Soil Combination	No. of Combined Segments, (length, km)	Initial Rgh IRI	Initial Rut Depth mm	Initial Cracks %	SNP	AADT	TMI	GF%
M	Subhumid High Expansive	7 (0.2 – 1.2)	1.23-3.05	3-8	0-10	4.93-5.33	6524-10361	7-15	5.48
	Subhumid Moderately Expansive	7 (0.2 – 1.0)	1.37-1.97	2-3	0	4.63-4.76	2409-3413	(-7)-1	4.28
	Subhumid Low Expansive	11 (0.9 – 1.2)	0.95-2.0	3-7	0-25	4.96-5.90	5898-9740	9-15	5.48-5.71
	Subhumid Non Expansive	10 (0.8 – 1.1)	1.24-1.99	2-4	0-22	4.68-5.61	2477-4880	(-8)-4	3.51-4.28
	Humid Moderately Expansive	3 (0.6 – 1.0)	1.17-1.34	5-6	2-16	5.56-5.57	8889	83	4.35-4.97
	Humid Non Expansive	7 (0.7 – 1.1)	1.33-1.83	4-8	0-4	5.23-5.70	8683-9664	59-83	4.97
A	Semi-arid High Expansive	5 (0.3 – 1.3)	1.34 - 2.32	2 - 6	0 - 18	4.1 - 4.3	3690 - 4672	(-22) - (-21)	4.54
	Semi-arid Moderately Expansive	10 (0.4 – 2.5)	1.37 - 2.68	2 - 3	0 - 10	4.0 - 4.3	1808 - 4672	(-23) - (-21)	4.54
	Semi-arid Non Expansive	9 (0.9 – 3.2)	1.58 - 2.73	2 - 7	0 - 10	3.8 - 4.2	1902 - 2424	(-37) - (-21)	2.5 - 4.54
	Subhumid High Expansive	25 (0.4 – 1.9)	1.11 - 2.69	2 - 5	0 - 8	3.0 - 4.2	306 - 4888	(-20) - 18	4.54 - 6.90
	Subhumid Moderately Expansive	19 (0.5 – 1.9)	1.33 - 2.59	1 - 5	0 - 9	3.2 - 4.2	306 - 4432	(-20) - 19	4.54 - 6.90
	Subhumid Low Expansive	8 (0.2 – 1.4)	1.35 - 2.34	2 - 7	0 - 1	3.5 - 4.1	1178 - 3588	(-20) - 17	4.54 - 5.68
	Subhumid Non Expansive	13 (0.3 – 1.9)	1.49 - 3.31	3 - 11	0 - 4	2.9 - 4.0	306 - 6918	(-14) - 17	4.64 - 6.90
	Humid Moderately Expansive	6 (1.4 – 1.5)	0.93 - 1.64	3 - 4	0	3.7 - 3.9	2216 - 3058	20 - 23	4.88
	Humid Low Expansive	3 (0.4 – 1.3)	1.71 – 3.22	3 - 6	0 - 2	3.6 - 3.7	1332 - 1896	41 - 45	3.72
	Humid Non Expansive	12 (0.7 – 1.7)	1.47 – 2.97	3 - 5	0 - 2	3.2 - 3.7	866 - 2008	22 - 50	3.72 – 5.31
B	Semi-arid High Expansive	13 (0.3 – 2.6)	1.31 – 3.2	2 - 5	0 - 7	2.7 – 3.4	146 - 3160	(-24) - (-21)	2.49 – 5.08
	Semi-arid Moderately Expansive	4 (0.8 – 2.5)	1.52 – 1.96	2 - 5	0 - 7	3.6 – 3.8	3160 - 4164	(-23) - (-21)	2.49
	Semi-arid Low Expansive	5 (0.7 – 1.7)	1.97 - 2.51	2 - 4	0 - 3	3.3 – 3.6	736 - 942	(-26) - (-24)	3.75
	Subhumid High Expansive	23 (0.5 – 3.9)	1.55 – 3.14	2 - 6	0 - 11	3.0 – 3.8	376 - 3612	(-20) - 3	3.30 – 5.08
	Subhumid Moderately Expansive	4 (0.3 – 2.4)	1.22 - 3.45	2 - 4	0 - 1	3.0 - 3.6	566 - 1838	(-12) - 3	3.30 – 5.08
	Subhumid Low Expansive	8 (0.5 – 2.9)	1.47 - 2.88	2 - 6	0 - 7	3.1 - 4.0	594 - 5018	(-11) - 19	3.30 – 5.08
	Subhumid Non Expansive	4 (0.2 – 1.9)	1.61 – 2.46	3 - 6	0 - 5	3.2 - 3.9	594 - 5018	2 - 19	3.30 – 4.54
	Humid Non Expansive	12 (0.7 – 2.6)	1.57 - 3.21	2 - 7	0 - 5	2.5 - 4.0	206 - 6594	20 - 96	3.87 – 4.54
C	Semi-arid High Expansive	6 (0.2 – 1.1)	1.67 – 3.13	2 - 3	0 - 12	3 – 3.6	562 - 2180	(-24) - (-21)	3.66
	Subhumid High Expansive	12 (0.4 – 3)	1.78 – 2.81	2 - 5	0 - 10	2.9 – 3.4	346 - 2458	(-20) - 15	3.66 – 4.78
	Subhumid Moderately Expansive	6 (0.3 – 1.4)	1.62 – 3.32	2 - 7	0 - 4	2.7 – 3.4	1294 - 5376	7 - 15	2.67
	Subhumid Low Expansive	8 (0.5 – 3)	1.79 – 2.85	2 - 5	0 - 1	2.8 – 3.2	340 - 760	(-20) - (-1)	3.66 – 4.53
	Subhumid None Expansive	6 (0.4 – 1.3)	1.56 – 2.66	2 - 4	0 - 2	2.9 - 3.3	806 - 1532	6 - 19	2.67 – 4.78
	Humid Moderately Expansive	4 (0.5 – 1.3)	2.08 – 3.37	2 - 4	0 - 3	2.9 - 3.3	2150 - 2956	30 - 45	5.34
	Humid Non Expansive	1 1.1	2.47	3	0	3.2	2150	48	5.34

The methodology used for calibrating the road deterioration models to local conditions is to determine the calibration factor for each individual road segment in a group and then averaging the factors of all segments in the group to determine the calibration factors for the group, and averaging factors of all groups to determine the class calibration factor. The calibration process has been done by creating a case study in HDM-4 including one network covering all segments for classes A and B with one vehicle fleet and two networks for classes M and C with one fleet for each, one for the adjusted roughness data (Adj) and the other for the roughness data from relevant database (Db). The process involved the following steps:

1. Estimate the actual progression rates of rutting and roughness for all segments, from actual (observed) condition data.
2. Identify the analysis period for each segment and its pavement condition at the start of the analysis period. Due to the variation in starting years and analysis periods, 3 to 6 projects were created.
3. Create HDM-4 project analysis case study and input all parameters that are required to run its models including trial calibration factors for rutting or roughness.
4. Assign a typical routine maintenance standard to reflect the normal routine maintenance practice as a single alternative.
5. Run HDM-4 models to predict pavement condition annually up to and beyond the analysis period and compare with the actual condition trend.
6. Repeat step five after adjusting the calibration factors until the predicted performance trends match closely the observed performance trends, see examples in Figure 1.

Table 4. Cracking calibration factors from previous study (Toole et al, 2004)

Class	Subgrade soil	K_{cia}	K_{cpa}
M	Stable	1.16	0.30
A	Stable	0.87	0.29
	Unstable	1.10	0.30
B	Stable	0.99	0.36
	Unstable	1.00	0.53
C	Stable	0.80	0.46
	Unstable	0.60	0.30

1. Stable = non expansive soil
2. Unstable = expansive soil

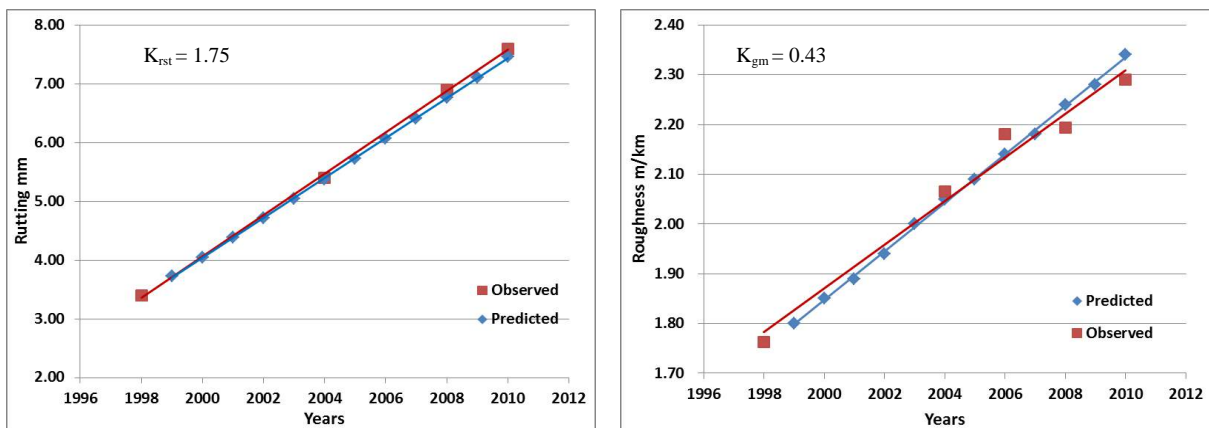


Figure 1. Example of performance trends of rutting and roughness for both predicted and observed data.

First the rutting progression model is calibrated from the relevant historical data and the calibration factors for the rutting structural deterioration (K_{rst}) are established. Then the roughness deterioration model is calibrated from the historical data and its calibration Factors (K_{gm}) are obtained. The ranges of calibration factors for each group are presented in Table 5, together with the average calibration factors for each group and class. Furthermore, Table 5 shows the calibration factors from a previous study (Hoque et al, 2008). In the previous study different criteria were used for grouping the segments, which prevent direct comparison. Therefore, the comparison had been done just for the closest groups.

Table 5. Calibration results for all road classes

Road Class	Climate/Soil Combination	Current study			Previous study closest groups		Average for class	
		K _{rst} (average)	K _{gm} Adj (average)	K _{gm} Db (average)	K _{rst}	K _{gm}	K _{rst}	K _{gm}
M	Subhumid High Expansive	1.2 - 2.85 (2.03)	0.1 - 0.67 (0.49)	0.01 - 0.45 (0.21)	1.17	0.36	2.63	0.29
	Subhumid Moderately Expansive	1.05 - 2.65 (2.07)	0.19 - 0.83 (0.59)	0.05 - 0.60 (0.22)	0.48	0.15		
	Subhumid Low Expansive	1.25 - 2.8 (2.05)	0.01 - 0.52 (0.27)	0.01 - 0.60 (0.22)	1.17	0.36		
	Subhumid Non Expansive	1.1 - 3.51 (2.10)	0.01 - 0.72 (0.19)	0.01 - 0.60 (0.09)	0.48	0.15		
	Humid Moderately Expansive	2.34 - 5.45 (3.85)	0.03 - 0.50 (0.16)	0.01 - 0.40 (0.14)				
	Humid Non Expansive	3.5 - 4.0 (3.7)	0.01 - 0.03 (0.02)	0.01				
A	Semi-arid High Expansive	1.0 - 1.64 (1.38)	0.34 - 0.93 (0.64)		2.13	0.53	1.33	0.48
	Semi-arid Moderately Expansive	0.85 - 2.10 (1.36)	0.41 - 1.70 (1.11)		2.13	0.53		
	Semi-arid Non Expansive	0.66 - 2.20 (1.03)	0.27 - 1.07 (0.61)		2.13	0.53		
	Subhumid High Expansive	0.58 - 3.68 (1.62)	0.13 - 1.25 (0.55)					
	Subhumid Moderately Expansive	0.45 - 3.06 (1.38)	0.01 - 0.93 (0.30)					
	Subhumid Low Expansive	0.65 - 1.87 (1.22)	0.02 - 0.75 (0.41)					
	Subhumid Non Expansive	0.50 - 3.87 (1.51)	0.01 - 0.44 (0.23)					
	Humid Moderately Expansive	0.91 - 2.08 (1.27)	0.05 - 0.20 (0.10)					
	Humid Low Expansive	0.83 - 1.70 (1.23)	0.26 - 0.51 (0.36)					
	Humid Non Expansive	0.68 - 2.71 (1.34)	0.03 - 1.28 (0.48)					
B	Semi-arid High Expansive	0.44 - 4.48 (1.55)	0.35 - 1.95 (0.98)		1.40	1.34	1.71	0.56
	Semi-arid Moderately Expansive	0.85 - 3.27 (1.67)	0.02 - 0.97 (0.46)		1.40	1.34		
	Semi-arid Low Expansive	0.55 - 2.76 (1.46)	0.52 - 1.45 (1.02)		1.40	1.34		
	Subhumid High Expansive	0.55 - 3.03 (1.44)	0.03 - 1.55 (0.62)					
	Subhumid Moderately Expansive	0.42 - 3.87 (2.15)	0.01 - 0.99 (0.37)					
	Subhumid Low Expansive	0.45 - 4.11 (1.62)	0.01 - 0.48 (0.26)					
	Subhumid Non Expansive	0.57 - 4.45 (2.04)	0.15 - 0.37 (0.23)		1.8	0.44		
Humid Non Expansive	0.58 - 3.16 (1.74)	0.30 - 1.11 (0.58)						
C	Semi-arid High Expansive	0.90 - 1.72 (1.34)	0.40 - 3.04 (1.41)	0.27 - 3.30 (1.66)			1.19	0.61
	Subhumid High Expansive	0.70 - 3.20 (1.57)	0.71 - 1.80 (1.17)	0.68 - 1.90 (1.17)				
	Subhumid Moderately Expansive	0.49 - 1.68 (0.98)	0.13 - 0.37 (0.22)	0.05 - 0.25 (0.11)	2.82	0.33		
	Subhumid Low Expansive	0.64 - 2.14 (1.16)	0.22 - 1.18 (0.64)	0.08 - 1.10 (0.50)				
	Subhumid None Expansive	0.69 - 3.13 (1.64)	0.05 - 0.85 (0.37)	0.01 - 0.95 (0.20)	2.82	0.33		
	Humid Moderately Expansive	0.89 - 1.54 (1.08)	0.26 - 0.38 (0.30)	0.35 - 0.53 (0.41)	2.82	0.33		
	Humid Non Expansive	0.58	0.14	0.20				

The closest groups from the previous study are low traffic with dry climate and medium traffic with temperate climate groups for class M, low traffic with dry climate group for class A, low traffic with dry climate group and low traffic with temperate climate groups for class B and low traffic with temperate climate group for class C as shown in Table 1. These groups are close to the study groups shown in Table 5.

Table 5 shows that the rutting calibration factor (K_{rst}) values obtained from this study are greater than the default value (1.0) for class M and variable for the class A, B and C. Also the roughness calibration factor (K_{gm}) values obtained from this study are lower than the default value (1.0) for class M and variable for class A, B and C. Furthermore, the (K_{gm}) values obtained from the adjusted roughness data are higher than those obtained from the database roughness data for class M and variable for class C. In addition, for the closest groups, the (K_{rst} and K_{gm}) values obtained from this study are higher than those obtained from the previous study for some groups and lower for the others.

5 VALIDATION OF AVERAGE CALIBRATION FACTORS

The following sections cover the validation of average group calibration factors and average class calibration factors using the correlation between the predicted and observed data.

5.1 Validation for Average Group Calibration Factors

The average calibration factor for the rutting and roughness of each group has been used as the calibration factor for all road segments within the group. Then HDM-4 was run to predict rutting and roughness over the analysis period. To validate the results, the predicted rutting and roughness data is plotted against the corresponding actual performance data and the coefficient of determination (R^2) of the best fit line is determined. For all groups it was found that there is good to very good correlation between the two sets of data with R^2 ranging between (0.64 and 0.98) for rutting and (0.81 and 0.99) for roughness. Figures 2, is an example that shows the correlation between the observed and predicted data. These results indicate that the criterion used for grouping the segments is successful.

5.2 Validation for Average Class Calibration Factors

The average class calibration factor for both rutting and roughness of each class was used as calibration factor for all segments within the class. As described above the HDM-4 was run and the predicted data was plotted against the corresponding actual performance data for validation. For all classes it was found that there is good to very good correlation between the two sets of data with R^2 ranging between (0.68 and 0.87) for rutting and (0.85 and 0.96) for roughness.

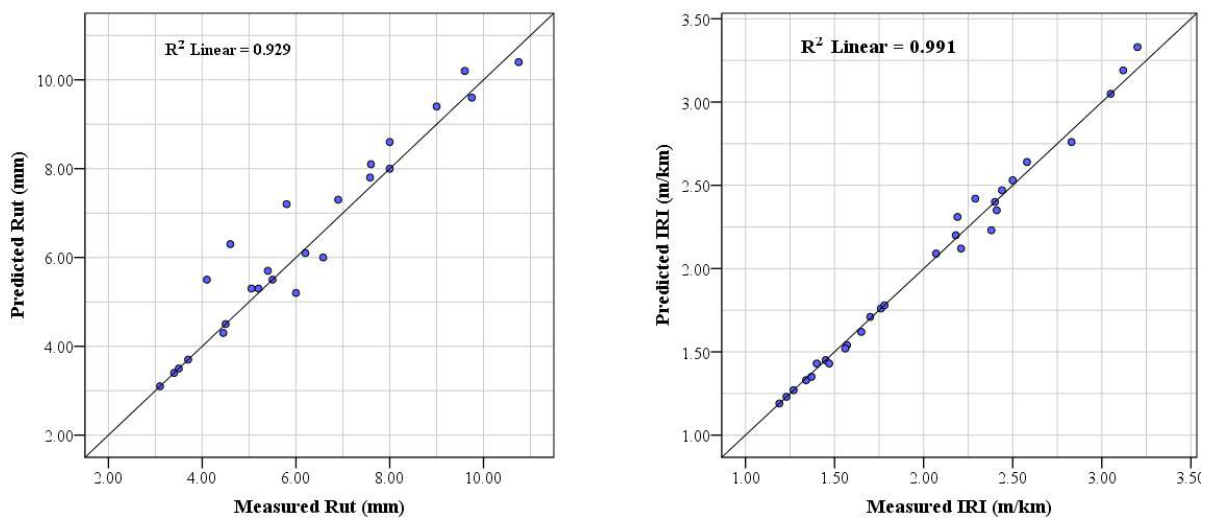


Figure 2. Correlation between observed and predicted rutting and roughness using average calibration factors for subhumid high expansive group in class M

6 CONCLUSIONS

This paper describes the process of calibrating HDM-4 road deterioration models for rutting and roughness using a large sample size to well represent Victoria's rural arterial conditions and roughness data

from the adjusted longitudinal profile data to make sure that the same sections are being compared over time. Only segments with positive rate of deterioration were used. These segments were grouped according to the subgrade soil type and climate condition into (6, 10, 8 and 7) groups for classes M, A, B and C, respectively. The HDM-4 road deterioration models were calibrated using the historical data to match the underlying rates of rutting and roughness progression. The calibration factors for both roughness and rutting progression were determined for each segment within each group of each class, and then average calibration factors for the groups in a class and for each class were determined. These factors were then used for each segment within the group in HDM-4 to predict rutting and roughness values over time over relevant analysis periods. The predictions have been validated by comparing their trends with the actual ones over the same period. The main findings are:

- The (K_{rst}) values are more than the default value of one for class M and are variable for class A, B and C
- The (K_{gm}) values for both adjusted roughness data and roughness data extracted from relevant database are less than the default value of one for class M and are variable for class A, B and C
- The (K_{gm}) values obtained from the adjusted roughness data are higher than those obtained from the database roughness data for class M but are variable for class C.
- The (K_{rst} and K_{gm}) values obtained in a previous study for the closest groups are higher than those obtained from this study for some groups and lower for others
- The criterion used for grouping the segments is successful as there is good to very good correlation between the predicted and observed data when using the average group calibration factors
- There is good to very good correlation between the predicted and observed data, when using the average class calibration factors.

When more time series condition data becomes available, the calibration exercise should be repeated to cover a longer study period and cover sections within the latter stages of the gradual deterioration phase. In addition to calibrating the deterioration models of cracking and potholing, when reliable data becomes available.

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A Proactive and Systematic Approach to Investing in Road Safety Shows Measurable Results – A Case Study of Road Upgrade Project in Samar, The Philippines

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1 INTRODUCTION

The US\$ 433.914 million Compact grant (Compact) from the Millennium Challenge Corporation (MCC) of the US Government was signed with the Philippine Government on September 23, 2010. The Millennium Challenge Account-Philippines (MCA-P) was thus created, and served thereafter as the Accountable Entity for the implementation of the grant.

The provinces of Samar and Eastern Samar, part of the Eastern Visayas Region, have long had some of the highest incidences of poverty in the Philippines. Aiming to improve living conditions and to provide better access to economic opportunities in the area, MCC granted \$214.44 million from the Philippines Compact for the reconstruction/rehabilitation of the 222-km Wright-Taft-Borong-an-Guiuan Road, which links the two provinces with the rest of the country.

The project, officially known as the Secondary National Road Development Project (SNRDP) of the Department of Public Works and Highways (DPWH), included the road upgrades and roadway rehabilitation activities including construction/reconstruction of paved or gravel shoulders, drainage and road safety improvements, resurfacing (overlay) and full-depth reconstruction, bridge replacement, rehabilitation and seismic retrofitting, road slip/landslide engineering solutions such as retaining walls, drainage improvements and other feasible and cost effective improvements including a range of Environmental and Social Mitigation (ESM) activities.

In addition, the project incorporated enhanced safety measures including paved shoulders, construction of sidewalks and curbs in areas of high pedestrian activity, improved gateway treatments to indicate where lower speeds are required, and increased use of road narrowing, median islands, and traffic humps to slow traffic speeds. The road segment passes through 14 municipalities and one city, and is the main passage between provinces. Improving the infamously rundown road has proved to not only help lower transport costs and travel time, but to also open up the area to new possibilities and new markets (MCA-P 2016) Road construction was substantially completed, and the road opened for traffic, on May 25, 2016.

SNRDP was implemented in four major contract packages (CP): (see Figure 1).

- CP 1- 16.3km from Buray to Tinani in Paranas, Samar
- CP 2- 63.78 km from Tinani, Paranas, Samar to San Julian, Eastern Samar
- CP 3- 64.58 km from San Julian to Balangkayan in Eastern Samar
- CP 4- 77.51 km from Llorente to Guiuan, Eastern Samar

This paper focuses on the CP1 pre and post construction road safety assessments and findings.



Figure 1 – Project Site

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Road Safety in the Philippines Context: In 2003, the population of the Philippines was around 94 Million (est.), with a 2.2% annual growth rate. The country has a land area of 300,000 square kilometers, a road length of 270, 000 km, and a total registered vehicle count of approximately 7,463,393 (2012) with a 4.4% annual increase rate. The Total Drivers Licenses Issued in the country was 13 Million (Lantin, 2013).

According to WHO data published in May 2014, Road Traffic Accidents Deaths in Philippines reached 9,758 per year, or 1.87% of total deaths. The age adjusted Death Rate of 11.56 per 100,000 of population ranks the Philippines number 123 out of 172 globally in terms of road accident risk (WHO 2014)

SNRDP Road Safety Program – Pre and Post Construction Assessment: In 2011, as part of the International Roads Assessment Programme (iRAP) Philippines project, iRAP¹ was invited by DPWH and the Millennium Challenge Corporation (MCC) to assess road projects in Samar which were being upgraded as part of SNRDP, provide advice on Star Ratings of the new road designs, and to establish a baseline. iRAP's Star Rating risk assessment results and a Safer Roads Investment Plan (SRIP) with economic analysis of safety countermeasure options were used to formulate an approach to road safety that was appropriate to the project. That assessment was conducted as part of national road assessments financed by the World Bank Global Road Safety Facility. The assessment found that the majority of the existing roads were rated in the highest risk 1- and 2-star categories. See Table 1

Table 1. Percentage of road rated 3-stars or better

Road user	Before upgrade
Vehicle occupants	23%
Motorcyclists	10%
Pedestrians	8%
Bicyclists	20%

The 16.3km Buray-Tinani Road section of SNRDP (CP1) was completed in 2014. In May 2015, a study was commissioned by the Millennium Challenge Corporation (MCC) to compare the baseline iRAP road safety assessment results produced in 2011 with post-construction assessment results for the recently completed upgrades, and to provide road safety recommendations that could be implemented along sections of the road identified as posing a risk to the travelling public.

2 POST-CONSTRUCTION SURVEY METHODOLOGY

Using a video camera, GPS and specially created software, iRAP inspections involve detailed road surveys and data collection focusing on more than 50 different road attributes known to influence crash likelihood and severity. These attributes include intersection design, numbers of lanes and markings, roadside hazards, footpaths and bicycle facilities. The road inspection data underpins subsequent designation of iRAP Star Ratings and preparation of Safer Roads Investment Plans (SRIP).

iRAP Star Ratings are based on road inspection data and provide a simple and objective measure of the level of safety which is 'built-in' to the road for vehicle occupants, motorcyclists, bicyclists and pedestrians. Five-star roads are the safest while one-star roads are the least safe. Importantly, Star Ratings can be completed without reference to detailed crash data, which is often unavailable in low-income and middle-income countries.

The post-construction survey of CP1 was undertaken as part of a three-day training course for DPWH, MCAP and MCC staff. The survey was undertaken as follows:

- Date: Wednesday 3 June 2015.
- Start time: 2:14pm

¹ The International Road Assessment Programme (iRAP) is a registered charity dedicated to preventing the more than 3,500 road deaths that occur every day worldwide. At the heart of iRAP is a spirit of cooperation. iRAP provide tools and training to help automobile associations, governments, funding agencies, research institutes and other non-government organisations in more than 70 countries make roads safe.

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- End time: 2:32pm
- Direction: East-bound.
- Start point: KM827.144 (intersection with Daang Mahalika).
- End point: KM843.5.
- Vehicle and driver: Toyota Hiace and driver organized by MCC.
- Camera: One (1) Contour+2 was mounted inside the windscreen of the vehicle.
- Images: Continuous video, 1280 x 960 pixels, 170 degrees field of view, .MP4 format.
- GPS: Collected using the Contour+2 video camera, downloaded in .CSV format.

Prior to the iRAP survey, a series of site visits to specific locations along the road were conducted by the joint iRAP, DPWH, and MCA-P team to inform the survey.

The post construction assessments were based on the ‘V3.02’ iRAP model, whereas the original baseline assessment was conducted using the ‘V2.2’ model. The V3 models series reflects experience in applying the V2.2 to more than 50,000km of roads in low and middle-income countries and new research on the relationships between road attributes and risk. As part of this project, the results from the baseline assessment were recalculated using the V3 model and migrated into iRAP’s ViDA software. This required some basic assumptions to be made for road attributes not previously coded. However, taking this step helps ensure a relatively high degree of compatibility between the analyses.

Traffic speeds: The risk level for death or serious injury on a given road is highly dependent on the speed at which traffic travels. iRAP policy is that risk assessments are made using the ‘operating speed’ on a road. Operating speed is defined as being the greater of the legislated speed limit or the measured 85th percentile speed.

There were a number of issues to consider in determining the operating speeds for CP1:

- Although the driver of the post-construction survey vehicle was asked to obey the road rules, the survey vehicle consistently exceeded the posted speed limits. On average, the vehicle travelled at 8km/h faster than the speed limit. The 85th percentile speed of the survey vehicle was 68km/h and the maximum speed was 79km/h. To the extent that the survey vehicle driver is representative of all drivers, then it is reasonable to assume that operating speeds exceed posted speed limits.
- The post-construction survey indicated that there was little relationship between changes in posted speed limits and changes in operating speeds (this is discussed in more detail later in the report).
- The post-construction survey indicated that there was little relationship between changes in area type (urban and rural) and changes in operating speeds (this is discussed in more detail later in the report).
- In the original baseline report, the following speeds were used for the analyses: 51-60km/h in urban/semi urban areas and 61-70km/h in rural areas. However, for this assessment, these speeds were reduced following a review of the survey vehicle speeds. In that case, the survey vehicle travelled at an average of 34km/h, an 85th percentile speed of 45km/h and a maximum speed of 51km/h. As in the post-construction survey, there was little relationship between area type (urban and rural) and operating speeds (and there were no speed limit signs in place).

Taking into account these factors, the speeds shown in the Table 2 below were used in the modelling. The speeds selected reflect an attempt to balance the issues discussed above. However, sensitivity tests with different speeds were also undertaken. Further discussion with the DPWH may be necessary to confirm the appropriate operating speeds to use on these roads.

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Table 2. Assumed operating speeds

Road	85 th percentile speed	Average speed (km/h)
Before upgrades	46-50km/h	40-45km/h
After upgrades	61-65km/h	55-60km/h

Risk worms: Figures 2 - 5 shows charts illustrating the Star Rating Scores (SRS) that underpin the Star Ratings for each of the road users (the higher the SRS, the greater the risk). The charts help to illustrate the relative level of risk each type of road user experiences as they move along the road. The dark blue lines illustrate risk before the road upgrade and the light blue lines illustrate risk after the road upgrade.

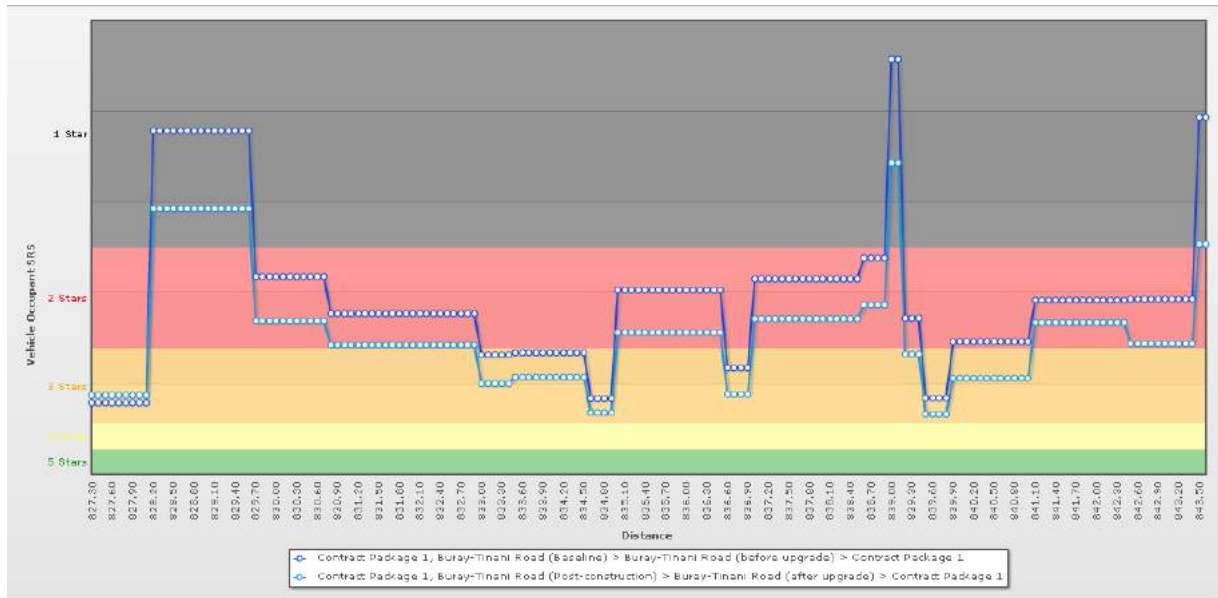


Figure 2. Vehicle occupant risk scores

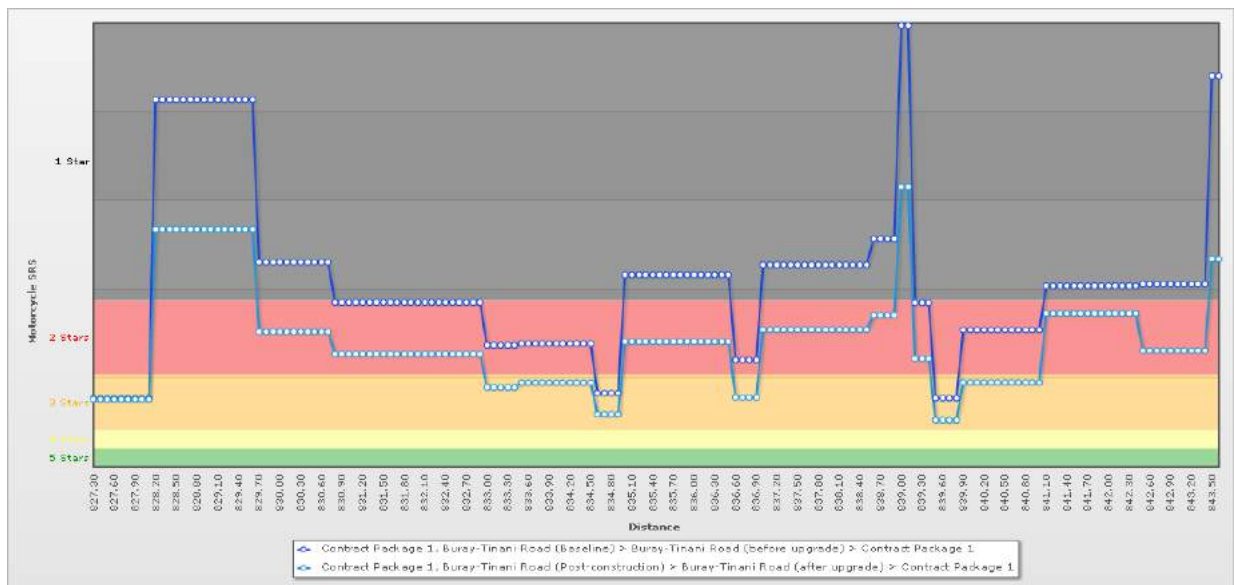


Figure 3. Motorcyclist risk scores

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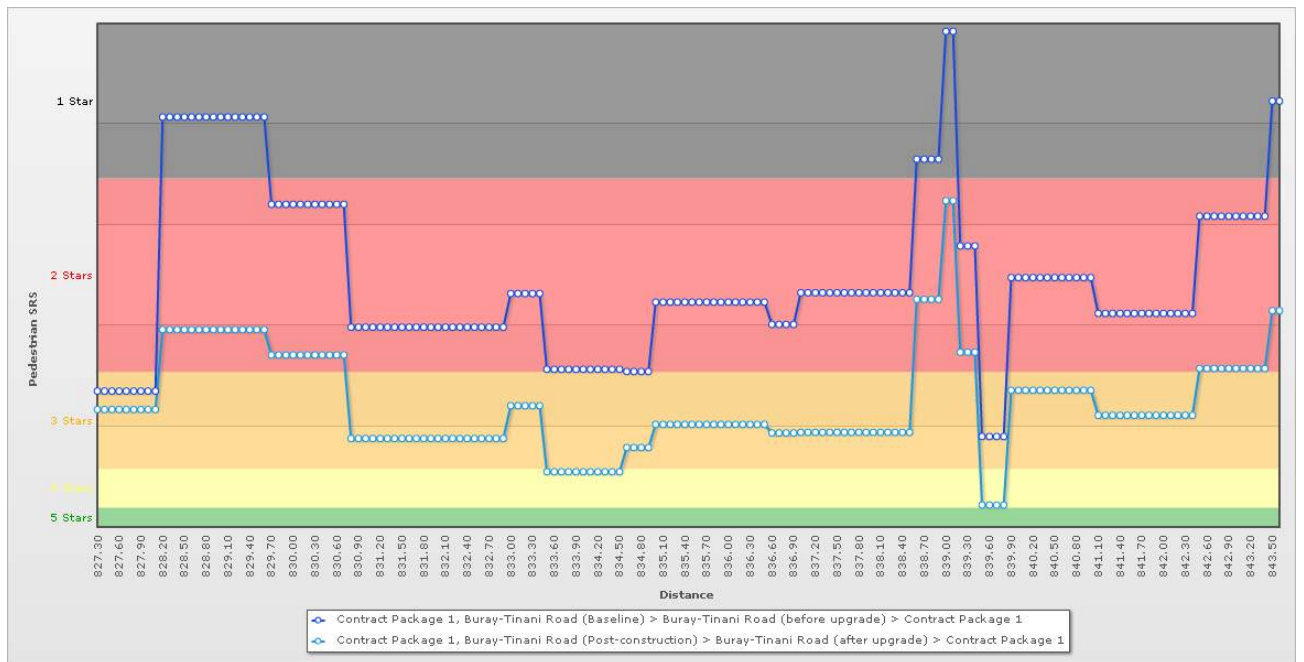


Figure 4. Pedestrian risk scores

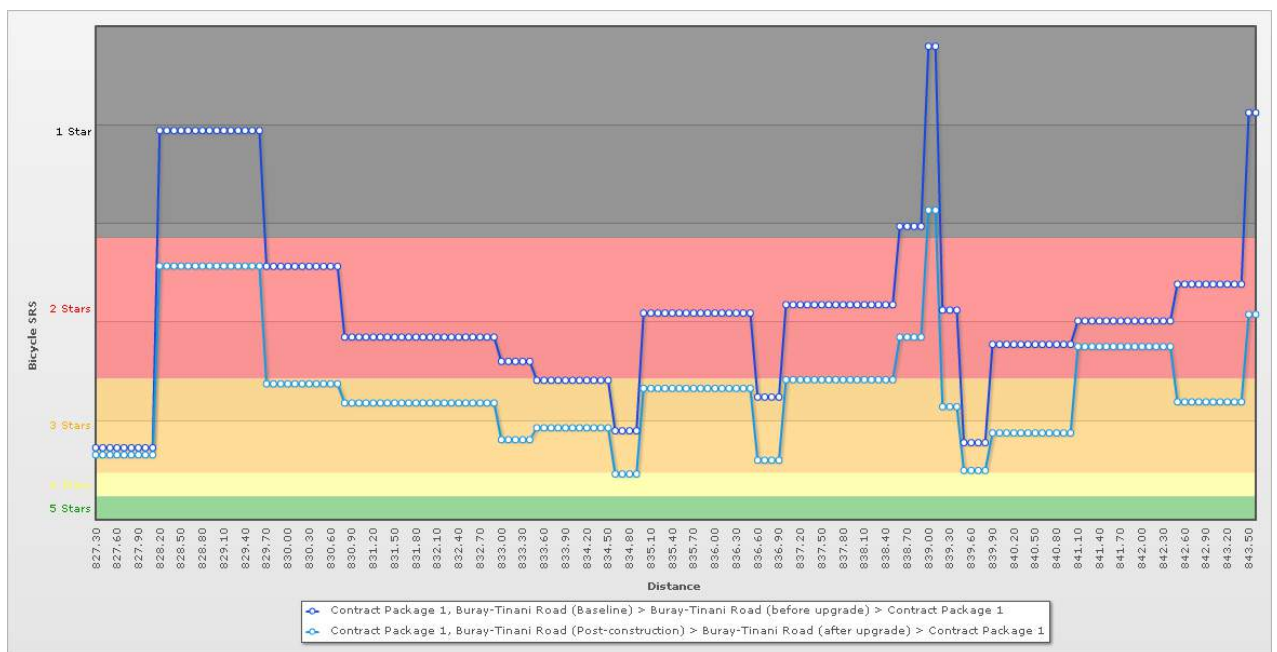


Figure 5. Bicyclist risk scores

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3 ECONOMIC ANALYSIS

Baseline number of deaths and serious injuries: As part of the iRAP model calibration, an estimate of the number of deaths and serious injuries that occur on the road, both in aggregate terms and by road user type, is required. It is noted these estimates only influence the economics component of the project, and not the risk assessments (Star Ratings). As was noted in the baseline report, there is no information available on the number of deaths and serious injuries on the Wright-Taft-Guiuan Roads and so it was assumed that there was a death rate of 0.5 deaths per km. For the Buray-Tinani Road in this report, a lower figure, 0.2 deaths per km, has been assumed, reflecting the lower volumes and speeds on this section. On this basis, it is estimated that $0.2 \times 16.3 = 3.3$ deaths occur each year, or 65.6 deaths over 20 years.

In the baseline report, it was assumed that deaths are equally distributed among vehicle occupants, motorcyclists, pedestrians and bicyclists. However, this assumption has been adjusted for this report following the observation that very few bicyclists use the road. Hence the distribution is as follows:

- Vehicle occupants: 33%
- Motorcyclists: 33%
- Pedestrians: 33%
- Bicyclists: 1%

An estimate of the number of serious injuries was made by assuming that for each death, 10 serious injuries occur. (McMahon and Dahdah 2008). Hence, it is estimated that 32.8 serious injuries occur on the road each year, or 655.8 serious injuries over 20 years. Combining this with the estimated number of fatalities gives an estimated 721 fatalities and serious injuries over 20 years.

Economic cost of deaths and serious injuries: iRAP uses a standard approach globally to estimate the economic cost of deaths and serious injuries, based on research undertaken by McMahon and Dahdah for the organization (McMahon & Dahdah 2008). It is the approach preferred by the Global Road Safety Facility for iRAP projects. It is noted that this approach may result in estimates that differ from those undertaken in the past due to new methods for calculating economic costs and changes in technology.

The key equations used are:

- the economic cost of a death is estimated to be: $70 \times$ Gross Domestic Product (GDP) per capita (current price)
- the economic cost of a serious injury is estimated to be: $0.25 \times$ economic cost of a death.
- For the purposes of this assessment, the baseline report costs were used, namely:
- the economic cost of a death in the Philippines is estimated to be $70 \times$ Php 97,840 = Php 6,848,800
- the economic cost of a serious injury is estimated to be: $0.25 \times$ Php 6,848,800 = Php 1,712,200.

Overall, it is estimated that 721 deaths and serious injuries occurred on the road segment surveyed over a 20 year period before the upgrade.

4 SAFER ROADS INVESTMENT PLAN

iRAP considers more than 90 proven road improvement options to generate affordable and economically sound Safer Road Investment Plans (SRIP) that improve a road's Star Ratings and save lives. Table 3 shows the safety countermeasures that could enhance safety and which are likely to be economically viable (with a benefit cost ratio greater than 1). The analysis indicates that if all the countermeasures in the enhanced safety package were installed, 279 fatalities and serious injuries could be prevented over 20 years, representing a 38% reduction. This would save approximately Php 226 million in estimated costs associated with deaths and serious injuries.

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Table 3. Suggested Road Safety Countermeasures (20 years)

Countermeasure	Length / Sites	FSIs saved	Safety benefit (Php)	Estimated Cost (Php)	Cost per FSI saved (Php)	BCR
Improve curve delineation	7.80 km	53	43,151,659	7,515,493	141,745	6
Roadside barriers - passenger side	5.40 km	50	40,931,298	13,768,920	273,774	3
Roadside barriers - driver side	5.30 km	49	40,272,062	13,513,940	273,103	3
Unsignalised crossings	30 sites	21	16,880,786	3,129,825	150,895	5
Clear roadside hazards - passenger side	7.00 km	18	14,590,809	1,238,300	69,071	12
Delineation and signing (intersection)	12 sites	16	12,741,072	4,600,054	293,836	3
Clear roadside hazards - driver side	6.70 km	16	13,187,209	1,185,230	73,147	11
Traffic calming	2.10 km	14	11,548,066	6,898,717	486,191	2
Parking improvements	3.70 km	12	9,712,267	5,550,000	465,072	2
Upgrade pedestrian facility quality	9 sites	10	7,906,919	1,189,776	122,463	7
Pedestrian fencing	0.30 km	4	3,437,169	450,000	106,552	8
Footpath provision driver side (adjacent to road)	0.90 km	4	3,285,158	2,880,000	713,484	1
Footpath provision passenger side (adjacent to road)	0.80 km	3	2,630,770	2,560,000	791,964	1
Footpath provision passenger side (informal path >1m)	1.90 km	3	2,174,091	1,540,205	576,566	1
Footpath provision driver side (informal path >1m)	2.00 km	3	2,309,977	1,621,268	571,209	1
Skid Resistance (paved road)	0.20 km	2	1,903,918	832,843	356,011	2
Total		279	226,663,230	68,474,571	245,865	3

5 CONCLUSION

The assessment shows in Table 4 that there has been an improvement in the Star Ratings for the Buray-Tinani Road, although a significant percentage of the road remains in the 1- and 2-star categories.

In summary:

- the percentage of road rated 3-stars or better for vehicle occupants increased from 23% to 32%
- the percentage of road rated 3-stars or better for motorcyclists increased from 10% to 30%
- the percentage of road rated 3-stars or better for pedestrians increased from 8% to 36%
- the percentage of road rated 3-stars or better for bicyclists increased from 20% to 75%.

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Table 4. Star Ratings: before upgrades (B) and after upgrades (A) and the difference (D)

Star Rating	Vehicle occupants			Motorcyclists			Pedestrians			Bicyclists		
	B	A	D	B	A	D	B	A	D	B	A	D
5	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
4	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	2%	2%
3	23%	32%	9%	10%	30%	20%	8%	36%	28%	20%	75%	55%
2	66%	57%	-9%	35%	59%	24%	30%	44%	14%	66%	21%	-45%
1	12%	12%	0%	55%	12%	-43%	62%	20%	-42%	14%	1%	-13%
Total	100%	100%	0%	100%	100%	0%	100%	100%	0%	100%	100%	0%

The assessment in Table 5 shows that there have been modest improvements in the road safety Star Ratings for the Buray-Tinani Road as a result of the upgrades, despite operating speeds increasing by an estimated 15km/h. It is further estimated that as a result of the road improvements, the level of road accidents will decline by 70 deaths and serious injuries (10%) over 20 years, saving an estimated Php 57 million in costs associated with road accidents.

Table 5
Improvement in Star-Rating

Percentage of road rated 3-stars or better Road user	Before upgrade	After upgrade
Vehicle occupants	23%	32%
Motorcyclists	10%	30%
Pedestrians	8%	36%
Bicyclists	20%	77%

Road attributes on the upgraded road that have led to a reduction in accident risk include: improved delineation, street lighting, paved shoulders, pedestrian crossings, sidewalks, pedestrian fences and increased roadside clear zones. However, a number of serious road safety issues were identified during the assessments, so numerous opportunities to enhance safety on the road remain. An economic analysis of safety countermeasure options identified approximately Php 68 million of investments that could prevent a further 279 (38%) deaths and serious injuries, potentially saving Php 224 million in accident-related costs. This investment would eliminate virtually all the 1- and 2-star sections of road. The improvements include: better traffic management at the section of road where there has been a landslip; making adjustments at schools such as extending sidewalks and reducing speeds; upgrading and extending safety barriers; improving delineation at curves and intersections; and installing village ‘gateway’ treatments. DPWH has acknowledged the proposed improvements are highly beneficial in further improving road safety condition and is willing to incorporate them to the maximum extent moving forward.

In addition to taking a more comprehensive approach to road safety engineering, significant benefits could also be realised through coordinated targeting of risk factors for road users (such as speeding, seat belt use and driving under the influence) and vehicles. This would be consistent with taking a Safe System approach to the road project. To this end, the iRAP assessment was followed up with a joint DPWH and MCA-P led public awareness campaign that aimed to communicate a message of road safety to both the travelling public and to communities along the road. Particular attention was paid to engaging administrators, students and parents of students in schools along the road alignment in an effort to instil awareness of road safety practices for not only drivers and users of vehicles, but for pedestrians as well. This engagement and awareness raising of road safety roles and responsibilities among the road’s user community formed an essential complementary socialization feature to the physical safety measures recommended by the iRAP assessment.

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6 BEFORE AND AFTER IMAGES

The following are images collected in the baseline survey (2011) and the post-construction survey (2015) for selected locations.

KM831 (left = before upgrade; right = after upgrade)



KM832 (left = before upgrade; right = after upgrade)



KM833.4 (left = before upgrade; right = after upgrade)



KM834 (left = before upgrade; right = after upgrade)

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KM837 (left = before upgrade; right = after upgrade)



KM838 (left = before upgrade; right = after upgrade)



KM840 (left = before upgrade; right = after upgrade)

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KM842 (left = before upgrade; right = after upgrade)



KM843.1 (left = before upgrade; right = after upgrade)



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PAPER TITLE	Development of Development of Non-contact Wireless Power Supply Pavement		
TRACK	Pavements or Mobility Management		
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KEYWORDS:

Pavement, EV, Non-contact, Wireless power supply, shield

ABSTRACT:

Electric vehicle (EV) is, no CO₂ generation by the service, has been defined as the environmentally friendly car. There is a can travel up to 200km in relation to the capacity of the battery mounted on a vehicle, for summer air conditioning and the load of the winter heating is large, one can only travel a maximum of about 100km in charge. For this reason, in addition to the charging stand to promote the spread of electric vehicles, it will need to charge in the parking zone of the stopped or running.

To improve the efficiency of the power supply coil embedded in the pavement, it was decided to use magnetic material sand generated during steelmaking slag purification. It decided to use the cement and asphalt emulsion mortar (CAM) in order to protect the buried coils from loading due to the moving track. The structures of the pavement have developed two types of asphalt pavement structure and dish type concrete structure using the CAM.

In this paper, with respect to leakage radio waves at the time of the non-contact wireless power supply, knowledge was obtained about the validity and the direction of the magnetic field at the time of the occurrence of an aluminum plate. Future advances the study of the structure and buried depth of the coil at the time of high power output.

Development of Non-contact Wireless Power Supply Pavement

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1 CONCEPT AND CIRCUMSTANCES OF NON-CONTACT WIRELESS POWER SUPPLY

Environmentally friendly electric vehicles can run only up to approximately 100 km on one charge due to the capacity of batteries installed in the vehicles. Therefore, the key for the promotion of electric vehicles is developing a charging infrastructure that allows batteries to be wirelessly charged while driving, as well as expanding charging stations. Measures need to be considered to prevent electromagnetic waves from moving laterally without losing the efficiency of the battery charge from the supply coil installed beneath the pavements of roads.

Furthermore, it is essential to install magnetic materials and a layer of shields under the coils, as the magnetic flux needs to expand upward for the coil installed on cars to efficiently receive electromagnetic waves generated from the underground coil. (Figure 1).

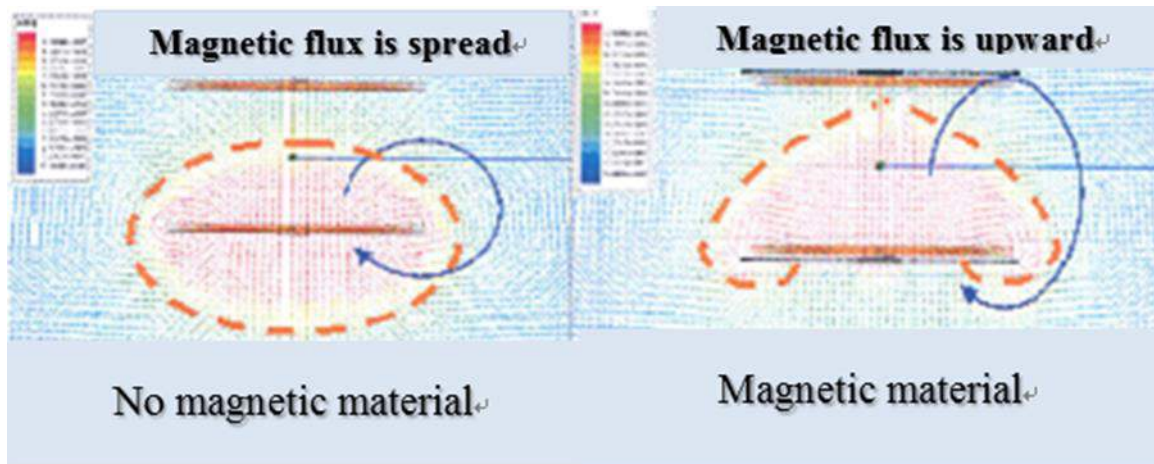


Figure 1. Image of dispersion of magnetic flux due to the presence or absence of a magnetic material.

2 COIL AND ELECTRICAL CHARACTERISTICS

A 12mm-diameter Litz wire was used for the coil. The receiver coil – a circular sextuple coil 40 cm in diameter – was, in principle, installed under the car.

The transmitting coil embedded in a pavement was a triple coil 1.6m in length and 30 cm in width, as described in Figure 2 (Throngnumchai, K. & Hanamura, A. & Naruse, Y. & Takeda, K. 2013).

The embedded transmitting coil was designed in a way that the inductance (L), which indicates the electromotive force generated in the coil, is 33 μ H under 40A current and the Q-factor (Quality factor)

reaches the maximum value under 80-100kHz frequency (Figure 3). The Quality factor under 90kHz was 440.

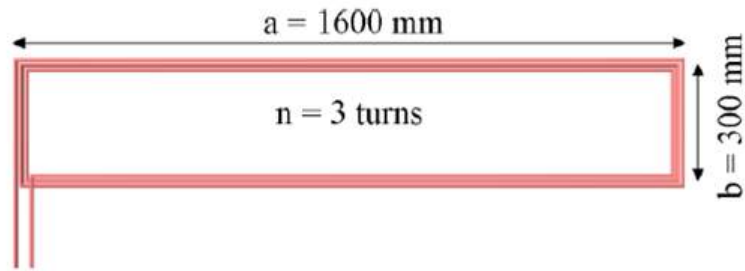


Figure 2. Structure of coil embedded in pavement in this study.

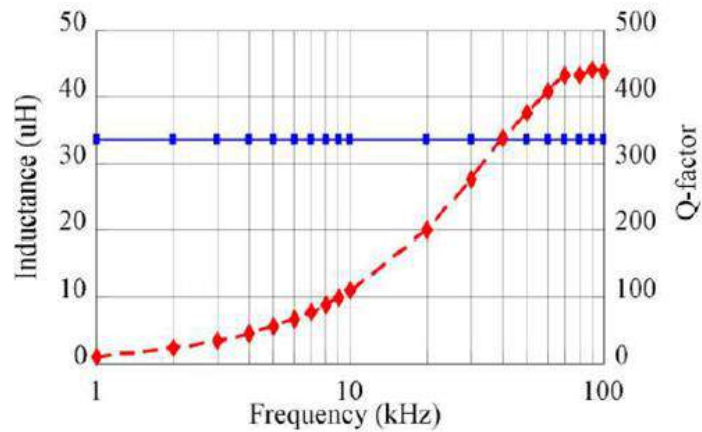


Figure 3. Inductance and Q-factor of embedded coil.

Based on experience of installing equipment under road pavements, it was decided to have a loop length of 1.6m and width of 300mm, and the electric field intensity was measured at a point 30m away while increasing the current A (Power W). The test condition is described in Photo 1, and the result is described in Figure 4.

This test result revealed that embedding the power supply coil in the ground mitigates the impact of electromagnetic waves to the surrounding areas. Following this result, we went on to study the structure that ensures the shielding performance against the surrounding areas, and the electric field intensity to the car driving above the embedded coil.



Photo 1. Measurement situation at a point 30m away.

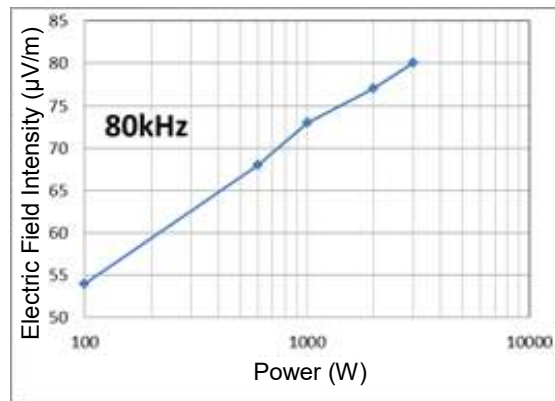


Figure 4. Generated electromagnetic waves and electric field intensity at a point 30m away.

3 MATERIALS FOR WIRELESS POWER SUPPLY AND TEST PAVEMENT

Materials for a non-contact power supply pavement also need to have functions for collecting electromagnetic waves generated from the coil and blocking the waves from traveling sideways and downward.

Materials typically used for pavements are aggregates such as crushed stones and sands as well as bonding materials such as asphalt and cement. Among those materials, we focused on slag, produced in the process of iron manufacturing, as the magnetic body. The elements of slag are described in Table 1, and aggregates we examined are described in Table 1. In comparison to crushed stones and sands of converter slag Agg. and steel slag (55) Agg., steel slag (75) sands have the highest level of Fe in their content (Photo 2). Steel sands are used as iron sands as the main element is Fe.

Table 1. Main elements of steel slag aggregates examined.

Item	Total Fe (T-Fe)	Lime (CaO)	Silica (SiO ₂)	Alumina (Al ₂ O ₃)	Magnesium oxide (MgO)	Phosphoric acid (P ₂ O ₅)	Sulfur (S)	Manganese oxide (MnO)	Total
Converter slag Agg.	21.4	38.42	15.81	1.94	7.1	2.27	0.04	5.12	92.1
Steel slag (55) Agg.	43.5	22.54	7.17	1.63	4.07	1.25	0.05	2.34	82.55
Steel slag (75) Sand	67.4	16.85	3.8	1.05	2.2	0.85	0.02	1.3	93.47



Converter slag Agg. Steel slag (55) Agg. Steel slag (75) Sand

Photo 2. Steel slag aggregates examined.

We measured the conductance (Q-factor) by securing a coil, wound into a circular form, to a permeability measurement specimen (Photo 3), built inside a room, to determine the permeability characteristics.

The result is described in Figure 5. The Q-factor of converter slag is lower than others. This test shows that just 20 mm of magnetic sand or CAM with magnetic sand has higher permeability than just a coil.

The asphalt structure we used is described in Figure 6 (Abe, N., Manabe, K. (2013)). 20mm of CAM including magnetic sands was laid under the coil.

We used deformed stainless rebar (SUS304), typically used as support members, in the concrete trough structure to integrate it into the structure as described in Photo 4. Each block in the concrete trough structure was 2m long. The flexural capacity and shear capacity were examined, and the stainless rebar structure was built with D13 main bars and 9mm distributing bars so that the structure can withstand a heavy vehicle driving over it (Figure 6).

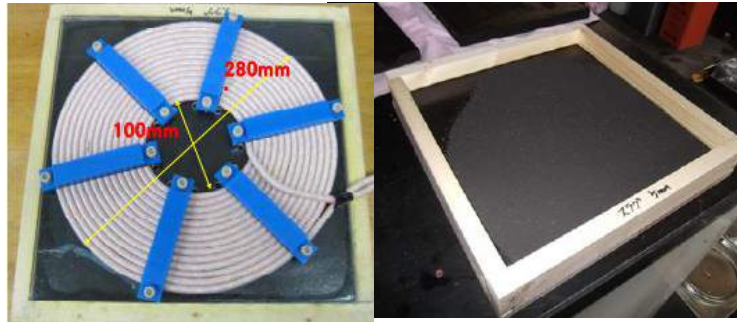


Photo 3. Specimen for permeability measurement and with laid coil.

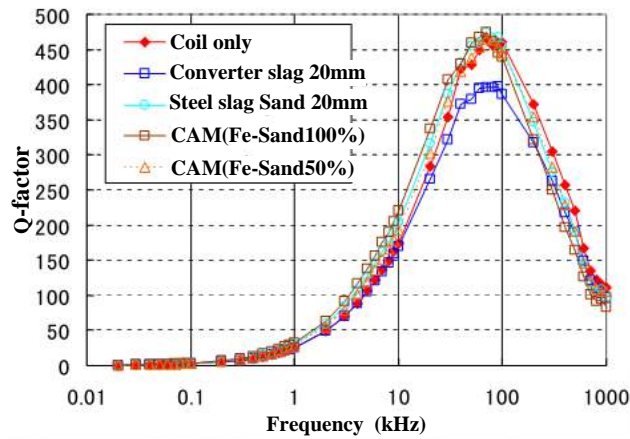


Figure 5. Q-factor against frequency of each specimen.

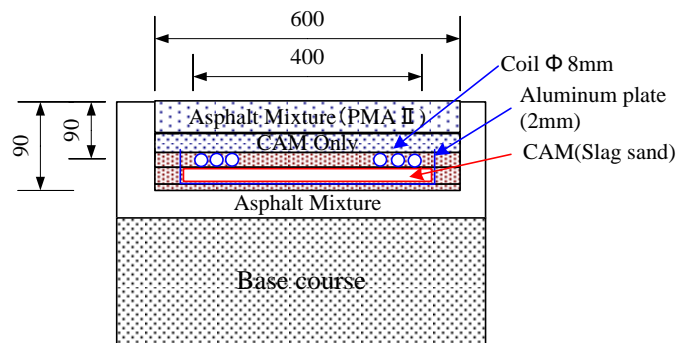


Figure 6. Wireless power supply asphalt pavement.



Photo 4. Stainless rebar used for concrete trough.

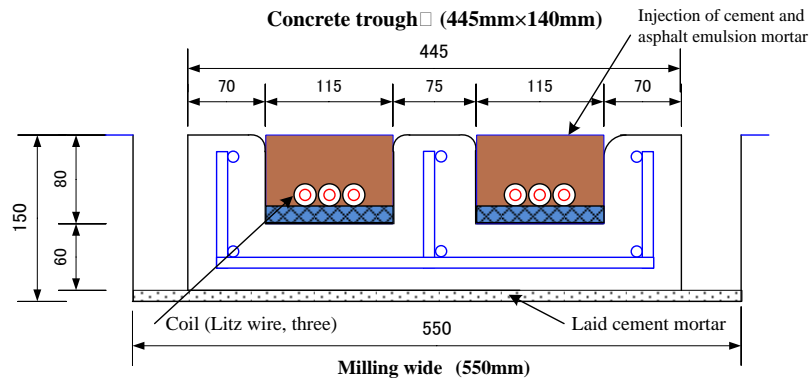


Figure 6. Wireless power supply concrete trough pavement.

Table 2. General description of magnetic materials used for pavement.

Type	Magnetic Body Description
A	Silica sands in the CA mortar are replaced with magnetic body A
B	As composite (aggregate) is replaced with magnetic slag
C	Aluminum rebar, electrically more stable than regular deformed rebar, is used for trough
D	Stainless rebar, electrically more stable and more resistant to corrosion than regular deformed rebar, is used for trough
E	Aluminum plate

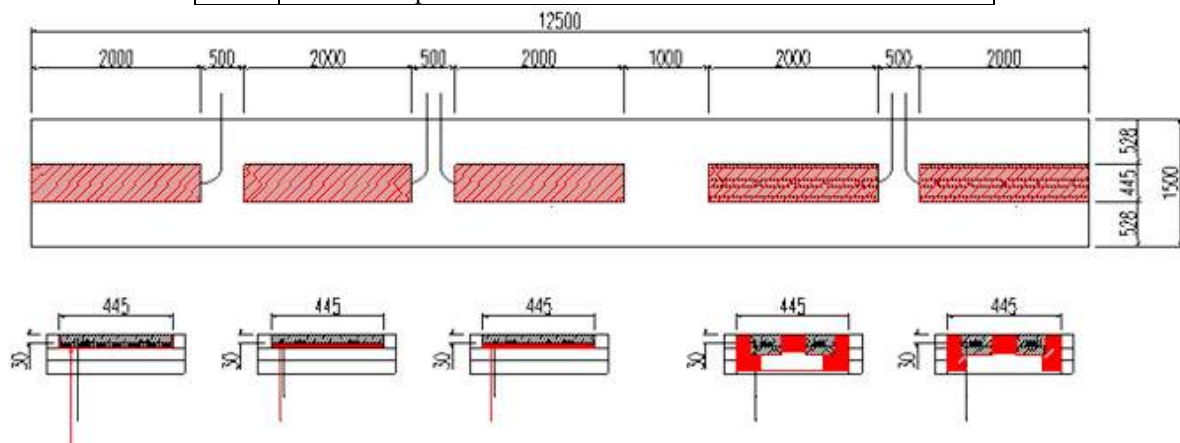


Figure 7. A plan view and a cross-sectional view of the test pavement.

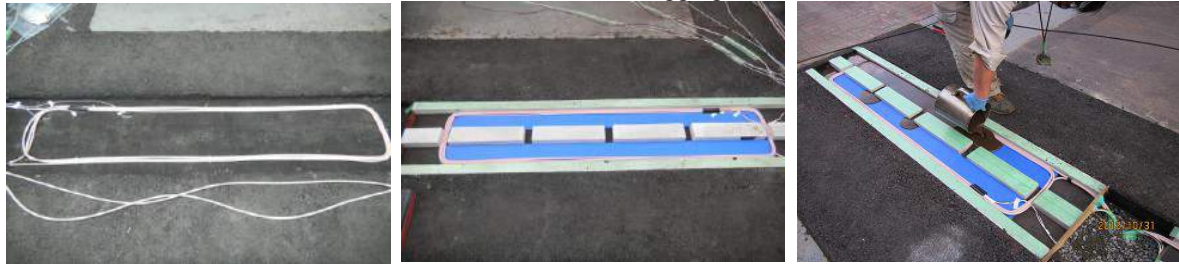
The magnetic body used in this test pavement and each work area of the test installation of shield materials are outlined in Table 2.

Type A is CA mortar (asphalt emulsions: cement: silica sand = 1.4: 1.0: 1.0), currently used as buffer materials installed under slab tracks for the Shinkansen. As a replacement, magnetic sands are used instead of silica sands (aggregate) in the CA mortar. Magnetic sands are burned iron oxide, which is the main

component, and in powder form, of which the particle size is nearly the same as a silica sand. Type B is sands of steel slag, which contains more than 65% iron oxide, being used as fine aggregate for asphalt composite.

Type C and Type D are secondary concrete products with stainless rebar and aluminium plates (1.2mm), which contain the shielding effect that reduces the amount of the magnetic flux diffusing laterally. More accessible aluminium plates were used in Type E. Figure 7 describes the test pavement plan and section. The pavement width of the power supply lane was decided to be approximately 450mm in consideration of wheel distance of the car, and all coils were buried at a point where the distance from the pavement surface to the bottom of the coil is 50mm. Three coils were buried in this test.

Photo 5 describes the coil installation of each type. We adjusted the finished surface level of magnetic materials to maintain the 50mm depth to the buried coils, and the upper surface of the coils was sealed with the standard CA mortar that used silica sands as aggregate.



Type B, Coil installation;

Type C, Coil installation;

Type C, Re-embedding of coil

Photo 5. Coil installation in each type of the test pavement.

4 VERIFICATION OF SUPPLY AND OF MAGNETIC BODY

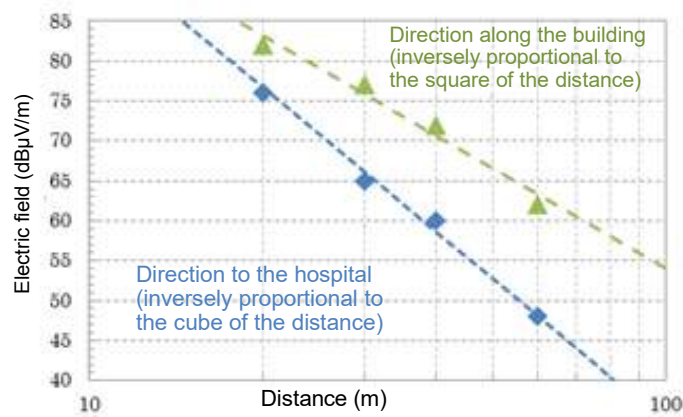
We measured surrounding areas when 200V voltage is applied to embedded in the measuring conditions of fields are described in measurements of the coil are described in Figure 8. The electric field intensity gradually decreases as the distance increases. It is inversely proportional to the cube of the distance in the direction perpendicular to the vehicle's running direction, and it is inversely proportional to the square of the distance in the direction parallel to the vehicle's running direction.



WIRELESS POWER MEASUREMENT TEST

radio wave leakage to the 5A current (1kW) at the power supply coil pavement. The electric and magnetic Photo 5. The electric field by the loop





Measurement of magnetic field

Figure 8. Electric field value against distance.

The permissible value of radio wave leakage is specified as being less than 3.46mV/m at a measurement distance of 30m. The results in this test indicate that the values are sufficiently small, and the current structure is capable of supplying power from 6kW to 9kW, although there are issues for the permissible value of the electric field intensity of the directions perpendicular and parallel to the vehicle’s running direction.

LED lights were connected from the power receiving coil under the car (Photo 6) through the batteries and installed inside the car (NISSAN New Mobility Concept). The LED lights are turned on when receiving the power so that one can visually confirm while driving if the car is receiving the electromagnetic waves from the embedded coil when current is being applied. Photo 7. shows the driving test being conducted. The LED lights clearly lit up when the car ran in such a way that the power receiving coil stayed within 450mm width of the power supply lane pavement, which indicates the power was being supplied wirelessly (Photo 8). However, the LED lights blinked randomly when the car ran in such a way that the receiving coil was off the supply lane by 150mm to the side, which indicates the power was not being received efficiently.

Therefore, we measured the magnetic field when the coil is away from the lane, using an Exposure Level Tester, ELT-400 (Narda) as indicated in Photo 9. The magnetic field, Tesla, in a case of the receiving coil being offset by 150mm to the side was a half of the value of the coil directly above the lane, and it was nearly impossible to measure the magnetic field in a case of the coil being offset by 300mm. Therefore, it is clear that minimizing lateral movement is necessary when driving electric vehicles during the wireless power supply stage on the power supply pavement.

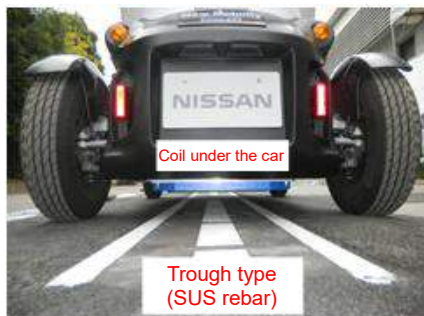


Photo 6. Power receiving coil under the car



Photo 7. Driving test



Photo 8. LED lights are turned on

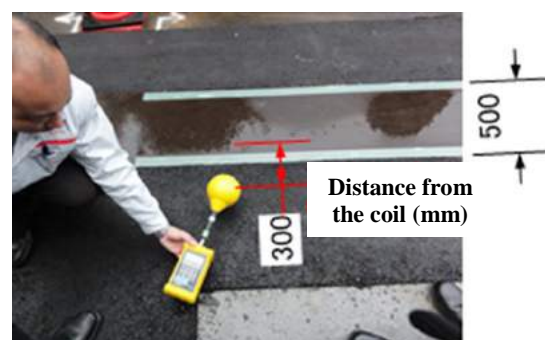


Photo 9. Magnetic field measurements when apart laterally from the coil.
(Exposure Level Tester, ELT-400 (Narda), Low frequency 1Hz-400kHz)

5 CONCLUSIONS

This test pavement construction and driving test are basic tests aimed for the practical use of a non-contact wireless power supply system, and there are many problems to be solved in our goal to develop the infrastructure. However, we were able to achieve the second stage for practical use as we could verify that the coil can receive electromagnetic waves, transmitted from the underground-embedded coil when the current is applied, installed in the car, and the batteries can be charged.

Our future tasks shall be optimizing the power supply system by designing more efficient switch systems and shields for magnetic flux, and pursuing the practical use of a non-contact wireless power supply for electric vehicles.

6 ACKNOWLEDGEMENTS

We would like to express our gratitude to Mr. Kraisorn Throngnumchai, Mr. Akihiro Hanamura, and Mr. Kazuhiro Takeda of EV System Laboratory, Nissan Research Center, Nissan Motor Co., Ltd. for their support in this research.

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PAPER TITLE	Development of the three-dimensional road surface measurement technology by the high-speed road surface measurement vehicle		
TRACK			
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KEYWORDS:

Inertia Measurement Unit (IMU)
 Global Positioning System (GPS)
 Three-dimensional
 Non-Contact Measurement

ABSTRACT:

Our Company has developed a vehicle to measure road surface of expressway. It is capable of measuring ruts, cracks and IRI at high speed since 1984. We have developed technology which incorporates Inertia Measurement Unit (IMU) and Global Positioning System (GPS) data with our high-speed road surface measurement vehicle since 2004. This technology enables detailed measurements of extensive three-dimensional road surfaces at any speed up to a maximum of 100 km/h (60 mph). The greatest advantage of this vehicle is that can measure road surfaces with non-contact measurement and allows smooth and safe measurement even though there are other vehicles traveling on the same road.

We can detect deteriorated road conditions very quickly such as rutting, impact and puddle of water. Thus, we can make countermeasure in a timely manner and secure the safety driving and contribute to disaster prediction. We have measured 5,710 km of road surfaces and made repair plans for 263 areas.

Development of the three-dimensional road surface measurement technology by the high-speed road surface measurement vehicle

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1. Introduction

Expressways in Japan are periodically inspected to check their road surface properties (rutting, cracks, international roughness index (IRI), etc.) in order to maintain good surface conditions. Vehicles that can measure the road surface while traveling at high speed have been developed and put into service to prevent causing congestion and minimize the risk of accidents during inspection. In recent years, expressway corporations have investigated a Pavement Management System (PMS) that takes into account various kinds of road surface evaluation data, as well as the conventional control standards shown in Table 1, to draw up highly detailed pavement repair plans.

Deformation of road surfaces, such as are caused by the road settlement at the boundary between a bridge and earthworks and between cut and embanked sections, becomes increasingly apparent the longer the roads are used. Such deterioration not only makes the road uncomfortable to drive on, but may also aggravate damage to road structures and deterioration of the roadside environment. Poor longitudinal and cross-sectional profiles caused by such settlement may also inhibit the smooth discharge of rainwater, resulting in water remaining on the road surface, which destabilizes and endangers driving. Therefore, understanding the longitudinal and cross-sectional profiles of the road surface is essential to construct an advanced PMS.

Deformation of road surfaces over relatively long distances has ordinarily been measured by restricting lanes. Thus, measurement has been difficult on heavy traffic routes such as the Tomei and Meishin Expressways because any lane restriction is likely to cause congestion, and on sections with temporary service, where the traffic volume may be small but taking the measurements can be dangerous.

Against such a background, we developed a system that can measure the three-dimensional profiles of the road surface while traveling at speeds of up to 100 km/h without needing to restrict lanes. This was achieved by adding a global positioning system (GPS) and an inertial measurement unit (IMU) to the vehicles previously developed by us to measure road surfaces. This system has been used since 2003.

Similar road surface measuring vehicles equipped with GPS and IMU are available. No organization other than us, however, has used these measuring vehicles to conduct longitudinal and cross-sectional measurements on expressways under the control of NEXCO group companies. The measurement technology has no compare anywhere else in the world.

This paper gives an overview of the technologies used in the new road surface-measuring vehicle.

Table 1: Target values in road surface repairs adopted by expressway companies

	Rutting (mm)	Level difference (mm)		Skid resistance coefficient (μV)	Roughness ($\sigma 3m$) or IRI (mm/m)	Cracking ratio ^a (%)
		Joint to a bridge	Joint to a traverse structure			
National expressways	25	20	30	0.25	3.5	20
Motorways						

^aWe analyzed the road in one-meter* sections – a quarter of the lane width.

Then the cracking ratio was calculated as the number of sections with cracking divided by the number of sections analyzed.

2. Overview of the technologies

2.1 Vehicle for measuring road surfaces (Road Tiger Ver.V)

In 1982, we developed the first version of the road surface-measuring vehicle, whose technology was assessed and approved by the then Ministry of Construction (present-day Ministry of Land, Infrastructure, Transport and Tourism) in 1983. The vehicle has undergone various improvements ever since. The vehicle used today can measure: 1) rutting, 2) cracking, 3) international roughness index (IRI), 4) flatness ($\sigma 3m$), and 5)



Photo 1: New road surface-measuring vehicle “Road Tiger Ver. V”

longitudinal and cross-sectional profiles; all simultaneously while traveling at speeds of up to 100 km/h, and thus does not cause traffic congestion. (Photo 1) (Table 2) (Figure 1)

Table 2: Basic specifications of the “Road Tiger Ver. V”

	Measuring method	Detecting device	Range of measurement	Measurement intervals (longitudinal)	Precision of measurement
Rutting	Multiple-point displacement measurement method	CCD camera	5.2 m (0.1-m intervals)	0.5m, 1m, 10m, 20m	Not exceeding ± 2 mm
Cracking	Continuous photographing	Streak camera	4.5 m	Continuous	Width: 1 mm or larger
IRI	Double integration of acceleration	Non-contact laser displacement meter	One survey line (OWP)	0.1m	Within $\pm 15\%$
Flatness $\sigma 3$ m	Non-contact 3-m profile meter	Non-contact laser displacement meter	One survey line (OWP)	1.5m	Within $\pm 15\%$
Longitudinal and cross section profiles	Global positioning data/ Inertial measurement unit	GPS/IMU	5.2m (0.1-m intervals)	0.5-m intervals	Mean vertical error: ± 11 mm

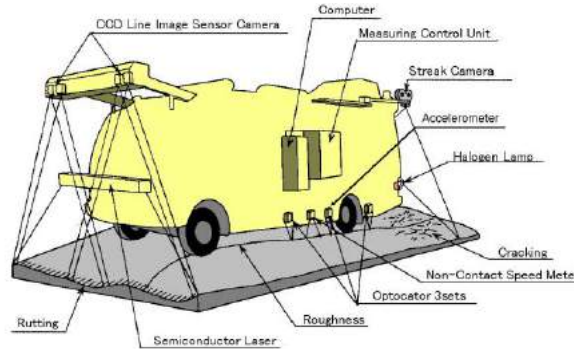


Figure 1: Schematic diagram of the “Road Tiger Ver. V”

2.2 Rut-measuring device

In the development of a three-dimensional road surface measurement system, the most critical of the existing technologies was the multiple-point displacement measuring technique used in the rut-measuring device, which has been used since our first road surface-measuring vehicle was developed in 1982. The technique requires a device with a complicated structure, which is thus difficult to calibrate, but it was the only mean by which we could obtain the correct vertical coordinate across the traveling direction at the necessary points, which is achieved by distance pulse control. Therefore, this technology was a prerequisite for the establishment of a three-dimensional road surface measurement system able to function at high speeds. The technology is described below.

The rut-measuring device consists of two laser slit projectors at the front of the vehicle and line sensor cameras. Each of the laser slit projectors projects 54 beams of light at 100-mm intervals across the road, with the two projectors combined covering a width of 5.2 meters. The light beams are projected diagonally toward the road surface, and thus the resultant shadows due to unevenness in the road surface are cast left and right. Their movement is detected by the line sensor cameras installed on the top of the vehicle. Because the positions from which the laser light beams are projected, the projection angles, and the angle of the line sensor cameras are mechanically determined, the changes in the positions of the light beams are monitored as observed angles, and the positions of the light beams (unevenness of the road surface) can be determined as intersections between projection and observation lines using the following equations. (Figure 2)

$$Y_s = \frac{\tan(\theta_1) \cdot Y_1 - \tan(\theta_m) \cdot Y_c + X_1 - X_c}{\tan(\theta_1) - \tan(\theta_m)} \quad (1)$$

$$X_s = -\tan(\theta_1) \cdot (Y_s - Y_1) + X_1 \quad (2)$$

where:

X_s, Y_s : coordinates of light beam,

X_c, Y_c : central coordinates of the lens of line sensor camera,

X_1, Y_1 : central coordinates of the light projected from laser slit projector,

θ_1 : projection angle of laser slit light, and

θ_m : observation angle of light beam.

The observation angle is determined from the mapping position of the CCD (Charge Coupled Device) pixels of the line sensor camera by the following equations.

$$lc = \frac{Lc}{Np} \times \left[N - \frac{Np - 1}{2} \right] \quad (3)$$

$$\theta_m = \tan^{-1} \left[\frac{lc}{Hc} \right] \quad (4)$$

where:

lc : mapping position, Lc : CCD length,

N_p : total number of CCD pixels,
 N : mapping position of pixel, and
 H_c : distance between CCD and the lens.

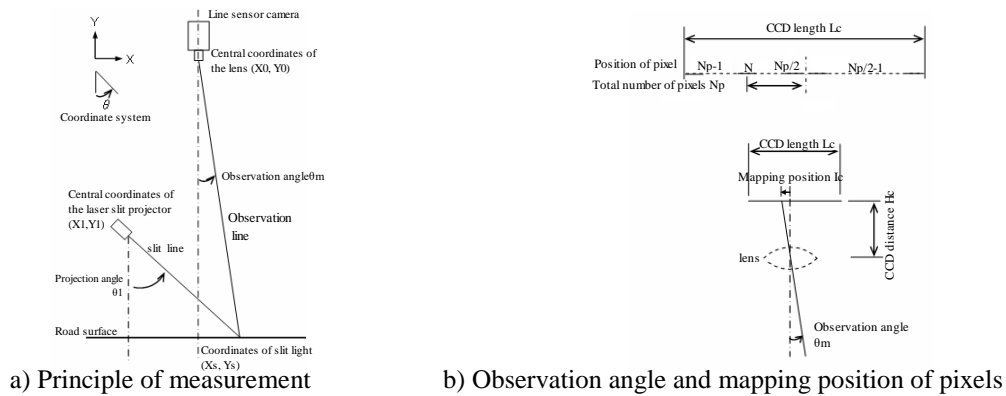


Figure 2: Principle of rutting measurement

High resolution is achieved by using a CCD that features as many as 5,150 pixels. Mechanical errors and optical strain of the lens are corrected by the measuring control unit and host computer, and excellent overall precision of ± 2 mm is achieved.

In order to measure rutting, a measuring range of about ± 100 mm was considered necessary to deal with the unevenness of the road surface and vibration of the vehicle. Such a large measuring range causes light beams projected at 100-mm intervals to interfere with the adjacent beams. The system manages to achieve a measuring range of ± 100 mm while retaining the small cross-sectional measuring intervals by emitting alternate beams of red (690 nm) and infrared (780 nm) light and detecting the beams using line sensor cameras with optical filters installed that preclude one of the wavelengths. (Photo 2)



Photo 2: Exterior view of the vehicle in measurement mode
Note: Only the 26 red light beams are visible (690 nm @ 200 mm)

The system can determine the vertical coordinates of the road surface across the traveling direction independently from the traveling speed by sampling 54 light beams simultaneously, which minimizes the effects of vehicular vibration while traveling at high speeds.

2.3 Longitudinal and cross-sectional measuring technology using GPS and IMU

The basic technology for this measuring system is kinematic GPS under the interferometric positioning method, which involves conducting measurements at unknown observation points using a traveling vehicle, receiving signals from GPS satellites based on known fixed electronic datum points, movable ground stations and virtual reference stations, and correcting errors using the phases of the carrier waves. Moreover, the inertial measurement unit (IMU), which consists of a triaxial optical gyro and three silicon accelerometers, precisely records the position and inclination of the vehicle at 200 Hz. The post-processing system combines the resultant longitudinal and cross-sectional measurements, enabling the road surface to be precisely assessed in three dimensions (at 0.5-meter intervals longitudinally and 0.1-meter intervals across the road) (Figure 3).

A prominent characteristic of the system is that the IMU technology enables the road surface to be measured even in areas where GPS satellite signals are weak, such as in steeply sided sections, near tall noise barriers, and underneath overhead bridges, as well as in places devoid of signals, such as in tunnels.

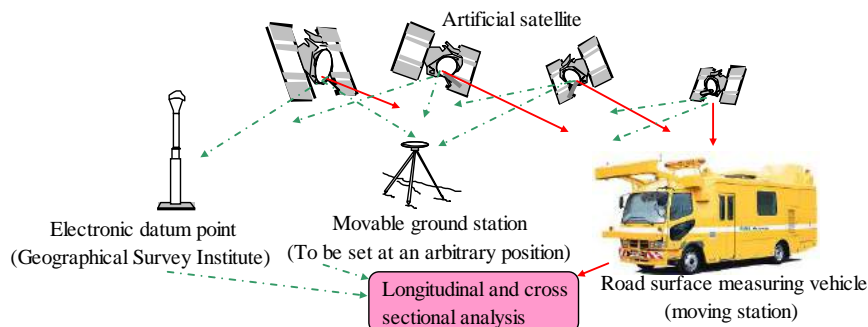
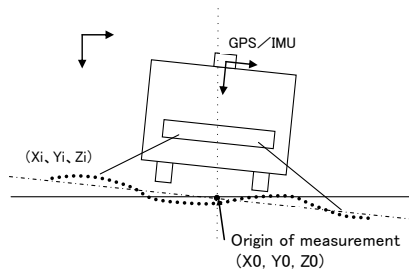


Figure 3: Kinematic GPS measurement using the road surface-measuring vehicle

Coordinates are determined by calculating the origin of measurement (X_0, Y_0, Z_0) from the position of the IMU in relation to the vehicle axis, which is determined from the position of the unit and the posture data of the vehicle (pitching, rolling and direction), adding the calculated rutting data ($0, Y_n, Z_n$) to the origin, and computing the position of each laser beam (X_i, Y_i, Z_i). From this position (X_i, Y_i, Z_i) and the data of the posture of the vehicle, the coordinates are converted into those of the North-East-Down coordinate system* to calculate the absolute coordinates of the points. (three-dimensional data link; Figure 4)



*North-East-Down coordinate system

North axis: Line that passes through the origin, is on a plane parallel to the tangent plane to a reference ellipse at the geographical position of the origin, and is positive towards true north

East axis: Line on the same plane as that of the North axis and is positive toward the east

Down axis: Line that is positive vertically downward to the NE plane

Figure 4: Schematic diagram of three-dimensional data link

The kinematic method is believed to be accurate to within about 2 to 3 cm. However, this measurement system achieves vertical errors caused by relative displacement of only 11 mm on average with the post-processing system and IMU correcting the GPS data and thus improving precision. The accuracy has been confirmed on expressways in service. This degree of precision is likely to be adequate to understand changes in the three-dimensional profile of road surfaces from the time of completion, but errors are further minimized by installing temporary benchmarks.

4. Conclusions

The three-dimensional road surface measurement system on a new measurement vehicle can measure and assess the present longitudinal and cross-sectional profiles of the road surface, and the measurements can be used to improve designs (profile modification) when restoring road surfaces that have been deformed by settlement of weak ground or earthquakes. The system is a completely new technology for measuring and designing road surfaces as it not only measures changes in road surface conditions, but can also use the measured coordinates as design data – unlike conventional methods, which only compare road surface data, such as rutting, cracks and IRI, with standard values.

The system has numerous advantages over leveling, which requires lane restrictions, including: 1) ensuring safety while taking measurements, 2) maintaining traffic convenience by not restricting lanes while reducing the risk of congestion and traffic accidents, 3) improving the efficiency and reducing the costs of taking measurements by enabling long sections to be measured quickly, 4) facilitating the work involved when ordering improvement projects, and 5) increasing the precision of road maintenance technologies through comprehensive assessment with conventional road control indices.

The system has already been developed and is in use on actual roads. With the original design modification aid system, the system can estimate future ground surface conditions from its measurements and calculate the costs for different profile designs freely. It has been used to measure the profiles of some 5,710 km of roadway, and design surface modifications for 262 sections.

The system still has several points to be improved, such as: 1) ranges outside the measurement width of 5.2 m need to be designed using estimated values, 2) it is difficult to measure sections at low speeds, such as at ramps, due to the properties of the measuring instruments, and 3) up and down lanes cannot be designed as a single profile due to measurement properties. These topics will be actively investigated.

In addition, there are not the operation results out of Japan to date, but the operation in foreign countries wants to let you accelerate the technology development that you put in the field of vision in future.

Innovative improvement to restrooms in rest areas

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1. INTRODUCTION

This paper describes a program of restroom cleaning on expressways managed by Central Nippon Highway Maintenance Nagoya (hereafter, “Central NEXCO HMN”). The program described herein comprises the following five points. This five-point program changed the nature of this work from simply cleaning to cleaning that provides hospitality to the customers. As part of this program, we changed the term used for “cleaning staff” to “area cast” and created a high-motivation workplace where staff can enjoy working, and we have succeeded in making expressway restrooms clean and attractive like those in a hotel.

- 1: Changing from conventional cleaning with water and detergent to cleaning with electrolyzed water
- 2: Use of a gum removal machine and floor washing machine
- 3: Hospitality provided to customers by the area cast
- 4: Improving the hospitality level through makeup training and customer service training
- 5: Restroom diagnosis, use of a cleaning manual, and use of a cleanliness level scoring system
- 6: Conclusion

1: Cleaning with electrolyzed water

The conventional way of cleaning expressway restrooms involved manual scrubbing using detergent, rinsing with large amounts of water, and allowing everything to dry naturally (Figure 1). This method had the following problems.



Figure 1: Conventional cleaning method

Problems

- (1) Because the entire floor became wet, there were instances when customers' trousers became dirty, and accidents when customers slipped and fell also occurred.
- (2) Because time was required for cleaning, restrooms were often unavailable when buses or tour groups arrived.
- (3) Because large amounts of detergent and water were used, the environmental impact was large.
- (4) Detergent foam and water puddles would remain behind after cleaning, and required much work for removal.

In order to improve customer service and lessen the workloads on area cast, Central NEXCO HMN introduced a new cleaning method using electrolyzed water.

Electrolyzed water is tap water that has been broken down by salt and electricity to produce two types of water: acidic electrolyzed water and alkaline electrolyzed water (Figure 2). Although there are three major methods of producing electrolyzed water, Central NEXCO HMN uses the "3-chamber electrolyzer with diaphragms" method. (Figure 3)

The 1-chamber electrolyzer method is not used because it requires dilute sulfuric acid in order to produce the electrolyzed water, posing a risk to the staff who produce it. The 2-chamber electrolyzer method is not used because it produces electrolyzed water with a high salt content that can corrode drainage pipes and other metal fixtures. The 3-chamber electrolyzer method was selected because the electrolyzed water does not contain salt and because it is safe, since the only materials required to produce it are salt and water.



Figure 2: Production of alkaline water and acidic water

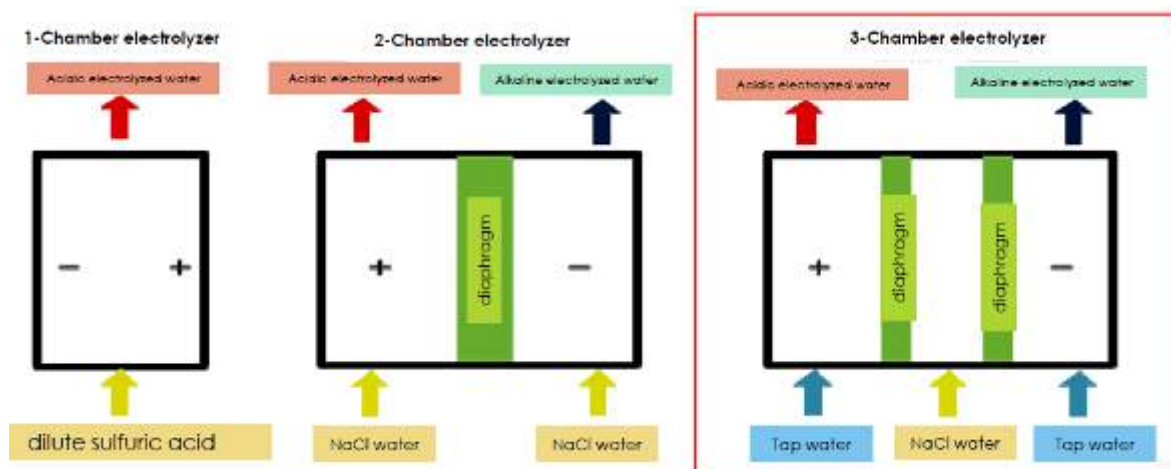


Figure 3: Formula for the 3-chamber electrolyzer production process

Central NEXCO HMN introduced the use of electrolyzed water in 2011, and now uses it for the restrooms of nearly all rest areas (92 locations).

Electrolyzed water is water that has been separated into acidic electrolyzed water and alkaline electrolyzed water, each with unique characteristics.

The characteristics of acidic electrolyzed water include high disinfecting and deodorizing effects. A test comparing the disinfecting effects of acidic water and the detergent previously used showed that the remaining bacteria count was 80% lower with acidic water than when detergent was used. Because of the powerful disinfecting effects, the deodorizing effects of acidic water are also high and pouring it into drainage pipes can prevent odors. The acidic water is hypochlorous acid, and is effective in disinfecting pathogens such as E. coli, salmonella, and norovirus. Consequently it can contribute to preserving the health of users and the area cast. (Figure 4) (Figure 5)

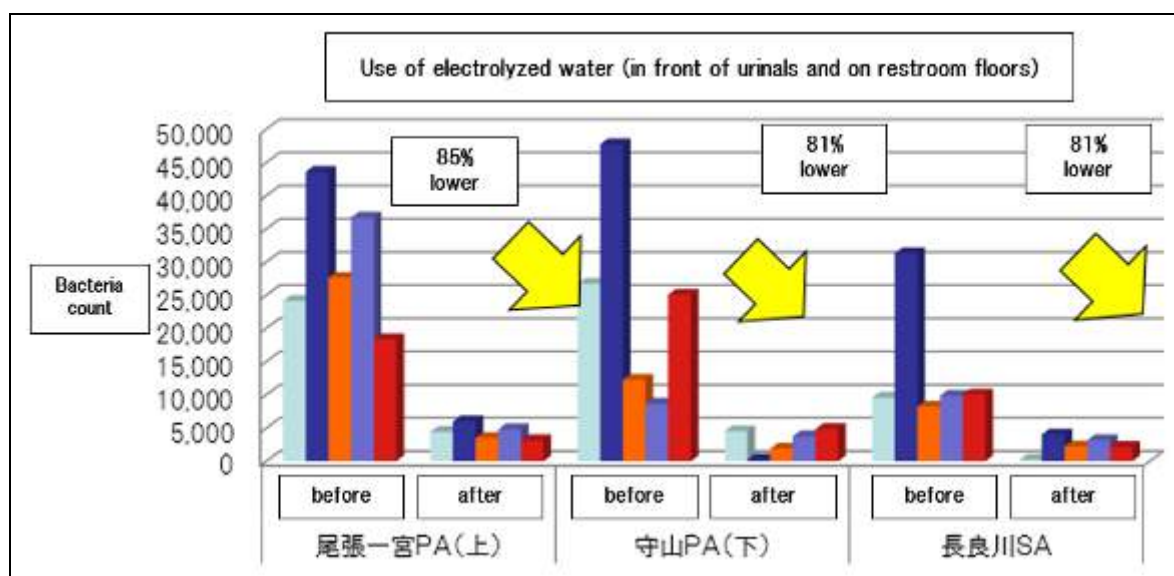


Figure 4: Graph of bacteria count reductions after the use of electrolyzed water

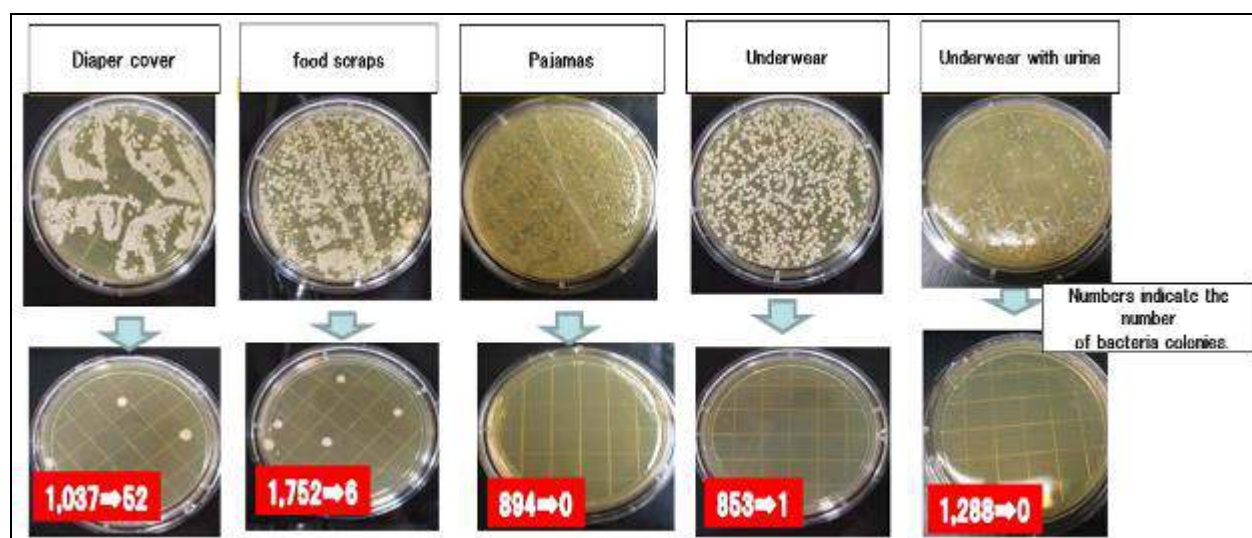


Figure 5: Sterilizing effects of electrolyzed water

Alkaline electrolyzed water is characterized by high cleaning effects. It has the same effect as soap on the oil spots that frequently occur in places where people walk on the floor, and on the protein stains that frequently occur in places where people touch. However unlike soap, the slipperiness can be eliminated simply by wiping it away, reducing the time required for cleaning. (Figure 6)

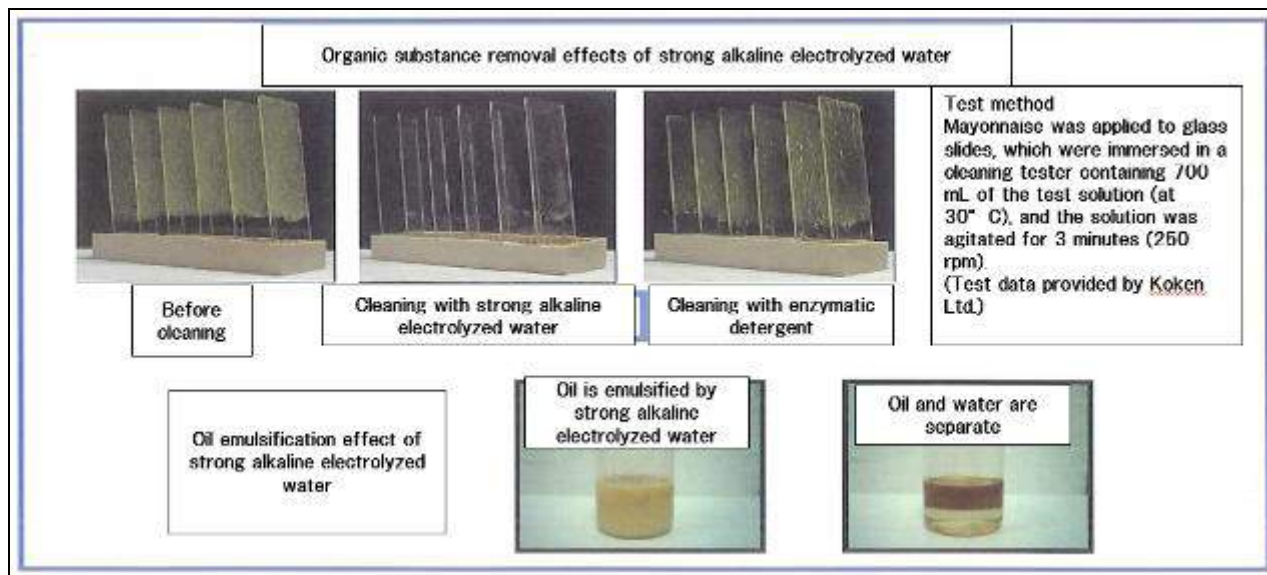


Figure 6: Oil removal test using alkaline water

Central NEXCO HMN uses an electrolyzed water mixture of the two types of water. The ratio of acidic electrolyzed water to alkaline electrolyzed water is 1:1. Even when mixed together, the water does not become neutralized for approximately 8 hours, and each water type retains its individual characteristics. The water is mixed just before the start of cleaning, and the electrolyzed water mixture is used in place of detergent as water with high disinfecting and cleaning abilities.

The acidity is around pH 3, and the alkalinity is around pH 11 – approximately the same as commercially available detergents.

The use of electrolyzed water mixture has resulted in a change from conventional wet cleaning with detergent to dry cleaning with electrolyzed water. As a result, the acidic water reduces the growth of bacteria and eliminates odors, while the alkaline electrolyzed water removes accumulated dirt and stains. It also shortens work times and reduces workloads.

The cleaning procedure is extremely simple, and involves filling a hand-held sprayer with electrolyzed water and applying it to the floor and toilets, then wiping it off. This has reduced work times by approximately 60% compared with the conventional cleaning method. It has also increased the time that the restrooms are available for customer use, and allows staff work on locations which could not be fully cleaned before due to lack of time, increasing the cleaning quality. (Figure 7)

However because electrolyzed water is a strong acid or alkali, some caution is required when handling it. Rules require that gloves be worn in order to prevent skin irritation, and that work areas be ventilated. As long as these rules are observed, electrolyzed water can be used for long periods without any adverse health effects.



Figure 7: Dry cleaning with electrolyzed water (spraying and wiping off only)

2: Cleaning with a gum removal machine and floor washing machine

In order to improve cleaning efficiency, Central NEXCO HMN has introduced compact cleaning machines that can be operated easily by anyone. This has changed conventional manual scrubbing work using brushes to work that is mechanized and efficient. The machines were selected from the perspective of the female area cast who would use them, and the selected products require little force to operate. Replacement of consumable parts and charging can also be done easily. The use of electrolyzed water with the machines has also improved the level of tile cleanliness. (Figure 8)

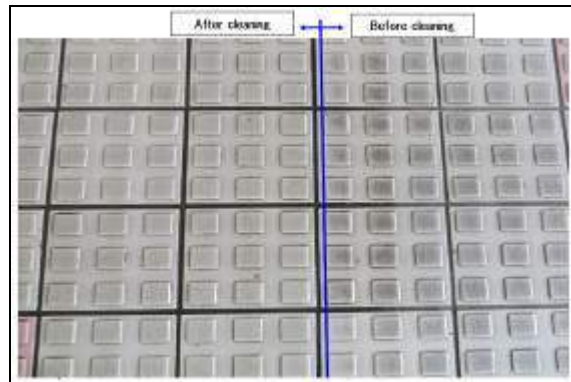


Figure 8: Cleaning with the machine

We also introduced a machine for efficient removal of discarded gum from rest area sidewalks. This machine discharges steam together with a special solvent that melts the gum, and then uses suction to pull off the melted gum. A single piece of gum stuck to the floor can be removed in 2 – 5 seconds. The machine is popular with the staff as it reduces the burden on the workers and shortens work time compared to the conventional work method of using a metal spatula. (Figure 9) (Figure 10)



Figure 9: Using the gum removal machine



Figure 10: Work with the gum removal machine (Left: Before work, Right: After work)

3: Hospitality provided to customers by the area cast

Central NEXCO HMN received positive evaluations from users and contracting agencies for the improved cleaning quality resulting from cleaning with electrolyzed water and the use of machines. As a result, this provided a motivation boost to the area cast, who began independently providing hospitality to the customers while cleaning. They began by placing seasonal flower decorations in the restrooms. At first, these were just small vases, but as customers commented on the beauty and relaxing effects of the flowers, this further increased the motivation of the cast. (Figure 11)

Later they began using pieces of wood from trees cut during construction and other supplies to make decorations at the entrances to the restrooms, and are working to provide hospitality that continues to delight the customers today. (Figure 12) (Figure 13)

Word of these efforts spread, and the area cast has been interviewed by newspaper companies and received observational visits from overseas organizations. (Figure 14)



Figure 11: Flower arrangements in the restrooms



Figure 12: Decorations using by-product materials



Figure 13: Decorations made from origami



Figure 14: Area cast being interviewed by a newspaper reporter

4: Improving the hospitality level through area cast makeup training and customer service training

The Central NEXCO HMN area cast now provides services to customers as they are cleaning. For this purpose, we introduced customer service training so that they can learn the correct manners for interacting with customers. Because more than 90% of the area cast members are women, we also included makeup training for them.

For customer service training, we conduct a training session each year and invite an instructor who previously conducted staff training at Disneyland. The training contents are not manual-based, but rather are intended to create area cast who can think independently and provide services from the customers' perspective on their own. This training was highly rated by the area cast who participated in it. (Figure 15)

They also learn the cleaning methods used at Disneyland, and learn how to provide hospitality to the customers while cleaning. In addition to the Disneyland instructor, we conduct manners training led by airline cabin attendants.

The makeup training for area cast has also been popular with the participants (Figure 16). For this training, we invite a lecturer from a cosmetics manufacturer to teach the participants about makeup manners for customer service, and about how to apply makeup that will stay in place even when they perspire in the summertime.

The area cast is composed primarily of older persons. The average age is 55 and the oldest member is 67. The majority of the area cast previously would work without makeup, but as opportunities for contact with customers increased, the number using makeup has also increased. This makeup training has been well received, with participants saying that it gives them confidence and is also useful in their private lives.



Figure 15: Training with a Disneyland instructor



Figure 16: Makeup training

5: Restroom diagnosis, use of a cleaning manual, and use of a cleanliness level scoring system

Improvements are made to the level of our restroom cleaning every year. However it is true that not every restroom is at the same level of cleanliness. Aiming to further improve these levels, the Central NEXCO HMN restrooms undergo inspections by restroom consultants who are certified based on the in-house certification system prescribed by the Japanese Ministry of Health, Labor and Welfare.

There are approximately 30 diagnostic items. Diagnosis is not only visual, and is also performed using instruments such as an odor meter and concentration meter. Locations such as inside drain pipes and behind equipment where dirt can adhere and cause odors are also checked. The diagnosis results are quantified and reported to the area cast of each restroom facility, with scores indicating places where cleaning was inadequate, and places that were cleaned well and worthy of praise. Because the restroom consultants evaluate all the restrooms based on the same inspection level and diagnostic items, this diagnosis has enabled Central NEXCO HMN to work for a consistent level of cleaning quality. (Figures 17, 18)



Figure 17: Diagnosis using a measuring instrument



Figure 18: Checking a drain for odor

The restroom consultants do more than just perform diagnosis. They also provide instruction in efficient cleaning methods that include methods of removing dirt that cannot be removed with electrolyzed water alone, cleaning methods that will not harm porcelain toilets, and how to polish mirrors and stainless steel products. Participants are also taught how to use a hand mirror to check places that are not directly visible in order to improve the level of cleanliness. (Figures 19, 20)



Figure 19: Restroom cleaning instruction



Figure 20: Checking for cleanliness with a hand mirror

A manual has also been created for new members of the area cast. It contains diagrams showing cleaning methods and locations that are likely to become dirty. Because the manual also includes customer service instruction as well as cleaning instruction, it can also be used for OJT of new employees. (Figures 21, 22)



Figure 21: Area cast manual (cleaning section)



Figure 22: Area cast manual (hospitality section)

5. Conclusion

Before the expressways were privatized around 10 years ago, the image of expressway restrooms was of dark and dirty places. The area cast who worked there also had little knowledge of cleaning, and the turnover rate of persons leaving the job was high. However the adoption of methods that make the facilities more comfortable to the customers, and methods for efficient cleaning, has resulted in customer expressions of gratitude and satisfaction reaching the area cast. This is a good example of employees who were able to improve the level of their own work when their work was praised by the customers. Now the expressway restrooms are featured in related pamphlets and other materials as model examples of clean restrooms. Warm-water washing toilet seats and disinfecting cleansers have been installed at all expressway restrooms, making them some of the top restrooms in Japan both in terms of cleanliness and equipment. As a result, supervisors of railway companies, park facilities, and theme parks have come to Central NEXCO HMN to learn about our cleaning methods. We are working to train area cast personnel and are studying more efficient cleaning methods, aiming for further improvements in the future.

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PAPER TITLE	Decision Support In Real-Time Traffic Management		
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KEYWORDS:

Include up to 5 keywords

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ABSTRACT:

Traffic managers know from experience what traffic is going to be like during a specific time and day of the week and can therefore handle incidents that occur regularly. They use real-time information from detector data, floating car data (FCD) and cameras. However, these data records only cover a certain part of the area. A daily challenge - because if something unusual happens or if several unprecedented incidents occur at once, traffic managers will be stressed quickly.

That is why they need support. Cities and regions are therefore increasingly making use of decision support tools for traffic management, which support traffic state prediction and scenario evaluation. In this paper we want to discuss different approaches for this task (mostly statistical vs. simulation-based), the role of big data and traffic demand and showcase a real-world application.

Decision Support in Real-Time Traffic Management

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1. INTRODUCTION

Around 50 per cent of all Germans have a smartphone (Statista 2015). Three quarters of those access Internet content on the go and 50 per cent use apps for different aspects of day-to-day life: and these numbers are rising. Transport-related applications are in the top five of the most-used apps (ARD/ZDF 2014). This includes the latest traffic information, navigation and route planning tools and timetable information. The current availability of real-time data is increasing people's expectations. "Real-time" is no longer just a buzzword but is part of the here and now, and modern traffic management can no longer shy away from it. But how can data streams be processed smartly, what are the limits to big data and how can demand modelling fill this gaping void? Worldwide, 2.5 Exabyte (or 2.5 billion bytes) of electronic data are produced per day. It is hard to imagine, but entirely true, that 90 per cent of all electronic data has been produced in the past two years (Barnes et al. 2014). And this statement will remain true in the future as the numbers of data sources and the volumes of information they generate are set to grow exponentially. Data on mobility behaviour make up a portion of this: With each piece of information on timetables and each route planning query, travellers disclose information on their mobility behaviour and contribute to the rise in data volume. And yet today this information is still often not available to transport managers. Their real-time information is fed from data from detectors, floating car data (FCD), automatic number plate recognition systems (ANPR) and accident- and road works reports. This is how they obtain an overview of the current traffic situation of the observed area and can react to regular disruptions thanks to their many years of experience.

2. METHODOLOGY

STATISTICAL VERSUS MODEL-BASED

Thanks to traffic forecasts, the transport manager is getting more and more room for manoeuvre for choosing the best measure to take. Today there are two main distinct approaches in traffic forecasting: the statistical and the model-based approach (FGSV 2012) (see Table 1). The statistical approach uses interpolation, inference, data collection, artificial intelligence and mathematical models to compare observed time periods with historical patterns. The traffic flow and speed variables are analysed and predicted without explaining and reproducing the underlying phenomena, i.e. the interaction between the vehicles and the driver behaviour. Statistical techniques are suited to projecting low-volatility traffic measures or recurring traffic patterns. They come up against their natural limits, however, if there is not enough historical data from sufficiently similar situations. This happens in particular with unusual situations, when accidents occur or road works are set up.

If someone wishes to react to these situations in real-time, model-based approaches can be used for the forecasting.

Table 1. Statistical and model- based approach in direct comparison

Objective	Method		
	Observed Data	Statistical Approach	Model-based Approach
Traffic estimation “What is going on?”	Maybe, with extensive measures	Yes	Yes
Traffic forecast “What is going to happen?”	No	Only “usual” conditions	Yes
Scenario evaluation & decision support “What would happen if?” “How should we act?”	No	No	Yes

MODEL-BASED TRAFFIC-MANAGEMENT IN REAL-TIME

In contrast to statistical processes, the model-based simulation approach relies on a physical interpretation of the traffic network and conditions. This is added to the supply through an explicit simulation of the interplay between travel demand and the transport network. model-based solutions such as PTV Optima, which can output dynamic forecasts for a time horizon of up to 60 minutes, combine for proven off- line traffic modelling with real-time data and algorithms.

On this basis, a traffic model created in the traffic planning software PTV Visum, for example, shows the “typical” day (e.g. workdays or week- ends) in the transport area under consideration. The transport supply and demand are then represented in the form of demand matrices. Dynamic traffic assignments calculate the time-dependent volumes and turn proportions on the network from the travel demand (for further information see Bellei et al. 2005, Gentile et al. 2005, Gentile et al. 2007 or PTV Group 2015). PTV Visum passes all this information on to PTV Optima (Figure 1).

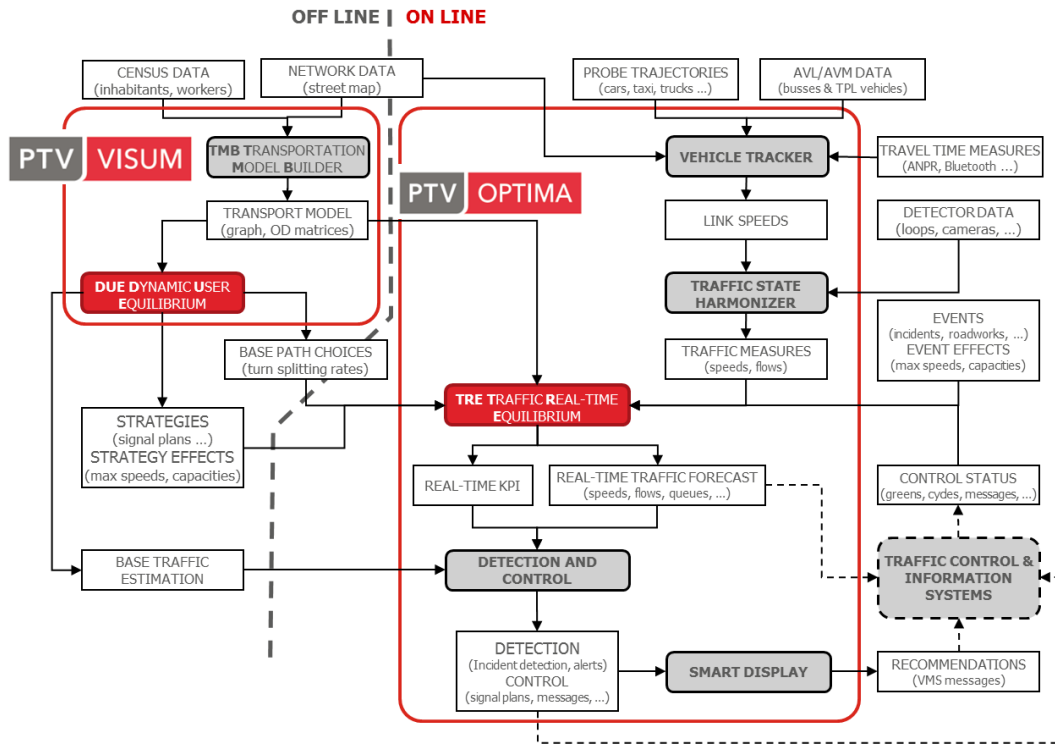


Figure 1. Logical architecture for model-based traffic management.

In PTV Optima, the online data therefore comes into play. The data is used in real time in PTV Optima to adjust base model capacities, speeds or volumes from PTV Visum locally to the latest circumstances. As PTV Optima explicitly takes into account the network structure, traffic flow dynamics and travellers' route choice behaviour, it also reproduces the traffic situation for links on which no detectors have been installed (spatial distribution) and can furthermore predict the effects of planned and unexpected results (temporal distribution) and then evaluate and compare different strategic measures.

OFFLINE METHODOLOGY

The offline part is devoted to the construction, calibration and update of the transport model, and to provide, by means of a dynamic assignment model, base estimations and forecasts of traffic behavior and travel time conditions for typical non-disrupted days and/or for possible incident patterns.

Once the model and the base traffic conditions are available, the online part will use the latter combining them with the measures coming continuously from the field into a real-time traffic model, that will basically adjust the traffic estimation and forecast to the measured traffic conditions of that particular day. The online part is completely automatic and runs "continuously": that is, a new traffic estimation and forecast is produced every few minutes.

The base transport model relies on a PTV Visum transport model either new or already existing. This base transport model is firstly calibrated and utilized off-line with the Traffic Real-time Equilibrium (TRE) add-in of PTV Visum in order to calculate a base traffic estimation. This is typically done for different day types, that are an a priori estimation of link flows, queues, travel times that reproduce average observed traffic conditions for different typical days as well as base path choices, in the form of turn splitting rates at nodes and represents how the travel demand will distribute on the different network for the same day type. Within operational planning activities, TRE allows also to assess possible control and management strategies, which can then be activated in real-time.

TRAFFIC MANAGEMENT SCENARIOS

When disturbances are modelled into the forecasts on the network, it is necessary to react and rectify the situation. Cities and regions have different objectives in this. Transport for London (TfL), for example, measures its success by the reliability of the transport network (Transport for London 2012). PTV Optima is flexible in its use of Key Performance Indicators (KPI) which deliver aggregated information on the overall network status in addition to the graphical feedback, and displays the basis for a decision that can be assessed quickly. The KPIs can be adjusted flexibly and it makes no difference whether the user has set the avoidance of traffic, the minimising of travel time or the reduction of negative effects of planned or unpredictable events as the main criteria. Typical tools for reacting to this include traffic light signals, the unblocking of traffic lanes, variable message signs and traffic information reported on the radio and Internet.

To resolve traffic jams, these tools are often used directly “on the living object”. Through their experience, experienced employees in traffic management offices can of course already now choose effective strategies. However, this way makes it impossible ever to find out whether a different strategy might not have been better. Seen from this angle, is it not better to test these and their alternatives in a virtual world? offline, in PTV Visum for example, strategies which have been developed can be introduced to the PTV Optima online environment, assessed, compared and ranked on the basis of the current traffic situation, before they are then sent onto the streets. PTV Optima is therefore capable of calculating many combinations of strategies simultaneously within a few minutes, including for large networks. This means that the best possible strategy can be applied as soon as possible. Using this process, the transport manager builds up his wealth of experience with unforeseen circumstances, develops confidence about the different scenarios and can choose from the optimal measures recommended by PTV Optima so that the traffic on the road network returns to “normal”.

ONLINE METHODOLOGY

Within the online part, real-time traffic data, representing measures of the road network state, are collected in the form of:

- probe trajectories and Automatic Vehicle Location data, coming from single vehicles, commercial fleets or public bus fleet
- travel time measures, derived from FCD trajectories from probe vehicles or comparable data sources like e.g. Automatic Number Plate Recognition (ANPR) systems
- detector data, that is flow and occupancy measurements from detectors on links

The first two data sources are associated in real-time to the link graph by the Vehicle Tracker (Figure 1), by means of an on-line map-matching procedure, and will calculate observed average link speeds.

The Traffic State Harmonizer (Figure 1) will complete the traffic data pre-processing, merging all observed measures (i.e. both traffic counts and FCD-derived speeds) into real-time traffic measures of vehicular flows, speeds, densities and capacities on the links of the road graph.

In parallel to traffic data, also planned and unplanned events, such as road-works, congestions alerts, accidents, but also current status of control devices like signal green and cycle times, are imported continuously into the system, and their effect on the road network in terms of temporary speed and capacity reductions is automatically calculated. Then, the simulation TRE is used in its real-time configuration, within a Rolling Horizon Dynamic Traffic Assignment procedure, the base transport model with the traffic measures and event effects in order to produce every few minutes a new real-time traffic estimation and forecast, that is the flows, queues and travel times on all network links, as well as their evolution over time in the next future.

Additionally, TRE is able to reproduce the effect of management scenarios, that is traffic management strategies (e.g. different signal programs or activations of variable message signs), isolated or combined together to represent complex scenarios. These strategies need to be prepared a priori, like the base model, but can be combined in real-time in any combination and simulated in parallel to the base simulation as additional scenarios. The comparison of the results of the different scenarios is assisted by the calculation of Key Performance Indicators (KPI) like e.g. the total travel time in the network.

TRE in its real-time configuration is aimed at reproducing as closely as possible the real operation of the whole network in terms of current traffic state (density, speed and capacity of each link) and of its evolution over time, producing every few minutes the estimation and forecast of the actual traffic flow condition on the network.

To this end, TRE exploits an advanced methodology for modelling transport demand and vehicle congestion, through which the behavior of drivers and the propagation of queues are explicitly represented. The connection to current traffic conditions is guaranteed by field measures at discrete points (loop detectors, speed radars, video cameras, probe vehicles) that are collected in real-time and sent to TRE, which is able to reconstruct the current and future traffic pattern on the entire network, hence extending the available data in time and space to provide useful information for driver navigation and transport optimization.

The modelling paradigm adopted within TRE, primarily based on the physical interpretation of the traffic phenomena, differs substantially from the mere interpolation of field measures through artificial intelligence methods. Most monitoring systems apply, in fact, data mining techniques to match the current time-series with historical patterns, thus providing forecasts only on local and typical conditions. However, the statistical inference alone may not allow to deduce the traffic state of unmonitored links from the observed data or to forecast the consequences of unpredictable atypical events such as road accidents.

PTV Optima is thus specifically conceived for metropolitan contexts, as said in Meschini and Gentile (2010), where the congestion is strongest, while the day-to-day variability and the within-day fluctuation of vehicle flows and travel times is not negligible; but it can also be installed in extra-urban frameworks, that are less complex by their nature.

The mathematical model underlying TRE is based on an explicit representation of traffic phenomena, with particular reference to flow and congestion propagation. In particular, this method adopts as its simulation engine the GLTM (Gentile 2008), which is a macroscopic dynamic network loading model based on the Simplified Kinematic Wave Theory. Key features of the GLTM are: the possibility to adopt a fundamental diagram with general shape; complete representation of general intersections, even signalized; no need for spatial discretization of links (contrary, for example, to the Cell Transmission Model). Thus, the proposed modelling approach is opposed to micro-simulation, in which individual vehicles are operated as separate elements. Therefore, the GLTM is computationally a lot faster than microscopic simulations - this in turn allows the simulation of larger or more detailed networks.

In order to obtain a continuous update of the traffic forecast, the GLTM is sequentially applied with a rolling horizon schema, exploiting both the Base Transport Model and the traffic measures and events gathered from monitored links.

Specifically, a continuously updated traffic flow forecast is achieved by performing a sequence of real-time dynamic traffic propagations over the network in rolling horizon. In order to correctly implement the rolling horizon context, each simulation will not start from empty network initial conditions, but instead will adopt as initial conditions the traffic state calculated by the previous simulation in correspondence of its initial instant; this way, the congestion situation will be “transmitted” from one simulation to the next one. Making sure that previously calculated queues and/or measure-derived variations are inherited.

This concept is schematically depicted in the example provided in Figure 2: within this example, a new simulation is launched every 5 min; each simulation has a simulation horizon of 35 min, spanning with respect to the launch time, from 5 min in the past (this way the current simulation takes into account all the measures and events arrived to the system between the launch time of the previous simulation and the launch time of the current simulation) to 30 min in the future (in order to allow producing e.g. 15 and 30 min forecasts).

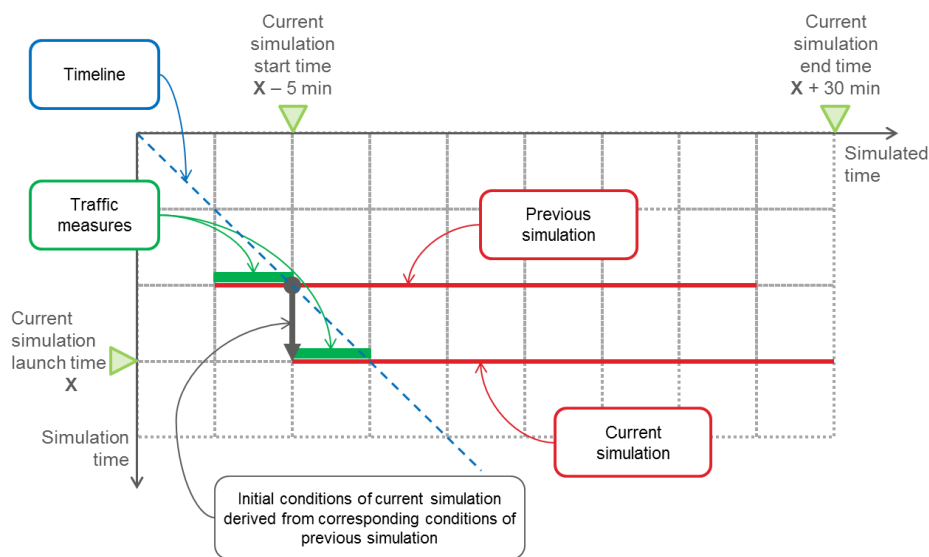


Figure 2. Example of a simulation timeline.

3. APPLICATION

ITS Vienna Region is the joint traffic management project of the three Austrian provinces Vienna, Lower Austria and Burgenland. After a very good experience with the old PTV product “Traffic Platform” the ITS Vienna Region decided to go for an update with the brand new product for traffic monitoring and traffic forecast: PTV Optima.

THE MODEL

The base of this system in Vienna was a model with dynamic assignment built by PTV starting from the static model already existing for the old platform. The model covers an area of approximately 27,000 km (Vienna region). The selected network is a part of the national unified network GIP and was built in PTV Visum using this reference network.

The total number of covered street links is 149,678 while the simulation runs on a simplified network (composed by all meaningful and major roads) of about 50,000 links. The study area was divided in 1,096 zones and the available historical data were used to build hourly O/D matrices for 3 typical days. Demand was corrected by Fuzzy matrix correction methodology (PTV Visum 2015). A Dynamic user equilibrium (using the TRE add-in available in PTV Visum) was calculated offline until the typical flow and speed curves were deemed to be satisfying with client requirements.

THE ITS INFRASTRUCTURE

ASFINAG, ÖBB, Wiener Linien (public transport operator in Vienna), the police, the traffic information center of Austrian broadcaster Ö3, Carsharing and Citybike Wien and all services within the public administration are all data partners of ITS Vienna Region (Figure 3). Over 3,500 taxis belonging to the taxi companies continually send in their GPS data, including information on position as well as speed, which is used to draw up the real-time traffic report. All the available data (mainly data from 780 loop detectors giving flow and speed, taxi trajectories and events generated by the third part systems mentioned above) are given in real time to the PTV Optima system.

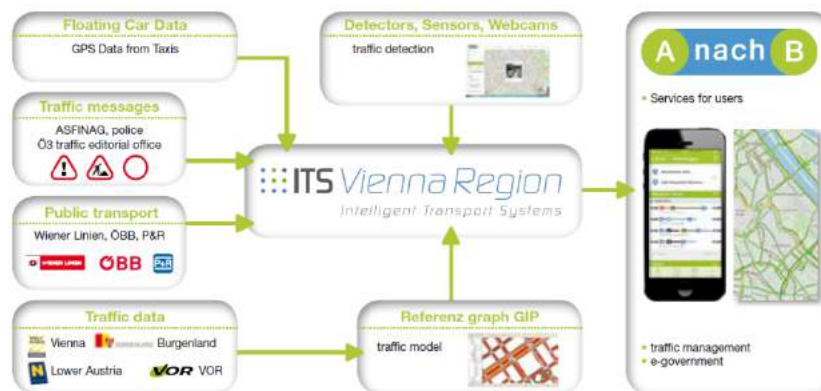


Figure 3. ITS Vienna Region project partners and schematic data flow (ITS Vienna Region 2016).

DATA FUSION

The Data Fusion engine processes the traffic state data in order to normalize them and to filter what is unreliable. In some case detectors can deliver only partial lane information, and thus PTV Optima fills the remaining lanes with a similar value, other times only taxi trajectories are available and thus speed is reconstructed according to consolidated map-matching algorithms (see Marchal et. al 2004). Unreliable (like unrealistic flow drops or flow raises) values are filtered. In case of multiple available data sources on the street link the system is able to weight the sources also according the number of samples (e.g. FCD speed data are more reliable whenever more cars are passing through the observed link).

Events are forwarded to PTV Optima by partners like ASFINAG using the well-known DATEX2 (see www.datex2.eu) European standard to exchange traffic information. PTV Optima then translates automatically the impact of any events in terms of capacity or speed restrictions that will affect the running simulation. These events are then available not only to the simulation but to every system that uses PTV Optima results.

THE ONLINE SIMULATION

Simulation is automatically recalculated every 5 min, in case of necessity it can triggered manually. Runtime is about 90 s. Results forecast the next hour in time slices of 15 min.

RESULTS

- The Transition to the new system was according to the client successful, with full and seamlessly integration in the client IT architecture.
- Estimation and forecast available in term of speed/flow for every main link of Vienna region. These data are used by the client not only for traffic information but also for planning and offline traffic estimation.
- On a selection of 100 working detectors the accuracy of the model for the 15 min forecast was measured. The comparison has been made between the current estimated out-flow from the single detector (which reflects the real-time data that are put inside the models as the inflow value of that link) with respect to the predicted outflow calculated 15 min before. This showed that for 80% of that detectors the mean daily GEH is 5.19 which is specifically for a real-time application a very satisfying value (for GEH reference please refer to UK Highway Agency 1996).
- PTV Optima results feed the national VAO journey planner for private transport where they are used to obtain more efficient routes for drivers, taking into account FUTURE short-term formation of congestions and queues. The public website of the VAO project is <http://www.verkehrsauskunft.at>.

4. CONCLUSIONS AND OUTLOOK

Fusing Real-time models and transport algorithms with huge amounts of real time data is a modern way to analyze, monitor and predict traffic. In fact this allows to reproduce dynamically emerging queues, speed drops and thus react to planned and unplanned events. It has been demonstrated that starting from a dynamic model it is possible to build a complete suite of functionalities that can help cities to plan, monitor, forecast the traffic and to give the right advice to every driver at the right moment.

Travel demand is very often produced in the transport planning using a classic demand model. This covers the trip generation (how many routes are done for each trip purpose?), the trip distribution (what destination has been selected?) and the choice of transport (with which means of transport is the destination reached?). By modelling the decision-making behaviour in this way, the model can be made more sensitive to measures connected with typical transport planning issues (Schlaich & Heidl 2016, Friedrich 2011). For traffic management which emphasises private transport, on the other hand, purely empirical matrices can be used, such as those obtained with mobile data (Friedrich et al. 2011).

With the steady rise in the availability of real-time data, the demand matrix calculated in advance can be replaced in the long run by real-time demand from data sources which actually come about by optimising individuals' mobility (apps, navigation services, and so on). In this way, real-time data can detect not only the current traffic situation, but also the current destinations of travellers who are potentially deviating from their "typical" day. In this case too, a forecasting model is essential for predicting the traffic flow using fast assignment procedures. These are processes that are currently used in real-time traffic management.

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PAPER TITLE	RESONANT RUBBLIZATION AS A METHOD TO REHABILITATE CONCRETE PAVEMENT		
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AUTHOR (Capitalize Family Name)	POSITION	ORGANIZATION	COUNTRY
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KEYWORDS:

Resonant Rubblization
Quasco LLC

ABSTRACT:

Resonant rubblization is the leading technology for concrete rehabilitation and breaking. Utilizing the industry leading resonant beam technology, resonant machines are able to fracture the concrete at a 40-degree shear plane, creating an interlocking base material that allows the concrete to be recycled in place and used as the new base.

Resonant machines complete the rehabilitation of a roadway much faster than traditional methods, minimizing the cost and energy needed for roadway repair. The rubblization process requires half the time of traditional concrete replacement, thus significantly decreasing the disruption to the community.

The rubblization process using resonant machines is sustainable. Rubblization recycles the existing concrete in its place, turning it into a base and allowing for a new roadway surface to be directly overlaid. The process is “green” and greatly reduces the impact to the environment.

RESONANT RUBBLIZATION AS A METHOD TO REHABILITATE CONCRETE PAVEMENT

Richard Koch

ABSTRACT

Resonant rubblization is the leading technology for concrete rehabilitation and breaking. Utilizing the industry leading resonant beam technology, resonant machines are able to fracture the concrete at a 40-degree shear plane, creating an interlocking base material that allows the concrete to be recycled in place and used as the new base.

Resonant machines complete the rehabilitation of a roadway much faster than traditional methods, minimizing the cost and energy needed for roadway repair. The rubblization process requires half the time of traditional concrete replacement, thus significantly decreasing the disruption to the community

The rubblization process using resonant machines is sustainable. Rubblization recycles the existing concrete in its place, turning it into a base and allowing for a new roadway surface to be directly overlaid. The process is “green” and greatly reduces the impact to the environment.

INTRODUCTION

Portland Cement Concrete (PCC) has played a crucial role in road construction around the world. It is incredibly durable and has the ability to handle heavy traffic loads. However, PCC, like all materials, eventually deteriorates. The question numerous countries have considered is how to effectively and efficiently deal with aging roads once they are nearing the end of their life cycle. The purpose of this paper is to discuss resonant technology as applied to repairing concrete roads.

The use of PCC roads has expanded exponentially over the past few decades. From 1990, the portland cement concrete used in China grew from 11,373 kilometers to 199,000 kilometers in 2003 (Chen et al. 2004a). Roughly 700 million tons of cement was manufactured in China in 2002 alone. That amount is roughly half of the world’s output for that year (Chen et al. 2004b). China has tried numerous rehabilitation methods with varying success. In China, Full Depth Repairing (FDR) and under-sealing the voids are common for deteriorated slabs. This method, however, failed in many areas or adjacent slabs in less than three years (Chen et al. 2004c). It is no wonder that China has adopted the resonant beam technology to rehabilitate roads in numerous Provinces around the country. Quasco LLC has rubblized well over 6,000,000 square meters of concrete in China alone.

Various methods, including direct overlay, crack and seat, complete dig-out and removal, drop/multi-head hammers and rubblization have all been used to rehabilitate concrete roads nearing the end of their life span. A study by (Witzak and Rada 1992) included a total of 454 field projects in 34 US states. The study tested hot mix asphalt (HMA) overlays placed over various rehabilitation techniques. The techniques included crack and seat, break and seat and rubblization. Rubblization was used on 19 sections of the test, crack and seat was used on 250 sections, 150 sections used break and seat, and the remaining 35 sections did not have complete data available. Several United States Federal and State organizations were involved, including the National Asphalt Pavement Association (NAPA), Transportation Research Board (TRB), and the Asphalt Institute (AI). Tests were used on the sections including a visual test based on the pavement condition index (PCI) highway methodology. In addition, non-destructive testing (NDT) using a falling weight deflectometer (FWD) was used. After extensive analysis, it was concluded the rubblization process performed the best (Tim & Warren 2004).

All methods have advantages, but rubblization with resonant beam technology has proven to be an inexpensive, quick, and long-term solution to concrete road rehabilitation. A properly executed rubblization project can extend the life of a concrete road by over twenty years. The key is to choose rubblization at the optimal time in the life cycle of the road. In some cases, it may be determined the concrete is beyond repair

and must be completely removed. Failing to properly time the rehabilitation to a road can significantly raise the cost, regardless of the method chosen.

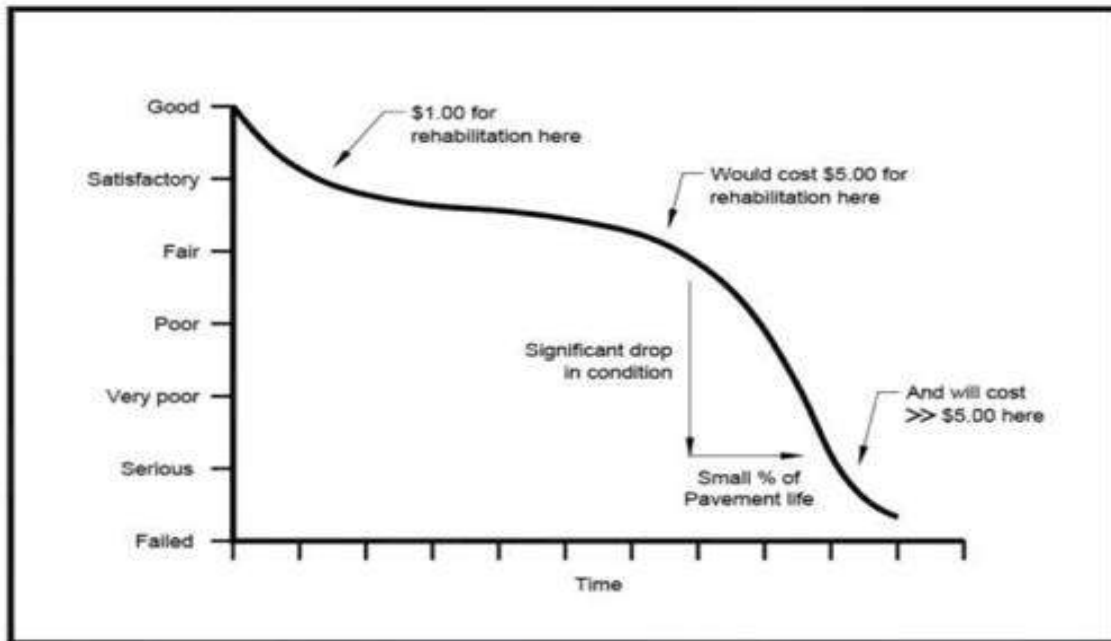


Figure 1. A graph displaying the ideal time to use rubblization on a concrete road nearing the end of its lifecycle.

RUBBLIZATION

Rubblization is the process of fracturing pavement of PCC into angular pieces for direct overlay. It destroys the integrity of existing concrete to allow for immediate overlay. Rubblization involves breaking an existing concrete slab in to various sized pieces, destroying any slab action, and overlaying with hot mix asphalt (APA 2002). Rubblization eliminates the problem of reflective cracking, since the technique is supposed to completely disintegrate the existing concrete slab (Recommendations 2013a). True rubblization, with resonant beam technology, does not damage or destroy the sub-base of roads. Unlike other methods, including the drop hammer and multi-head hammer, the resonant beam fractures the concrete on a shear plane, at a 40-degree angle. This method allows for an interlocking, angular fracturing pattern that provides for a greater modulus as opposed to a vertical fracturing pattern from drop or multi-head hammers. The angular fracturing pattern spreads the load over a larger area and allows for a greater modulus. The sizes of broken pieces usually range between 2 to 6 inches (Recommendations 2013b).

Rubblization with resonant beam technology was first used in the United States in 1986 as a method to rehabilitate concrete roads nearing the end of their life-cycle (Fitts 2006a). New York was the first state to adopt the technology. Since then, rubblization has been used in 37 US states, two Canadian Provinces, and many countries around the world (Fitts 2006b).

The resonant energy involved in rubblization originates from the resonant beam. The beam is attached to a pedestal and shoe that in unison strike the concrete 44 times per second (44Hz), at an amplitude of only 20 millimeters per strike (See Figure 2 on the following page). This high frequency, low amplitude impact dissipates all breaking energy by the time the fracture spreads through the slab. The process causes the bottom of the slab to remain smooth and does not drive broken concrete pieces into the base. Rubblization protects the integrity of the base as well as underground utilities. It also provides a base that is not only durable, but also interlocked for greater strength. A study by the North American Pavement Association

shows that the strength of a rubblized layer of concrete is 1.5 to 3 times stronger than a high-quality dense graded crushed stone base (NAPA 1994).

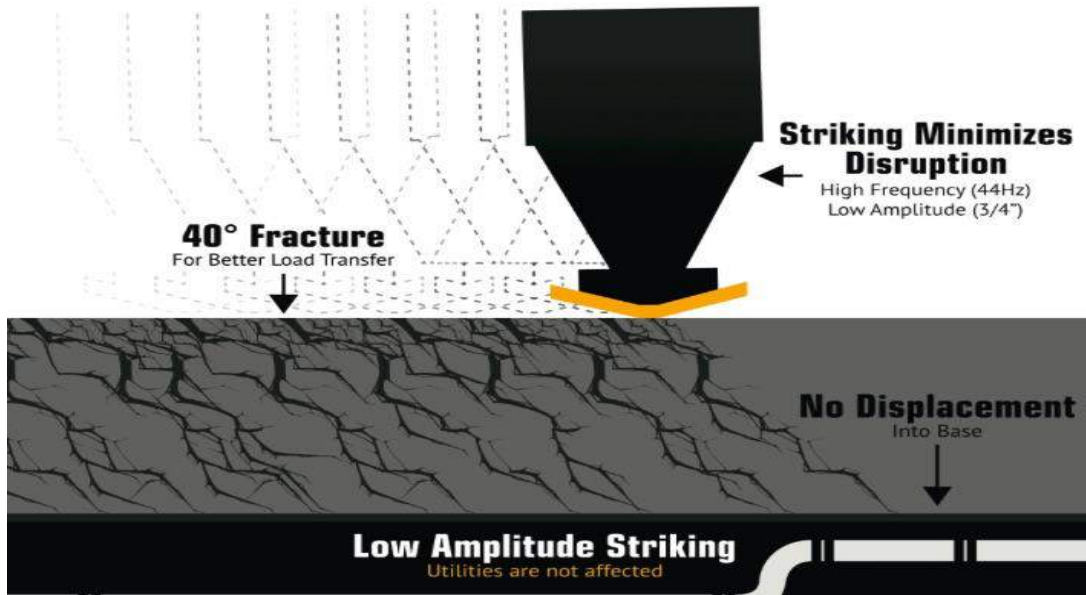


Figure 2. Graphic with the pedestal shown fracturing the concrete on a shear plane.

A demonstration of the entire rubblization process can be seen at <http://www.rubblization.com/process.html>.

Complete rehabilitation of a road section can be completed in one day. Once a project is selected, the area should be closed to traffic. It is recommended to begin rubblization on the free edge, or shoulder of the road and break towards the opposite edge or shoulder. If reinforcing steel bar is present, it will be completely de-bonded from the concrete once rubblization takes place as shown below.



Figure 3. De-bonded reinforcing bar after rubblization.

The resonant machine works best, and is most effective, when 500 linear meters or more of concrete is closed to traffic. This allows the machine to maximize each pass and provide the greatest amount of production in a given workday. Each resonant machine is capable of breaking one linear kilometer of two-lane road, or roughly 6,000 - 7,000 square meters in a given 8 hour workday.

Proper rehabilitation of PCC requires a high quality of rubblization and maintaining a strong base and/or subgrade soil. In order to achieve this process, a correctly installed drainage system is essential. Poor performance can occur when the underlying soils are saturated. Installation of edge drains prior to rubblization has proven to be successful for this type of condition (Chen et al. 2004d).

Water intrusion into pavement structures is a major factor in reducing the service life of a road. The principle drainage management procedures in the United States are still frequently based on the American Association of State Highway and Transport Officials (AASHTO) Guide for Design of Pavement Structures.

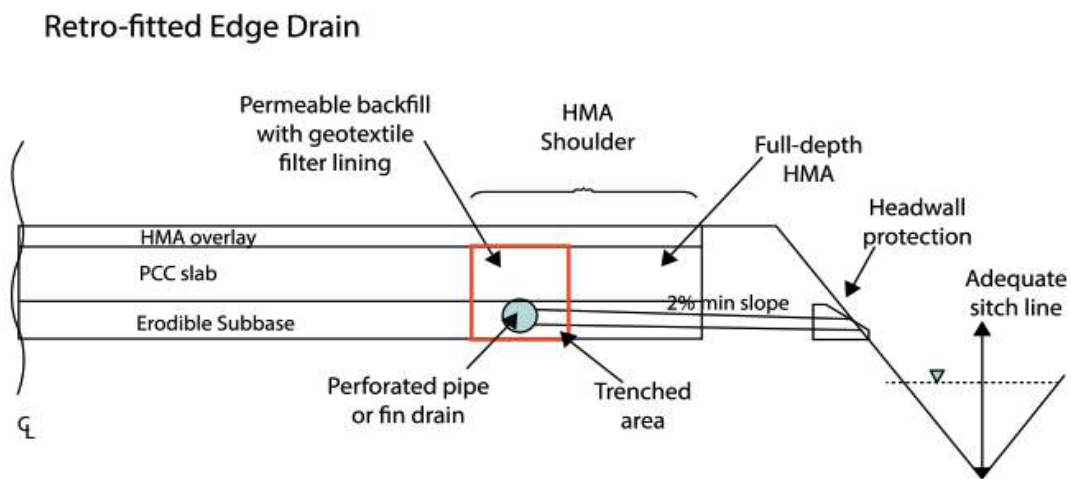


Figure 4. A diagram of a retrofitted edge drain design for a rubblized roadway.

The edge drain allows water to effectively drain away from under the slab. Proper drainage results in a dry, firm base and in consequence a higher modulus. Vibrations from traffic will allow previously trapped water to escape. This process will only work if the base is not penetrated by broken concrete. Each place the base is punctured is a potential water trap. Resonant technology allows the concrete slab to be fractured without piercing the base. Once proper drainage is established, rubblization can be effectively carried out.

A study in Colorado by the state's Department of Transportation tested rubblization with the resonant breaker as well as the multi-head hammer. Half of the project, roughly 39,361 square yards, was rubblized using the resonant breaker (LeForce, Robert, P.E. 2006a). After rubblization, the study noted there was no reflective cracking from the old concrete in the five years since construction, and no base-related distresses were seen on the project (LeForce, Robert, P.E. 2006b). The results of the study show the effectiveness of resonant technology. As stated in the report, the use of rubblization and overlay with hot mix asphalt should be incorporated into the Colorado Department of Transportation design manual (LeForce, Robert, P.E. 2006c).

Once rubblization is complete on a given section, two or three passes with a vibratory roller are all that is needed before paving of the road can begin. There is no need to apply a prime or tack coat before the HMA overlay as no detrimental effects could be identified (Chen et al. 2004e). Traffic should not be allowed on the rubblized concrete slab, due to the risk of "unseating" the particles of rubblized pavement (Decker & Hansen 2006a). If traffic is absolutely necessary, it should be kept at a minimum in order to maintain the integrity of the rubblized slab. It is recommended that the HMA overlay over a properly compacted

rubbleized PCC pavement occur within a 24-hour period from the compaction process (Decker & Hansen 2006b).

In most cases, a hot-mix asphalt overlay is used on the rubbleized concrete. HMA overlays have been used for decades and provide an inexpensive and long-term solution to pavement rehabilitation. Different countries have different standards when it comes to the depth of asphalt that is needed on a rubbleized road. A chart from the National Asphalt Paving Association (NAPA) shows the recommended overlay amounts based on a number of different factors.

Required HMA Overlay Thickness																	
Existing PCC slab thickness	Structural number subbase	Low Traffic				Medium Traffic				Heavy Traffic				Very Heavy Traffic			
Pavimento Rígido (PCC) CM	Total SNsub	Subgrade Soil Category				Subgrade Soil Category				Subgrade Soil Category				Subgrade Soil Category			
		Poor	Med	Good	Exc	Poor	Med	Good	Exc	Poor	Med	Good	Exc	Poor	Med	Good	Exc
20.32	0	17.78	12.70	12.70	12.70	22.86	16.51	15.24	15.24	26.67	20.32	17.78	17.78	35.56	29.21	21.59	20.32
20.32	0.4	17.78	12.70	12.70	12.70	22.86	16.51	15.24	15.24	26.67	20.32	17.78	17.78	35.56	27.94	20.32	20.32
20.32	0.8	17.78	12.70	12.70	12.70	22.86	15.24	15.24	15.24	26.67	17.78	17.78	17.78	35.56	26.67	20.32	20.32
20.32	1.2	16.51	12.70	12.70	12.70	20.32	15.24	15.24	15.24	24.13	17.78	17.78	17.78	34.29	24.13	20.32	20.32
20.32	1.6	13.97	12.70	12.70	12.70	17.78	15.24	15.24	15.24	22.86	17.78	17.78	17.78	33.02	22.86	20.32	20.32
20.32	2	12.70	12.70	12.70	12.70	15.24	15.24	15.24	15.24	20.32	17.78	17.78	17.78	30.48	20.32	20.32	20.32
22.86	0	15.24	12.70	12.70	12.70	19.05	15.24	15.24	15.24	24.13	17.78	17.78	17.78	33.02	26.67	20.32	20.32
22.86	0.4	15.24	12.70	12.70	12.70	19.05	15.24	15.24	15.24	24.13	17.78	17.78	17.78	33.02	25.40	20.32	20.32
22.86	0.8	15.24	12.70	12.70	12.70	19.05	15.24	15.24	15.24	24.13	17.78	17.78	17.78	33.02	24.13	20.32	20.32
22.86	1.2	12.70	12.70	12.70	12.70	16.51	15.24	15.24	15.24	21.59	17.78	17.78	17.78	31.75	21.59	20.32	20.32
22.86	1.6	12.70	12.70	12.70	12.70	15.24	15.24	15.24	15.24	20.32	17.78	17.78	17.78	30.48	20.32	20.32	20.32
22.86	2	12.70	12.70	12.70	12.70	15.24	15.24	15.24	15.24	17.78	17.78	17.78	17.78	27.94	20.32	20.32	20.32
25.4	0	12.70	12.70	12.70	12.70	16.51	15.24	15.24	15.24	21.59	17.78	17.78	17.78	30.48	24.13	20.32	20.32
25.4	0.4	12.70	12.70	12.70	12.70	16.51	15.24	15.24	15.24	21.59	17.78	17.78	17.78	30.48	24.13	20.32	20.32
25.4	0.8	12.70	12.70	12.70	12.70	16.51	15.24	15.24	15.24	21.59	17.78	17.78	17.78	30.48	21.59	20.32	20.32
25.4	1.2	12.70	12.70	12.70	12.70	15.24	15.24	15.24	15.24	19.05	17.78	17.78	17.78	29.21	20.32	20.32	20.32
25.4	1.6	12.70	12.70	12.70	12.70	15.24	15.24	15.24	15.24	17.78	17.78	17.78	17.78	27.94	20.32	20.32	20.32
25.4	2	12.70	12.70	12.70	12.70	15.24	15.24	15.24	12.70	17.78	17.78	17.78	17.78	25.40	20.32	20.32	20.32

Figure 5. Provides the specifications as set forth by the National Asphalt Paving Association. The depth is based on the existing thickness of the concrete, the structural number on the sub-base, and the amount of traffic.

The biggest obstacle to HMA overlays is reflective cracking. The key is to completely fracture the slab before adding the HMA overlay. Rubblization fractures the slab into fragmented interlocking pieces. This process destroys the existing slab and eliminates reflective cracking. Rubblization is the only approach that utilizes the existing concrete pavement and fully addresses slab movement responsible for reflective cracking (Recommendations 2013c).

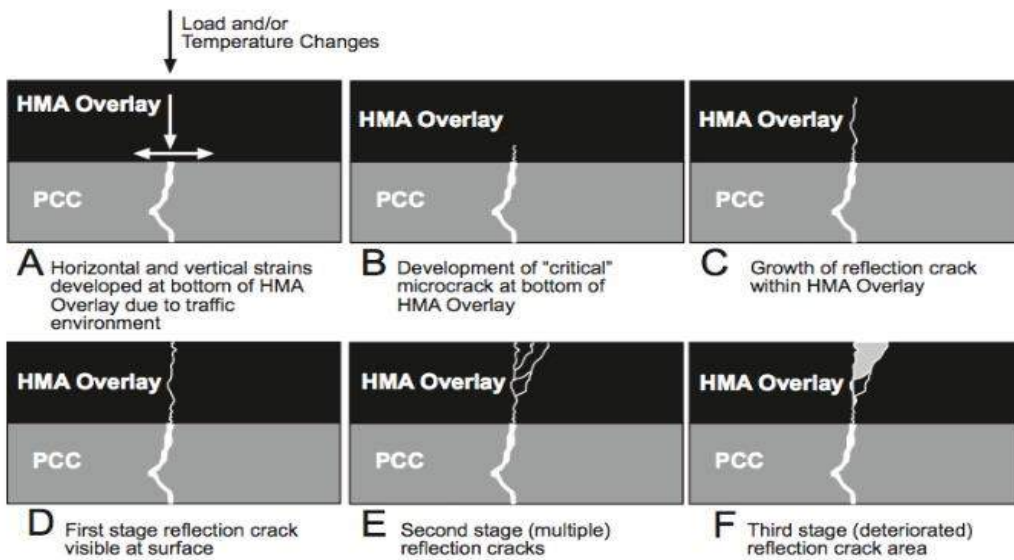


Figure 6. Demonstrates the reflective cracking issue as related to HMA overlays.

When the rubblized PCC modulus decreases by the slab being more intensely fractured, the likelihood of reflective cracking from the HMA overlay is greatly reduced. The structural capacity of the PCC decreases and requires a thicker HMA overlay. The ultimate goal is to reduce the rubblized slab modulus (EPCC) value to a minimum to eliminate reflective cracking, but not so low that the rubblized slab requires an excessive amount of HMA overlay.

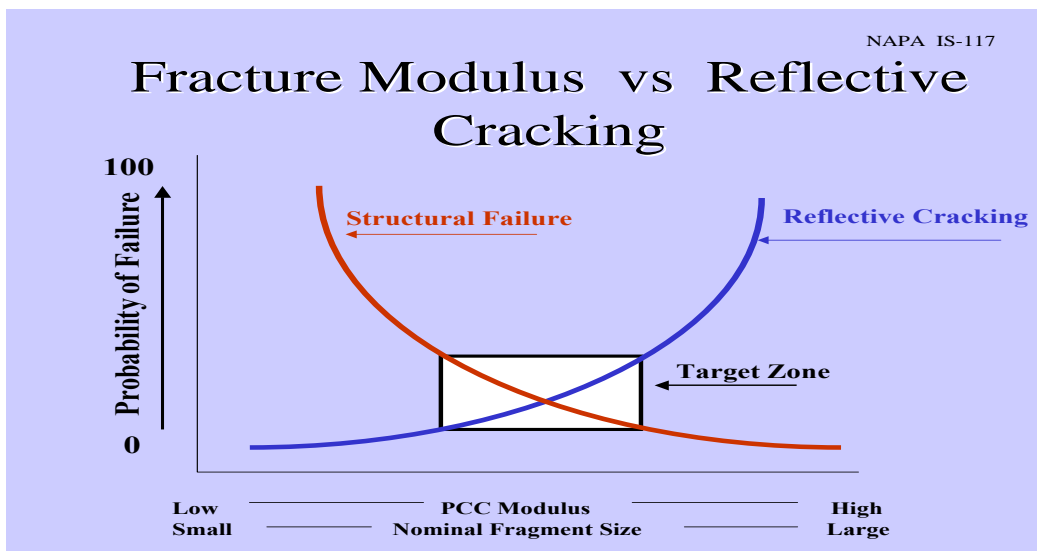


Figure 7. A graph from the National Asphalt Pavement Association shows the relationship of the rubblized slab modulus (EPCC) to the functional distress caused by reflective cracking and the structural requirements of the HMA overlay.

Resonant technology has consistently proven to retain the highest modulus, highest structural coefficient and an ability to distribute surface loading over a much wider base. The structural coefficient of a crushed stone base is typically .14 and a stabilized base is .25. A slab supported by sand or an unconsolidated base material

will produce a structural coefficient between .16 and .25. The structural coefficient of a rubblized slab ranges from .25 to .28. Rubblized pavement tested with a falling weight deflectometer (FWD) method have shown the strength of the rubblized layer to be 150% to 300% of the effectiveness in load base distribution compared to a high quality dense graded crushed stone base.

The strength of the rubblized PCC measured by a FWD in subsequent tests have improved year over year. The small particle size on the top layer of the rubblized concrete is constantly being driven deeper into the fracture lines on the rubblized slab. It was concluded that rubblization using a resonant breaker was not detrimental to the stiffness of the base, sub-base and subgrade. In addition, the modulus values would increase with time until they reach equilibrium values (Chen et al. 2004f).

The resonant machine will not damage underlying utilities. In a study conducted in China, it was concluded that the resonant machine yielded minimum disturbance during the rubblization process due to its low impact and high frequency operation. In fact, there were no complaints during rubblization from business owners and residents from Huqingping Highway and Kim Shan Broadway (Chen et al. 2004g). Both projects were in the greater Shanghai area. There was a priority to quickly and efficiently rehabilitate the roads with minimal disruption to the community. Underground utilities and buildings close to the road's edge were not damaged by the machine's operation. In many cases, resonant machines are specifically used in urban areas for this purpose. Other technologies, including drop and multi-head hammers, have been known to break utility pipes and damage buildings.

SUSTAINABILITY

A major concern affecting the Earth is rapid industrialization. Pollution, resource depletion, climate change and waste disposal are some of the major issues facing the entire human population. The world's yearly cement production of 1.6 billion tons accounts for about 7% of the global loading of carbon dioxide into the environment (Mehta, P. Kumar 2001a). Concrete ordinarily contains about 12% cement and 80% aggregate by mass. Globally, for concrete making, the Earth is consuming gravel, sand and crushed rock at a rate of 10 to 11 billion tons every year (Mehta, P. Kumar 2001b). In North America, Europe, and Japan, about two-thirds of the construction and demolition waste consists of masonry and old concrete rubble (Mehta, P. Kumar 2001c).

Preservation is key in order to maintain and preserve the planet for future generations. Rubblization with resonant beam technology allows the concrete to be rehabilitated in its original place. The fact the concrete can be reused allows an enormous amount of energy and resources to be conserved. Other methods require the old concrete to be removed and transported to another location. Not only is this method expensive, it's also incredibly harmful to the environment. The used concrete, if not recycled for other uses, is dumped into landfills.

CONCLUSIONS

Rubblization using the resonant beam technology is the premier solution to concrete road rehabilitation. The resonant machine can rehabilitate a concrete road five times as fast as alternative methods at half the total cost. Resonant machines fracture the slab on a shear plane that eliminates reflective cracking. The high frequency, low amplitude process will not damage underlying utility pipes or structures close to the road.

In addition to the speed and cost-savings, resonant technology is also sustainable. The concrete can be recycled in its place and used as the new base for the road as opposed to a complete dig out. Given the fast-paced in which the world is changing, it only makes sense that resonant technology is the preferred method for long-term repair of concrete roads.

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PAPER TITLE	Protecting Vulnerable Road Users: Traffic separation with colored pavement markings		
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Bike lane
Pedestrian crossing

ABSTRACT:

According to the World Health Organization, almost half of the world's road traffic deaths occur among vulnerable road users. The provision of innovative and forgiving road infrastructure and significant change in the behavior of all road users are two key factors to succeed in halving the number of traffic fatalities. Colored pavement markings are an effective way to address both: They separate vulnerable road users from the other motorized traffic and raise the attention of road users through visual stimuli.

This presentation will look at practical case studies of colored pavement markings that separate different modes of transportation and thereby contribute to the safety of vulnerable road users. They can be found all around the world: from advanced stop boxes for motorbikes in Indonesia, over separate bus lanes in Chile, bike lanes in Poland and the United States, to pedestrian crossings in Thailand and China.

Besides this paper reports the results of both lab and field tests on the performance of MMA cold plastic as one standard material for colored pavement markings

Protecting Vulnerable Road Users: Traffic separation with colored pavement markings

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1 INTRODUCTION

Infrastructure development is a constant requirement for global challenges such as urbanization, growing population and accident increases.

Today over half of the world's population (54%) live in urban areas and urbanization is expected to increase further in the upcoming decades (United Nations 2014). By 2050, 66% of the world's population is expected to reside in urban areas. Asia today is home of 53% per cent of the global urban population. And Asia is expected to urbanize faster than other regions in the world over the next decades. While the number of mega-cities with more than 10 million inhabitants is expected to increase from 28 today to 41 in 2030, the fastest growing urban areas will be cities mid-size cities (with 500,000-1,000,000 inhabitants) located in Asia. Increased urbanization requires infrastructure development, including adequate road infrastructure.

Luoma et al. 2010 examined the modes of transportation in megacities. Different modes of transportation co-exist between each other: transport on foot and by bicycle co-exists besides private motorized traffic (car and motorbike) and public transport. While transport on foot and by bicycle is currently dominant in Chinese megacities, public transport dominates in Indian megacities. The authors predict a substantial increase in personal vehicle ownership in these megacities until 2025, along with an increase in the number of road fatalities in these megacities.

Managing the co-existence of different modes of transportation and reducing the number of accidents, many cities reacted with projects separating traffic: separating cyclists from motorized traffic by separate bike lanes, separating pedestrians from motorized traffic by pedestrian zones and well-marked pedestrian crossings, separating public from private motorized traffic by establishing bus lane networks, so-called BRTs (bus rapid transportation).

Separating traffic has become a widely used and accepted way to protect vulnerable road users (VRUs). VRUs are typically defined as non-motorized road users, such as pedestrians and cyclists as well as motor-cyclists and persons with disabilities or reduced mobility and orientation. VRUs typically are at greater risk for accidents and bear more severe consequences from accidents. In 2015, 49% of the road traffic deaths around the world were pedestrians, motorcyclists and cyclists. This figure is the same for South-East Asia with motorcyclists having the highest share with 34% (WHO 2015). Although several countries have police in place to protect vulnerable road users, changing the behavior of road users and the provision of innovative road infrastructure has become a key approach to decrease the number of these accidents.

2 EFFECT OF COLORED PAVEMENT MARKINGS ON VRU

The importance of horizontal road markings in reducing traffic accidents has been well recognized. Modern road marking systems are providing various signal functionalities and safety features that make them a very efficient tool to improve road safety. However, they must be clearly visible and stay in shape at all times and at all climatic conditions. Important safety features of a pavement marking system are visibility and skid resistance (to avoid slipping), for instance.

Cities have tested separate bus/bike/pedestrian lanes and found a cost efficient solution to increase safety with colored pavement markings. Exclusive lanes for buses, cyclists or pedestrians separate fast from slow moving transport and separate vulnerable road users from higher motorized traffic. Experience suggests that separating different modes of transport, not only reduces travel times and makes transportation more efficient, but also increases road safety. Typical applications are:

- Bus lanes
- Bike lanes
- Advanced stop lanes (ASL)
- Pedestrian zones
- Pedestrian crossings

Traffic separation by giving cyclists, pedestrians and public traffic their own space, can be achieved simply by using (white) line markings. However, empirical research points out that applying color to these facilities has a much

stronger positive effect on safety. Coloring these separate lanes different from general traffic generates greater awareness and minimizes the encroachment of other traffic participants. Color improves the visibility of the facility, encourages its use by the designated user group and discourages the use by other road users. In the end this leads to less traffic conflicts and increased safety.

Giving color to bike lanes increases the visibility of the facility. This is especially important at areas of potential conflict, e.g. it raises the awareness of both cyclists and motorists at intersections and at traffic lights. Hunter et al. (2000) found that colored bike lanes increase the cyclists' comfort as his space is clearly delineated. Color discourages illegal parking on bike lanes and more motorists yield to cyclists at colored intersections. Similar research and experience has led more and more countries to specify the use of color in their national guidelines on bike lane design. E.g. the Manual for Uniform Traffic Control Devices (MUTCD) in the US and the 'Recommendation on red bike lanes' from the German Research Association for Road Markings (DSGS) in Germany. In China, a working group is currently elaborating on area marking standards.

A similar effect of color can be seen at Advanced Stop Lanes (ASL) or boxes: drivers are less likely to encroach on colored bicycle spaces than uncolored ones. (Koorey and Mangundu 2009). But color also encourages the use of ASL by cyclists. Loskorn et al. (2010) found that cyclists are more likely to use ASLs when they are colored compared to uncolored.

Colored road markings also increase the visibility of bus lanes. Usually they are treated with a red color that makes sure other motorists are aware of the bus lane – and neither drive or park on it. Several studies showed that color is effective in reducing the unauthorized use of bus lanes, including both illegal driving, standing and parking (Carry et al. 2012, New York City DOT 2011).

3 CASE STUDIES ON COLORED PAVEMENT MARKINGS

This section introduces case studies around the world, successfully contributing to the safety vulnerable road users with colored pavement markings. The choice of material, the intention of the initiative and the effect on safety is described. The design of these cases will be illustrated by pictures in the presentation.

3.1 Advances stop boxes in Bandung, Indonesia

The idea of ASL for bicycles has been adopted for motorcycles on several signalized intersections Indonesia, among others in Bandung (Bali). Red boxes for motorcycles at signalized intersections designate a space for motorcyclists to wait at a red light. Motorcyclists stop in front of other types of vehicles and can proceed through the intersection first when the light turns green. This solution reduces conflict between motorcycle and other types of vehicles and increases intersection performance. Traffic flows more orderly, safely and smoothly.

3.2 BRT in Santiago de Chile, Chile

By separating bus traffic from automobile traffic, the average bus speed increases, thereby reducing travel times and making public transport more attractive to users. With the implementation of the Transantiago system, the city of Santiago recognized the need for a better infrastructure for the buses. This is why red colored „bus only“ lanes requiring a high friction, durable and skid resistant surface were introduced. In September 2011, the transport minister announced plans to color the surface of Transantiago “bus only” lanes red, with the tacit warning to motorists being: “Do not use this lane.”

3.3 Bike lane in Syracuse, NY, US

Since 2000, many cities throughout the United States have reported a doubling or tripling in the number of people using bicycles as a primary mode of travel for work, school and recreation. With bicycling accounting for roughly 2% of all traffic fatalities and 6.3% of all traffic injuries nationwide, many cities, like Syracuse, NY, have established goals and master plans to decrease the number of crashes by adding safer infrastructure for bicyclists and pedestrians. As of today, Syracuse has incorporated new bicycle lanes as part of a new Connective Corridor project and plan to incorporate additional miles in the next two years. As part of this Connective Corridor project, the city of Syracuse has applied green bike lanes, or “greenways”, using DEGAROUTE² based MMA area markings to provide a safer biking network for students and residents navigating through the city

3.4 Bike lane in Warsaw, Poland

The intersection of John Paul II Avenue, Prosta Street, and Swietokrzyska Street is a major junction in the center of Warsaw. At this location the six-lane thoroughfares in the Polish capital meet at the four-lane Rondo ONZ traffic circle, which is also a hub for three commuter rail lines and one subway line. Approximately 450 cyclists also pass by every hour. Making sure everyone stays safe here thus poses a major challenge. Indeed, the expansion of a safe network of bike paths and lanes has become a top priority in Poland. The resins make road and other surface markings last longer and also ensure a very good tire grip on wet surfaces. The red bike path markings increase safety for cyclists because they ensure motorists can clearly see the bike lanes—it sort of sends a signal.

3.5 Bike lane in Poznan, Poland

The objective of this application was increasing safety of cyclists on a two-way bicycle path running along a highly trafficked street. Especially at junctions, the visibility of the bicycle path had to be improved to reduce car-bicycle conflicts. Since the red color had been applied, no serious accidents have been recorded in this place.

3.6 Pedestrian crossings in Denizli, Turkey

Due to increasing traffic density, the city of Denizli applied a new design at crosswalks all over the city, to ensure increased safety for pedestrians, drivers, and cyclists. Different colors and warning signs were used in front of crosswalk lines to increase attention. Applications showed great results: the number of accidents reduced and the city received positive feedback from citizens.

3.7 Pedestrian crossings at school zones in rural Thailand

The safety of children at school-zones along the rural roads in Thailand is critical, as often footpaths are not provided. Colored anti-skid rumble stripes and zebra crossings have been applied on the road to help to reduce the speed of vehicles, guide students, pedestrians, cyclists, cars and buses smoothly and safely through the school area.

3.8 Pedestrian crossing at school zones in Chongqing, China

Four red and white, anti-slip zebra crossings, covering a total area of about 400sqm were applied in front of the Liyuan road primary school and two other schools in Chongqing city. Observation statistics showed that the rate of road accident is reduced more than 50% after using a colored zebra crossing.

3.9 Pedestrian crossing in Denver, Colorado

Cold Plastic (MMA, methacrylate resin based) area markings were recently applied at three intersections along one of the busiest roads in Colorado to increase pedestrian safety. The color stability of the area markings and the retained retroreflectivity of the accent stripes are getting the attention of drivers passing by and are expected to reduce the number of pedestrian vs. vehicle accidents at these locations. Known for their high durability, increased wet-night visibility, skid resistance and optimal color stability, contrast area markings based on cold plastic are increasingly being used to apply bright crosswalks to high traffic areas.

3.10 Pedestrian zone in Graz, Austria

In the context of an urban development program, the city planners of Graz wanted to regenerate the Jakomini district to attract residents and tourists once again, and to get craftspeople and artists to settle there. Since September 2010, a 750-meter running track with a methacrylate (MMA) cold plastic area marking has been ushering pedestrians, cyclists, cars, trolley cars, and buses through the Jakomini district of Graz more safely and pleasantly than ever before. Graz was named a UNESCO City of Design in March 2011.

4 PERFORMANCE OF COLORED PAVEMENT MARKINGS

The choice of road marking material has an impact on the performance of the pavement marking: its durability, its visibility over time and its skid-resistance (i.e. non-slippery surface). The performance of pavement markings is typically measured by its skid resistance over time. This can be done by lab tests using wear simulators (advantage: equal conditions for all tests) or by field tests (advantage: real traffic and weather conditions).

Cold plastic MMA based pavement markings have been tested both in lab and field tests with excellent results. Cold Plastic/MMA based pavement markings are characterized by great UV-stability and excellent color retention over time. Besides, the durability of the cold plastic / MMA material helps to ensure a high skid resistance of the marking over years.

4.1 Lab test

Cold plastic MMA pavement markings based on DEGAROUTE resins have been tested at AETEC turntable. Results show (Figure 1), that even after 8 Million wheel passages (simulating 8 years of traffic) – the double of the highest category - SRT (Skid Resistance Tester) Values are still above the requirements of 45. Figure 1 shows the SRT values for two samples over time.

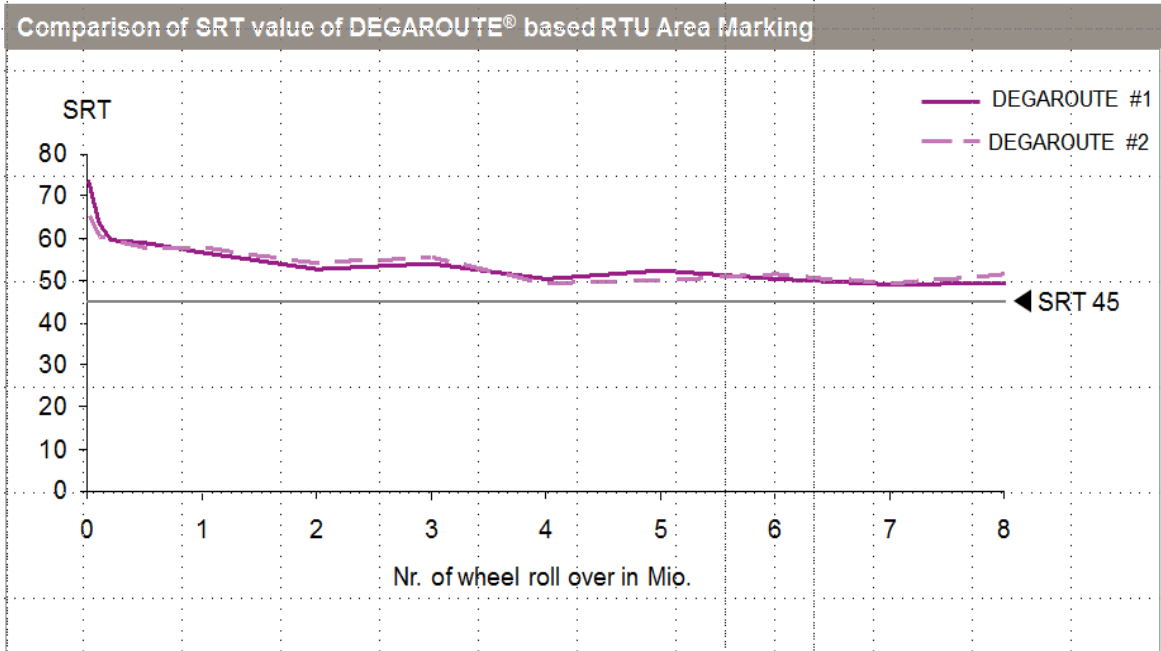


Figure 1. SRT values of DEGAROUTE based pavement marking at AETEC turntable test

4.2 Field test

Cold plastic MMA pavement markings based on DEGAROUTE resins have been applied at a roundabout in St. Martin, Austria. After 3 and 5 years, skid values still exceeded the minimum requirement of 45% (Figure 2).

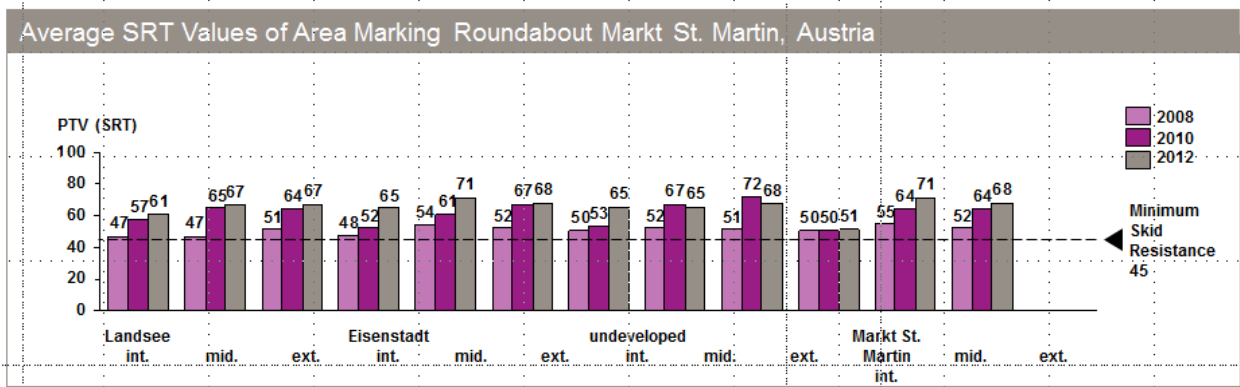


Figure 2. Average SRT Values of DEGAROUTE based pavement marking at St. Martin roundabout

6 CONCLUSIONS

Vulnerable Road Users are in the focus, when it comes to reducing road traffic fatalities. Many cities approach this issue with separating VRUs from other forms of traffic by installing bus lanes, bike lanes, ASLs, pedestrian zones and crossings, etc. Research has shown that adding color to these separate lanes or areas improves their visibility and contribution to road safety. Thus, colored traffic facilities should be used more frequently and be part of design guidelines authorities and institutes demand.

When it comes to material choice for colored pavement markings, cold plastic MMA based markings showed outstanding durability in both lab and field tests. Their practical application and positive effect on safety has been demonstrated by several case studies around the world.

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Include up to 5 keywords

Asset management, Dry air injection system, Main cable, Anti-corrosion, Monitoring

ABSTRACT:

Since a main cable is one of the most important members of a suspension bridge and its replacement work entails much difficulty, it is necessary to prolong the service life as long possible. This motivated the Honshu-Shikoku Bridge Expressway Co. Ltd. (HSBE) to install a “Dry Air Injection System for main cables (DAIS)” in all HSBE’s suspension bridges to improve humidity in the cable causing the rust of wires.

The maintenance management of the cable by DAIS is being carried out in a systematic manner with introducing a thought of asset management; the humidity inside the cable of the suspension bridges is being continuously monitored, the coating on the cable preventing a leakage of dry air is being regularly inspected, and consistent improvement is being done based on the results.

This paper describes consistent improvements of DAIS based on the results of humidity monitoring and coating inspection of the main cables of the HSBE’s suspension bridges.

Asset Management on Main Cables of Suspension Bridges

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1 INTRODUCTION

The Honshu-Shikoku Bridge Expressway (HSBE) consists of three routes (total length of 172.9km) between Honshu and Shikoku (Figure 1). It unifies the region of Seto Inland Sea and plays an important role as a part of the arterial high-standard highway network. The HSBE has 17 long-span bridges in total including the Akashi Kaikyo Bridge that is proud of the world's biggest scale as a suspension bridge and the Tataru Bridge that is a world-leading cable stayed bridge. These long-span bridges composed of ten suspension bridges, five cable-stayed bridges, one arch bridge and one truss bridge, are gigantic structures constructed with utilizing the world-highest level technologies. Implementing maintenance of these long-span bridges is based on the both concepts of preventive maintenance and asset management with a sustained effort to develop new technologies resolving several problems.

A main cable is one of the most important members of a suspension bridge. Therefore, protecting the cable from corrosion is an important task in maintenance and a “Dry Air Injection System for main cables (DAIS)” is installed in the ten suspension bridges as an anti-corrosion measure. This paper describes the maintenance of the suspension bridges’ main cables, which is being implemented based on a concept of asset management.

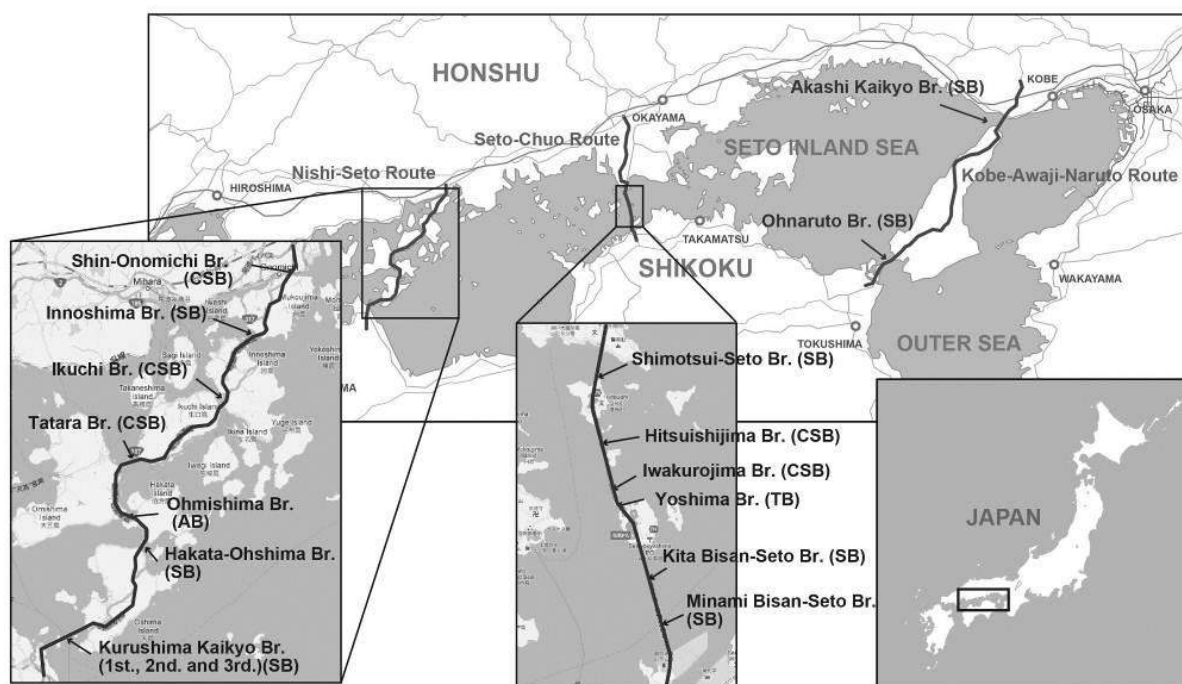


Figure 1. Honshu-Shikoku Bridges
(SB: suspension bridge, CSB: cable-stayed bridge, TB: truss bridge, AB: arch bridge)

2. MAINTENANCE OF HONSHU-SHIKOKU BRIDGES

(1) Features in Maintenance (HSBE 2015)

The HSBE's long-span bridges have the following features in terms of maintenance.

- 1) Volume of the maintenance target is massive. Signature example is 4,000,000m² of the external surface of the steel structures that needs to be recoated multiple times during the life time of the structures.
- 2) For new materials or special structures utilized in the bridges, its maintenance experience is scarce and durability is not fully known.
- 3) Since they are located over straits, they are in very harsh corrosive environment such that they are exposed to strong wind that brings chloride particles from the sea. In addition, since the superstructures are in high location over straits, they are not easily accessible and construction of scaffoldings required for inspection or repair works becomes extensive work.
- 4) Since there are no alternative routes, repair works under traffic closure are not favorable.
- 5) Since the construction cost is huge, replacement and large-scale repair works are difficult.

Above-mentioned features motivate us to implement the maintenance management under a policy of preventive maintenance, which is to conduct adequate measures before deterioration progresses become serious. The preventive maintenance policy could realize safe and reliable structures, minimize the life cycle cost (LCC) and extend the life time.

(2) Asset Management (HSBE 2015)

A basis of maintenance for the HSBE's long-span bridges is to take periodical and scheduled actions in order to prevent the progress of deterioration with time and to maintain the bridges' functions. One of the representative actions is coating on concrete and steel structures. In order to secure implementation of the preventive maintenance, we are promoting a concept of asset management as shown in a flow of Figure 2.

Procedures involved in the asset management need to perform bridge inspections and surveys according to a prescribed plan, to evaluate the bridge's overall state, to predict the deterioration and to take a proper action at an optimum time. Then, this cycle is appraised and its feedback is incorporated into the plan. This would enable implementation of effective maintenance, which could lead to realize the bridges' service life of more than 200 years.

It can be said that a key element in the flow would be inspection, which grasps the state of all structural members precisely. Furthermore, in order to secure implementation of the flow, so-called PDCA (Plan, Do, Check and Action) cycle, accumulation of the inspection data, appropriate evaluation and deterioration prediction based on analyses of the data would become important as well. It is believed that establishment of repair plans and its implementation would result in effective maintenance management and minimization of the LCC.

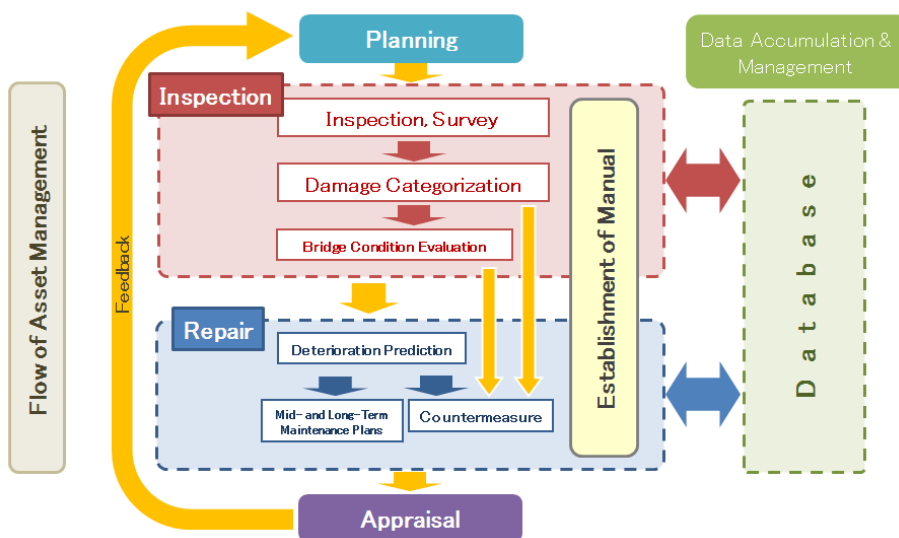


Figure 2. Flow of Asset Management

3. DEVELOPMENT AND INSTALLATION OF DAIS

3.1 Background for Development of DAIS (Fujikawa et al. 2000)

A conventional anti-corrosion measure is composed of paste, wrapping wire and coating as shown in Figure 3, which aims to prevent water from penetrating into the cable from the surface. In 1989, an investigation of the cables of the Innoshima Bridge, the first HSBE's suspension bridge, was conducted. In the investigation, it was found that the deteriorated paste retained water and that induced corrosion of the cable wires within a few layers from surface as shown in Figure 4. This indicated that the conventional measure was insufficient for corrosion protection of the main cables of suspension bridges. Following the investigation, we initiated the development of a new anti-corrosion measure, DAIS, which will improve corrosive environment inside the cable by injecting dry air into the cable, in addition to preventing water penetration.

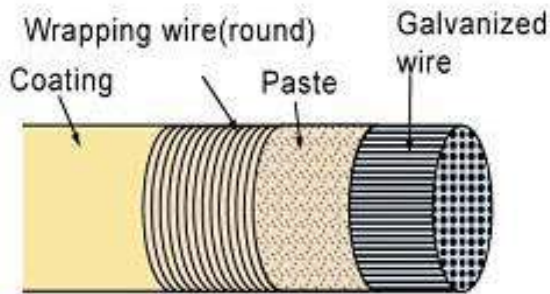


Figure 3. Conventional corrosion protection system for cable

Figure 4. Unwrapped Cable at Innoshima Bridge

3.2 Outline of DAIS (Fujikawa et al. 2000)

The DAIS consists of filter unit, dehumidifier, roots blower, piping, injection cover and exhaust cover as shown in Figure 5. A salt-removal filter is also installed because injecting dry air with chloride ion would make salt accumulate inside the cable with time, which is unfavorable in terms of corrosion protection. The filter has the capacity to eliminate 99.7% of particles with the size of more than $0.3\mu\text{m}$ (typical chloride particle size: $0.3\text{--}30\mu\text{m}$).

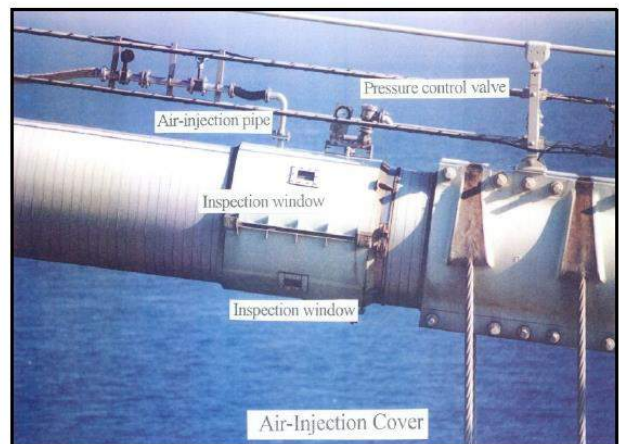
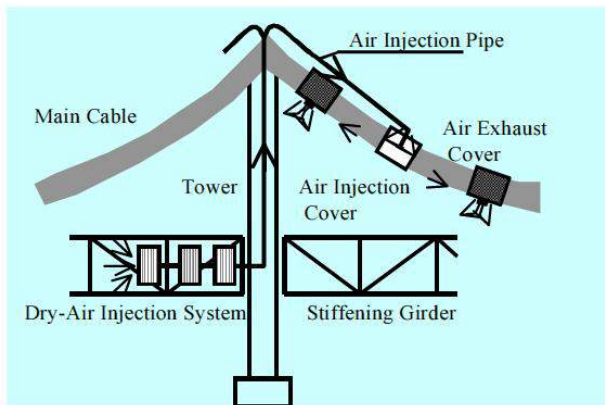


Figure 5. Dry Air Injection System (DAIS)

3.3 Installation of DAIS (Ogihara et al. 2013)

The DAIS was developed in parallel with the construction of the Akashi Kaikyo and Kurushima Kaikyo Bridges and installed in the bridges during its construction stages in 1997 and 1999, respectively. It was also installed in the remaining eight suspension bridges between 1997 and 2002. Since it was found that the system had not dehumidified inside of the cables well and the relative humidity (RH) inside the cable had remained high even after its installation in some bridges, an additional injection cover was installed to shorten the air blowing length as shown in Table 1.

Table 1. DAIS in HSBE's suspension bridges

Bridge Name	Central Span Length (m)	Bridge length (m)	Opened Year	Location	Number of Injection Points		Number of Units	
					Original	Present	Original	Present
Akashi-Kaikyo	1,991	3,911	1998	Hyogo	32	32	8	6
Ohnaruto	940	1,629	1985	Hyogo Tokushima	13	27	2	2
Shimotui-Seto	940	1,400	1988	Okayama Kagawa	6	12	3	3
Kita Bisan-Seto	990	1,538	1988	Kagawa	6	12	3	3
Minami Bisan-Seto	1,100	1,648	1988	Kagawa	6	12	3	3
Innoshima	770	1,270	1983	Hiroshima	8	8	2	2
Ohshima	560	840	1988	Hiroshima	8	8	2	2
1st Kurushima Kaikyo	600	960	1999	Ehime	8	8	2	2
2st Kurushima Kaikyo	1,020	1,515	1999	Ehime	12	8	3	2
3rd Kurushima Kaikyo	1,030	1,570	1999	Ehime	12	8	3	2

4. MAINTENANCE OF MAIN CABLE

4.1 Setting of Target Value of RH (Fujikawa et al. 2000)

Since it is necessary to set a target value of RH inside the cable in order to properly operate the DAIS, several studies were carried out. As a result, it was found that a galvanized wire with a tiny amount of adherent salt (less than $0.1\text{g}/\text{m}^2$) doesn't corrode in a dry air environment with RH of less than 60% RH as shown in Figure 6. Given the fact, the target value inside the cable was set to be 40% RH taking into account unevenness of RH along the cable and other unknown factors. On the other hand, the target value inside a spray chamber in an anchorage was set to be 50% RH, because the cable condition can be easily identified by visual inspection.

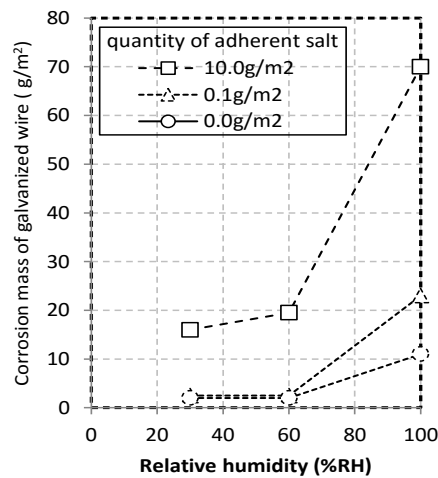


Figure 6. Relationship between RH and corrosion

4.2 Continuous Improvement of DAIS based on PDCA Cycle

(a) PDCA Cycle

Maintenance management of the cable is being implemented based on PDCA cycles as shown in Figure 7, which incorporates a concept of asset management. The outer cycle corresponds to a concept level, which is to make a basic policy for the cable maintenance, to prepare a maintenance manual and a long-term maintenance plan, to perform inspection and monitoring according to the plan, and to periodically evaluate the cable's condition. Then, the cycle is appraised and its feedback is incorporated into the manual or plan. The middle cycle corresponds to a strategy level, which is to draw up a 3-year maintenance plan, to secure the airtight cable, and to improve the system for more reliable and economical operation. The inner cycle corresponds to an implementation level, which involves annual activities as described below;

- ✓ Monitoring
- ✓ Securing an airtight cable
- ✓ Installing a pre-cooler
- ✓ Installing an additional injection point
- ✓ Performing an economical operation test

These activities allow us to accumulate data, which is indispensable to do "Check" and "Action" in the cycles.

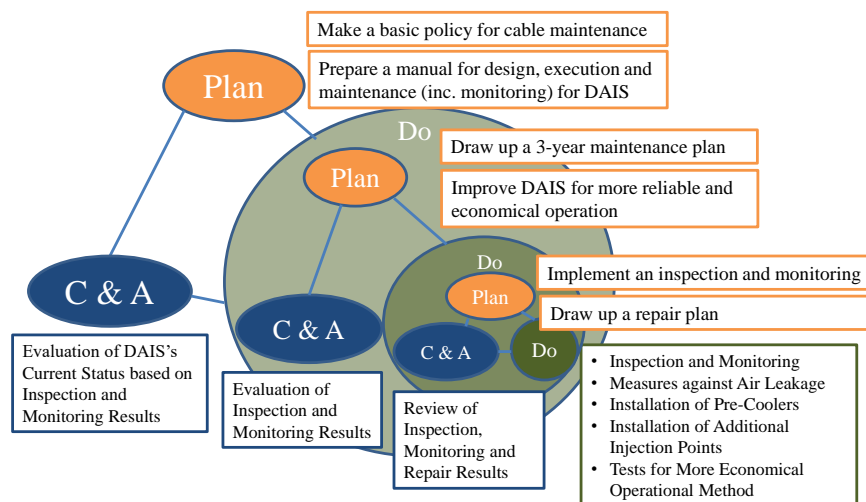


Figure 7. Asset Management Cycle for Cable Maintenance

(b) Monitoring

An operating status of the DAIS in the HSBE's suspension bridges is being continuously monitored by automatically measuring both temperature and RH of exhausted air at the cover. In addition, a manual measurement is periodically made to compensate the automatic measurement, because automatic measuring points in some bridges are coarse and malfunction of sensors for the automatic measurement sometimes happens. Especially in summer, when outside air humidity is normally high and RH inside the cable increases in proportion to outside air humidity, the manual measurement is mainly made by an inspector on a day of good weather. Items of the manual measurements are both temperature and RH of injected and exhausted air at the covers, as well as of produced air at the dehumidifier, and blowing pressure at injection points.

(c) Improvement of DAIS (Kusuhara et al. 2015)

The monitoring data showed that the system had not dehumidified inside of the cable well and RH in some portions of the cable had remained higher than the target value, 40% RH. The causes were considered as the following.

- ✓ Air leakage at defects of sealing of the cable band, injection and exhaust covers
- ✓ Air leakage at cracks of the cable coating
- ✓ Air leakage at cracks of piping defect or at loosened joints of the pipe
- ✓ Malfunction of the DAIS's mechanical equipment
- ✓ Malfunction of the monitoring devices
- ✓ Lack of the DAIS capacity

In order to solve the problem, two countermeasures has been mainly implemented; improvement of the DAIS and securing an airtight cable. As for the former measure, machinery improvements for more efficient dehumidification and installation of additional injection/exhaust covers to shorten the blowing length have been employed. As for the

latter one, measures for the deterioration of cable coating and the defects of sealing of the cable bands have been applied. Examples of the system repairs and improvements are described in the following section.

5 Improvement of DAIS


5.1 Example at Seto-Ohashi Bridges

(1) Installation of Additional Injection Points

Original installation arrangement of injection points at the Seto-Ohashi Bridges (three suspension bridges) was set as shown in Table 2 based on the experience in other bridges and the result of preliminary air flow tests performed at the sites. The system, however, had not dehumidified inside of the cables well and the relative humidity in the most portions of the cables had remained high even after the installation of the system. Since the blowing lengths of the Seto-Ohashi Bridges were over 200m and high humidity portions in the cables were found to be limited to vicinities of the exhaust points, this fact seemed to be responsible for the insufficient blowing capacity of dry air up to the exhaust points. Therefore, additional injection points were installed to shorten the blowing length. Changes of the blowing lengths by installation of additional injection points are also shown in Table 2.

Effects of the installation of additional injection points in the Kita Bisan-Seto Bridge are shown in Figure 8. It was found that RH at all three points were drastically reduced owing to the installation, although RH in front of the anchorage has been occasionally over the target value, 40% RH. The cable in front of the anchorage was confirmed to be in sound condition and is being periodically monitored by visual inspection. Also, a pre-cooler was installed to improve capability of producing dry air especially during summer seasons, and its effectiveness was confirmed.

Table 2. Changes of Blowing Lengths by Installation of Additional Injection Points
(Seto-Ohashi Bridges, after 2005)



Year	Shimotsui										
	230 m	I	228 m	E	269 m	I	236 m	E	228 m	I	230 m
1999	230 m	I	228 m	E	269 m	I	236 m	E	228 m	I	230 m
	115 m	I	115 m	E	114 m	I	114 m	E	118 m	I	131 m
2014	115 m	I	115 m	E	114 m	I	114 m	E	118 m	I	131 m
	115 m	I	115 m	E	114 m	I	114 m	E	118 m	I	131 m
1999	274 m	I	240 m	E	249 m	I	262 m	E	240 m	I	274 m
	128 m	I	146 m	E	125 m	I	115 m	E	131 m	I	118 m
2014	128 m	I	146 m	E	125 m	I	115 m	E	131 m	I	118 m
	128 m	I	146 m	E	125 m	I	115 m	E	131 m	I	118 m
1999	274 m	I	268 m	E	288 m	I	275 m	E	268 m	I	274 m
	137 m	I	137 m	E	134 m	I	134 m	E	144 m	I	144 m
2014	137 m	I	137 m	E	134 m	I	134 m	E	144 m	I	144 m
	137 m	I	137 m	E	134 m	I	134 m	E	144 m	I	144 m

I: air injection point, E: air exhaust point

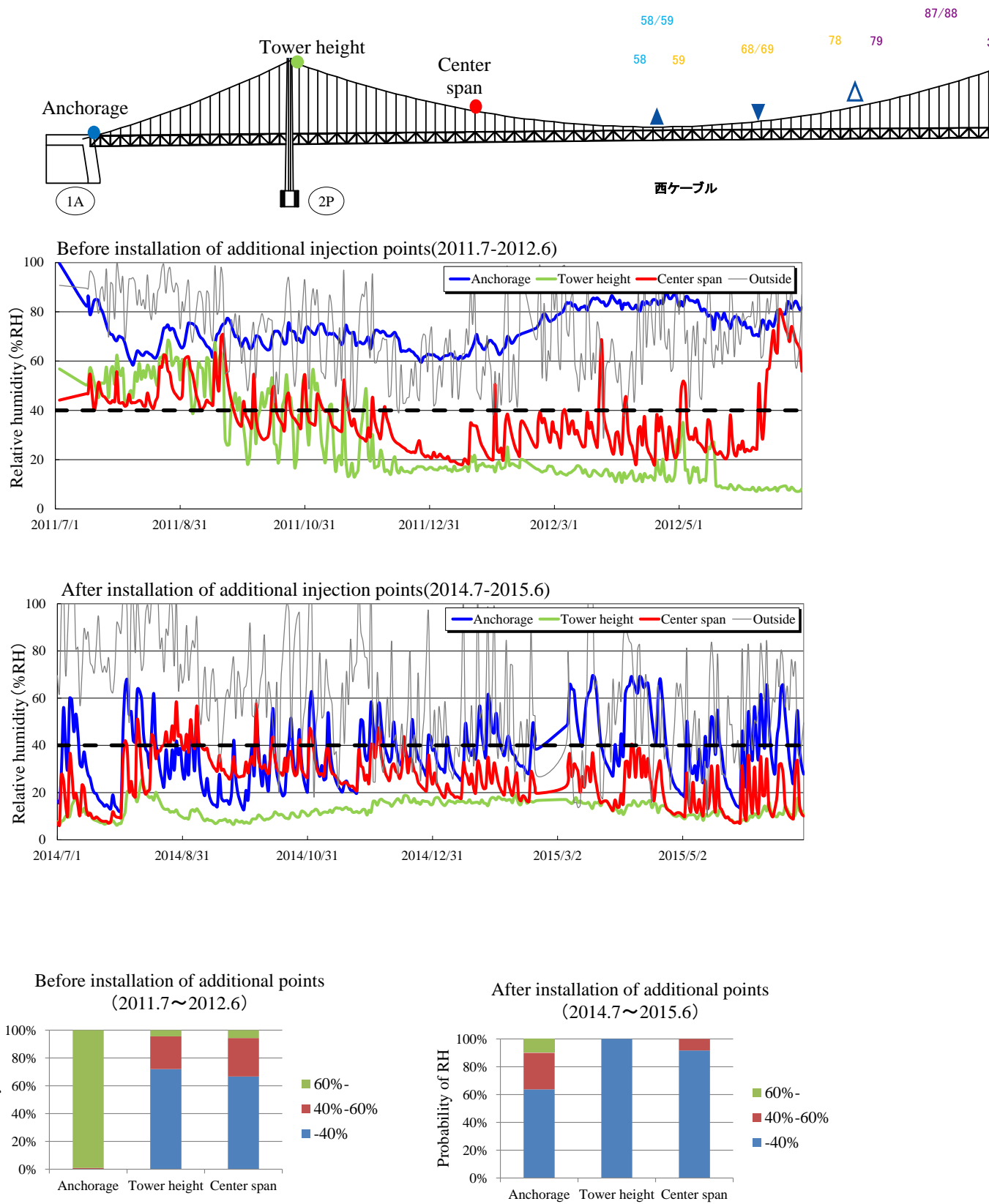


Figure 8. Effects of Installation of Additional Injection Point at Kita Bisan-Seto Bridge (West Cable)

(2) Improvement of Humidity inside Spray Chamber

It is necessary to dehumidify not only inside the cable but inside a spray chamber. This chapter shows the improvement of humidity inside a spray chamber.

(a) Installation of Additional Fans

It is difficult to dehumidify inside a spray chamber installing only a dehumidifier because a dehumidified volume is big. Therefore, two improvements were implemented installation of additional fans to circulate air inside a spray chamber, effective running of fans by installation of the sensor to high humidity portions. These installations enabled RH in a spray chamber to be almost below control target value of 50% expect bigger volume of 4A anchorage as shown in Figure 9.

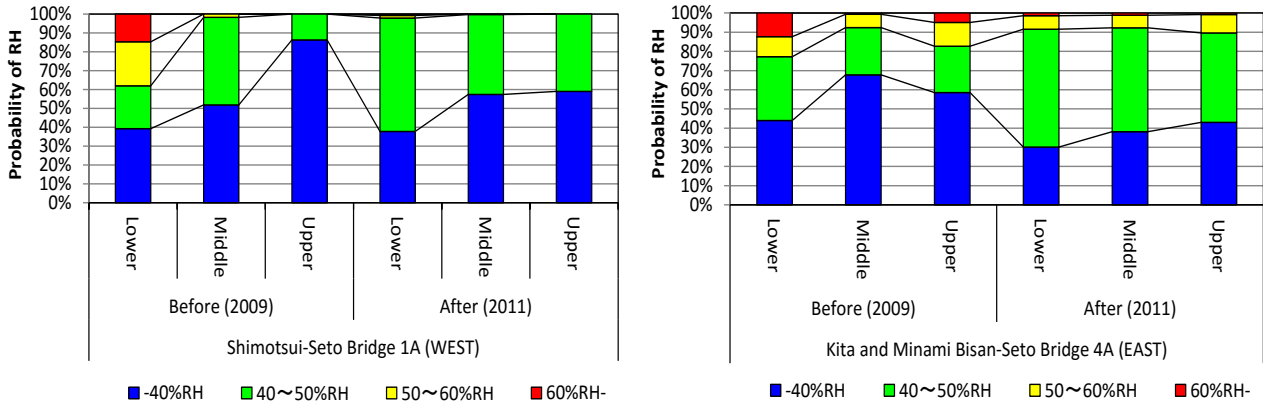


Figure 9. Comparison of RH before and after Installation of Fans

(b) Installation of Membrane Structure

It is obvious that less spatial volume in a spray chamber would contribute to higher efficiency of dehumidification. In order to decrease a spatial volume and to prevent the strands from leakage of water from ceiling, membrane structures covering strands were installed in the 1A and 7A anchorages of the Kita and Minami Bisan-Seto bridges. Table 3 shows the comparison of a spatial volume in a spray chamber in the Seto-Ohashi Bridges.

Since the 4A anchorage bilaterally used for the two suspension bridges where configuration of strands crossing each other is complicated, the membrane structures, which covers only strands above the crossing portion to decrease a spatial volume of chamber, was installed in 2013 to 2014. Figure 10 shows the installation of membrane structure in 4A anchorage of the Kita and Minami Bisan-Seto bridges, and Figure 11 shows the photo of membrane structure. The spatial volume decreased from 21,400m³ to 15,400m³ by installation of the membrane structure.

Table 3. Comparison of Spatial Volume in Spray Chamber

Bridge Name	Spray chamber volume (m ³)
Shimotsui-Seto	1A 6,500
	4A 9,500
Kita Bisan-Seto	1A 13,700 (7,000)
	4A 21,400 (15,400)
Minami Bisan-Seto	7A 19,400 (7,000)
Akashi-Kaikyo	4A 11,430

() : after installation of membrane

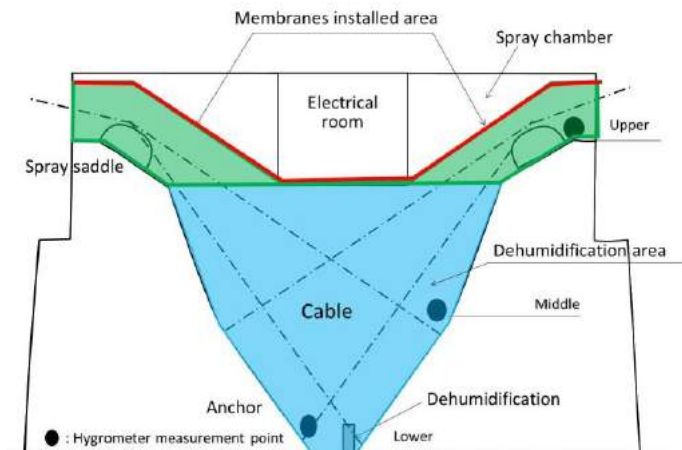


Figure 10. Installation of Membrane Structure in 4A Anchorage of Bisan-Seto Bridges



Figure 11. Membrane Structure

5.2 Case at Akashi Kaikyo Bridge

(1) Installation of Pre-Cooler and Reduction of Running Cost

(a) Operational Status

The DAIS's specification installed in the Akashi Kaikyo Bridge was designed to dehumidify moisture intruded inside of the cable during its construction within one year. It was confirmed that the system successfully dehumidify inside of the cable below the target value of 40% RH by approximately 150 days after its operation was initiated. Then, it was also confirmed that RH inside the cable occasionally exceeded the target value, especially during summer season with high-temperature and humidity.

In order to improve this situation, the system was altered so that extra dry air was recirculated and its cost- and performance-effectiveness was confirmed in 2001. Although the system has been in stable operation since the alteration, exceeding the target value of RH inside the cable was occasionally observed during summer season.

In response to this, a pre-cooler was installed in a dehumidifier in one circulation line on a trial basis in 2015 and the installation has soundly kept RH below the target value. The installation will be discussed more in (b).

(b) Cost Reduction aiming to Economical Operation for a Long Term (More Than 200 Years)

As part of our efforts to reduce the operation cost, we changed the operational method by reducing blowing air and optimizing the output of the system unit in 2007, when the system became to be in stable operation. Blowing air was cut by half and the power consumption of a root blower was reduced by changing the number of a motor pole from four to six as shown in Table 4. The change saved approximately 27 million JPY on the operation cost during the last ten years.

Table 4. Cost Reduction by Measure Implemented in 2007 at Akashi Kaikyo Bridge

Measure	Implemented Content	Investment for Equipment	Cost-Saving Benefit
Reduction of blowing air and optimization of operation	<ul style="list-style-type: none"> • Halve blowing air • Reduce the power consumption of a root blower by changing the number of a motor pole from 4 to 6 • Reduce the number of operating units at the towers from 4 to 2 	¥3.2 million	¥2.7 million/year ¥27 million saving is expected in 10 years, which is the mechanical life

In addition to the measure implemented in 2007, two additional measures were taken in a trial basis in 2015; one is installation of a pre-cooler, whose effectiveness was verified at the Kurushima Kaikyo Bridge, the other is reduction of heating temperature for drying a silica gel rotor in a dehumidifier. Cost-saving benefit based on the result during the last six months' operation is tabulated in Table 5.

Table 5. Cost Reduction by Measure implemented in 2015 at Akashi Kaikyo Bridge

Measure	Implemented Content	Investment for Equipment	Cost-Saving Benefit
Installation of pre-cooler and reduction of heating temperature for silica gel rotor	<ul style="list-style-type: none"> • Install a pre-cooler in one circulation line out of eight lines as shown in Figure 3 • Reduce heating temperature for drying a silica gel rotor from 140 to 70 degree 	2.1 million JPY/line (12.6 JPY/6 lines)	0.4 million JPY/year, line (370->270 kwh/day) 24 million JPY saving by 6 lines is expected in 10 years, which is the mechanical life.

The Seto-Inland Sea region where the Honshu-Shikoku Bridges including the Akashi Kaikyo Bridge are located has a warm climate. Figure 12 shows an annual RH data recorded at Kobe in every hour in 2015 (8760=24*365 data). It indicates that data with RH more than 40% and 60% account for 96% and 63%, respectively. Besides, an annual temperature data in every hour in 2015 shows that data with temperature more than 19 degree and less than 18 degree account for 55% and 45%, respectively. These imply that the region is under high humidity and wide-range temperature environment for wire corrosion.

As described earlier, since exceeding the target value of RH inside the cable was occasionally observed during summer season, a pre-cooler was installed in a dehumidifier in one circulation line on a trial basis. Advantageous points of the pre-cooler are; it enables to efficiently dehumidify air in high humid and temperature with temperature of more than 19 degree, and it is composed of small-sized instruments which can fit into the existing dehumidifier. As shown Figure 13, humid outside air is firstly dehumidified at the pre-cooler by getting cool the air in a cooling coil and draining liquefied water to outside, then secondly dehumidified at the silica gel rotor. The pre-cooler will not be able to dehumidify air with temperature of less than 18 degree because of frost getting stuck at the cooling coil. Accordingly, the pre-cooler is set to halt when temperature of outside air becomes less than 18 degree. It was verified that the hybridization of a dehumidifier and a pre-cooler successfully dehumidifies outside air throughout the year, even during summer season with high-temperature and humidity.

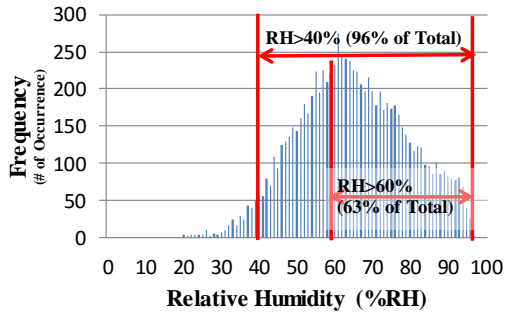


Figure 12. Annual RH Data Recorded at Kobe in 2015

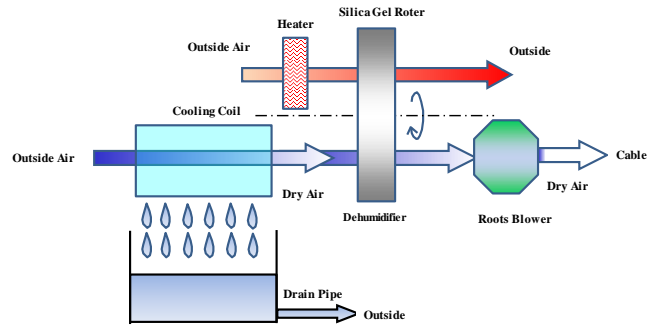


Figure 13. Dehumidifying Process

In addition, heating temperature for drying a silica gel rotor in one circulation line was experimentally reduced from 140 to 70 degree in the period of August 2015 through February 2016 in order to save electricity consumption increased by the hybridization. As a result, electricity consumption was decreased to 70%, from 370 to 270kwh/day, and RH inside the cable was almost kept below the target value of 40% RH under wide-range temperature variation, from -2.5 to 27 degree.

The additional two measures have been taken for all six circulation lines from this year. Although it costs 12.6 million JPY, it is expected to save 24 million JPY during the next ten years.

6 CONCLUSIONS

After installation of the DAIS in all HSBE's suspension bridges, improvements of the DAIS have been consistently implemented based on inspection or monitoring results. These efforts have made RH inside the cable to be kept below the target value, and thus the cables seem to be in sound condition at this moment. In order to maintain the cables for a long time, improvements of the DAIS will be continued based on a PDCA cycle, which incorporates a concept of asset management. Besides, further studies in terms of material of the wrapping system with higher durability and economical way of operation are going to be performed.

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<p>KEYWORDS: Expansive soils, Gilgai, roadside vegetation, waveband roughness, pavement deterioration</p>

<p>ABSTRACT: Non-uniform ground movement due to moisture variation in an expansive soil subgrade often results in increased pavement roughness that effects ride quality and causes passenger discomfort. It is a common belief that the presence of roadside vegetation can contribute to increased roughness levels and progression rates, but this has not yet been quantified in any published literature. In order to evaluate the influence of roadside vegetation on roughness progression, roadside vegetation data such as number of roadside trees, size of tree canopy area, height of tree, relative distance between tree and pavement seal, and a density rating for any grass present in the unsealed shoulder was developed. This data was collected using satellite imagery for a rural highway in North-Western Victoria, Australia, which was then analyzed against historical road roughness data to develop a model to quantify the influence of vegetation on road roughness progression rates.</p>

INFLUENCE OF ROADSIDE VEGETATION ON ROUGHNESS PROGRESSION FOR PAVEMENTS FOUNDED ON EXPANSIVE SOILS

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1 INTRODUCTION

In Australia, it is quite common for pavements founded on expansive soils to experience extensive roughness, premature distresses and structural deformation due to non-uniform ground movements. Overall, six out of the eight Australian capital cities and surrounding areas are affected by such geological conditions. All light structures, such as road pavements, parking lots, residential dwellings, light commercial buildings, etc. are all vulnerable to such ground movements associated with expansive soil behavior (Delaney et al., 2005, Fityus et al., 2004, Krohn and Slosson, 1980). Expansive soils are defined as those that shrink and swell significantly when subjected to variations in moisture. They are typically clay type soils that yield high liquid limits and levels of plasticity. More specifically, it is the presence of “active” clay minerals (i.e. montmorillonite) within the soil that creates the greatest potential for expansion and contraction. Literature suggests that factors responsible for expansive behavior can be classified as either intrinsic factors or driving factors. The intrinsic factors include clay content (Anderson et al., 1973, Franzmeier and Ross, 1968, Komornik and Zeitlen, 1965, Russell, 2002), organic content (Davidson and Page, 1956), cation exchange capacity (Franzmeier and Ross, 1968, Gill and Reaves, 1957), type of adsorbed cation (Anderson et al., 1973), charge density (Komornik and Zeitlen, 1965), amount and type of clay minerals present (Anderson et al., 1973, Komornik and Zeitlen, 1965), and bulk density (Finn and Strom, 1958, Komornik and Zeitlen, 1965). The driving factors that act to instigate expansive behavior of a soil includes climate (Krohn and Slosson, 1980, Lambe, 1960), site drainage (topography) (Mathewson et al., 1975) and vegetation (Hammer and Thompson, 1966). Any of these factors or their combination can initiate expansion and contraction of the subgrade, which may result in increased roughness levels (loss of shape) and longitudinal cracking. Longitudinal cracking usually initiates due to the desiccation and shrinkage of soil beneath the edge of the pavement, while road roughness can be attributed to non-uniform volume changes and associated movements beneath the pavement. Such non-uniform volume changes often occur due to development of Gilgai.

Gilgai is a topographical feature that comprises of a series of mounds and depressions (i.e. undulating surface terrain). Gilgai is an Aboriginal (Australian) word that means “small waterhole”, which refers to the water captured in the depression of the undulating ground. Aitchison (1953) was the first to recognize and introduce the principles of Gilgai formation in Australia. Many others, namely: Knight (1980), Maxwell (1994), Paton (1974) have all since proposed slightly different theories about Gilgai formation. Nevertheless, the best and most complete description of Gilgai and its formation can be found in the Australian Handbook of Soils (Hallsworth, 1968). In brief, it is the presence of an expansive soil deposit that is responsible for Gilgai formation. As deep shrinkage cracks allows water to penetrate deep into the ground, deep seated swelling occurs and initiates the formation of the mound. This mound then slowly grows each climatic drying and wetting cycle.

At present, approximately 30 per cent of all Australian dwellings and approximately 50 per cent of Victoria’s surface area is covered by moderate to highly expansive soils (Holland and Richards, 1982, Richards, 1965). Moreover, a considerable amount of Victorian soil deposits have shown significant trends in developing Gilgai characteristics. In this specific area, Gilgai formations have been measured successfully using road profile data and associated waveband analysis (Evans et al., 2015). As the variation in moisture plays a major role in expansive soil behavior and the formation of Gilgai, the presence of roadside vegetation should influence the ground moisture and thus influence the development of Gilgai and associated pavement roughness development. Current literature suggests that trees do significantly affect subsurface moisture

levels and variations (Blight, 2005, Snethen, 2001). The influence has been measured vertically up to 10 m below the ground surface and horizontally up to 1.5 to 2 times the height of the tree (Richards et al., 1983). However, no studies have ever quantified the relationship between presence of roadside trees and roughness levels. This paper aims to develop relationships between various tree parameters and roughness progression rates.

2 SITE DESCRIPTION

The Borung highway is located in North-Western Victoria and runs from St Arnaud to Litchfield to Warracknabeal to Dimboola. This highway was selected as the test site for this study based on the following criteria: (i) distribution of expansive soil deposits, (ii) presence and possible development of Gilgai in surrounding areas, (iii) favorable climate condition that promotes significant shrink-swell behavior, (iv) typical populations of roadside trees present in road reserve, and (v) availability of quality historical pavement roughness data. Among hundreds of sections along the Borung Highway, a total of 109 sections (each 100 m in length) were selected to evaluate the influence of vegetation on pavement roughness levels. All selected sections are rural highway sections between the township of Litchfield and Dimboola (approximate 80 kms in length) which have shown a wide variation of roughness progression, as shown in Fig. 1. In this figure, the white region is considered to represent geology that is non-expansive. The yellow to dark orange regions represent alluvial expansive soils, and the pink regions represent residual expansive type soils (i.e. weathered basalt). Most importantly, the darker the color in this figure, the more expansive the soil.

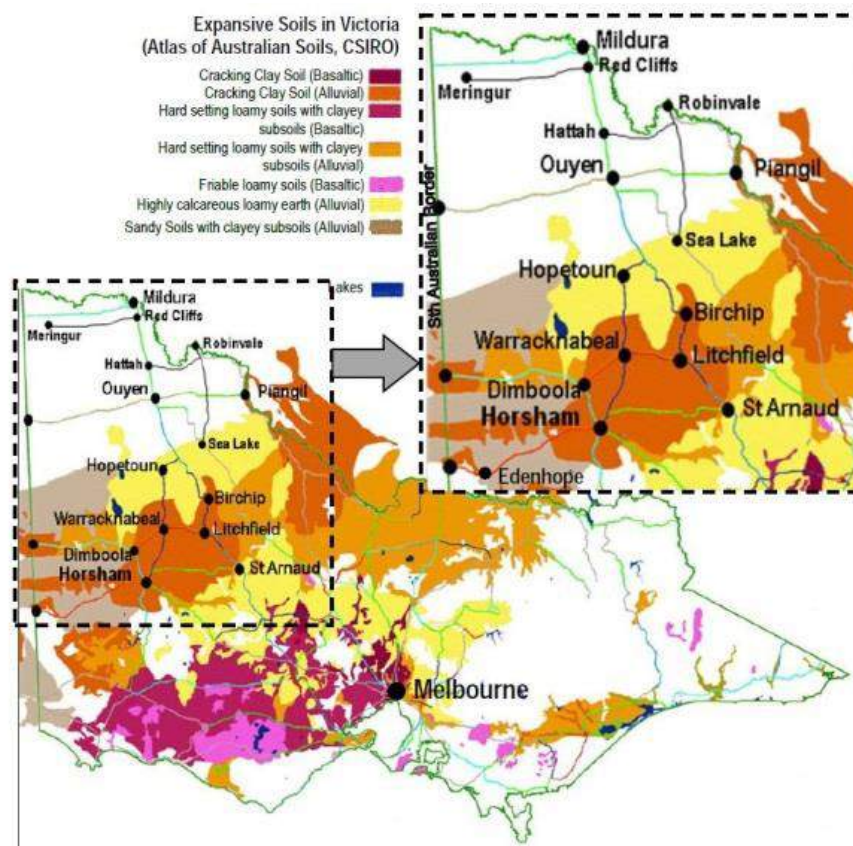


Figure 1. Distribution of expansive soil in Victoria (Australia) and location of test site (Evans, 2013)

3. DATA COLLECTION AND METHODOLOGY

The data collection of this study comprised of two parts: (a) pavement roughness data and (b) roadside vegetation data. The raw pavement roughness data was provided by VicRoads (the State Road Authority of Victoria, Australia) and the roadside vegetation data was collected using satellite imagery software. The roughness data was collected every two years using a vehicle-mounted laser profilometer

device to measure the raw longitudinal road profiles. These road profiles were then used to calculate the International Roughness Index (IRI) via the quarter car model at 100 m intervals. The IRI is the most popular and acceptable way of interpreting pavement roughness. In general, the higher the IRI value, the lower the ride quality and the higher the passenger discomfort. The quarter car model used to calculate the IRI theoretically responds to wavelengths between 0.5 m and 100 m, but it is mostly influenced by wavelengths between 1.2 m and 30 m and is most sensitive at 2.4 m and 15.4 m. These two wavelengths are known for their characteristics of causing high passenger discomfort. Although the IRI represents a summary of the overall pavement roughness level within a pavement, it may become necessary to distinguish between long and short wavelength roughness content (i.e. the type of roughness present in the pavement). It is believed that the influence of short wavelength roughness is limited to deformation of the upper layers of the pavement, while roughness within the lower layers (i.e. subgrade) is represented by long wavelength roughness. In order to evaluate the cause of roughness within the pavement, waveband analysis was performed to determine the contribution of each waveband in terms of roughness. To calculate the waveband roughness contribution, the raw longitudinal road profile data was passed through various band pass filters (namely the four-pole Butterworth filter using Mean Absolute Values (MAV) in this study). The RoadRuf software package was used for these calculations. The waveband intervals were setup based on double octave bandwidths. The Double Octave Bands (DOB) were classified into seven categories to cover the full definition of roughness (i.e. 0.5 m to 100 m). However, indices DOB3, DOB4 and DOB5 were considered to best match the wavebands similar to the IRI. These octave bands (DOB1 to DOB7) with their respective lower and upper wavebands have been defined in Table 1.

Table 1. Definition of Butterworth (MAV) double octave waveband indices

Profile Name	λ_{lower}	λ_{centre}	λ_{upper}
Butterworth (MAV)- DOB1	0.354	0.707	1.414
Butterworth (MAV)- DOB2	0.707	1.414	2.828
Butterworth (MAV)- DOB3	1.414	2.828	5.657
Butterworth (MAV)- DOB4	2.828	5.657	11.314
Butterworth (MAV)- DOB5	5.657	11.314	22.627
Butterworth (MAV)- DOB6	11.314	22.627	45.255
Butterworth (MAV)- DOB7	22.627	45.255	90.51

Google Earth Pro was used to collect the roadside vegetation data as it offers recent satellite imagery with acceptable precision and accuracy. Tree data was collected using both satellite view and street view. Distance and canopy area measurements were taken from the satellite view option within Google Earth Pro, while the street view option was used to determine elevations.

(a) PAVEMENT ROUGHNESS DATA COLLECTION AND MANAGEMENT

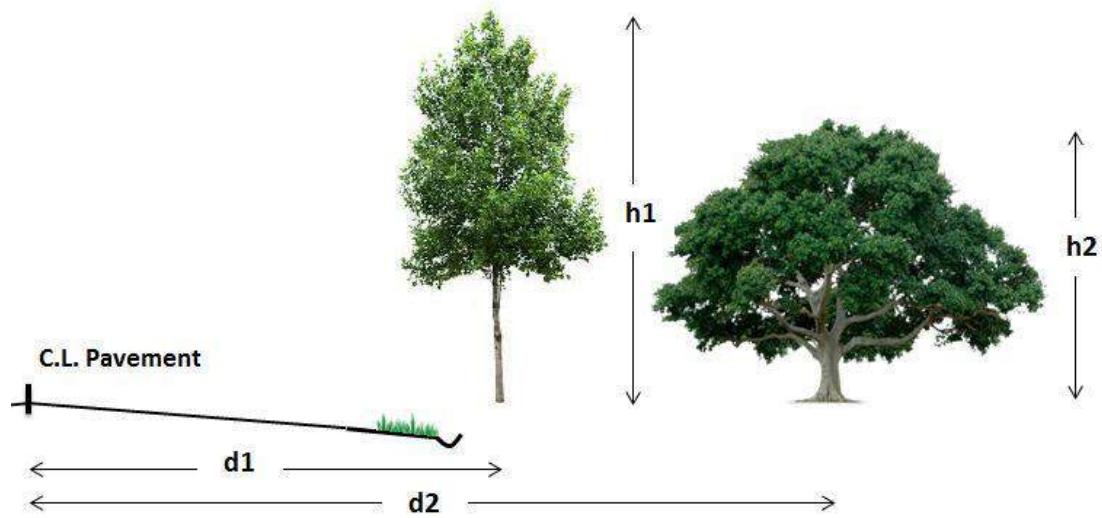
VicRoads usually conduct detailed road surveys once every two years for rural highways across the State of Victoria, which consists of measuring longitudinal road profiles and calculating the IRI at 100 m intervals. For the Borung highway, consecutive road roughness data was available from 1995 to 2009, with the exception in 2003. From the 80 kms of potential Borung highway to select, 150 sections (each 100 m in length) were initially selected as they contained abnormally high levels of roughness. However, to maintain a good distribution of roughness levels, only 109 sections were finally selected and used in this study. The selected test sites were identified according to Victoria's State Road Referencing System (SRRS), which is based on chainages. This was essential for the proper identification and management of the test site data.

Roughness progression rates (for all indices) were calculated for each selected test site. These roughness progression rates were also separated into pre-maintenance and post maintenance rates. This was to increase the actual accuracy of the deterioration. Maintenance activity can easily be identified from the roughness data over cumulative years from any sudden drop in level. This approach was used to determine the pre-maintenance and post-maintenance roughness progression rates.

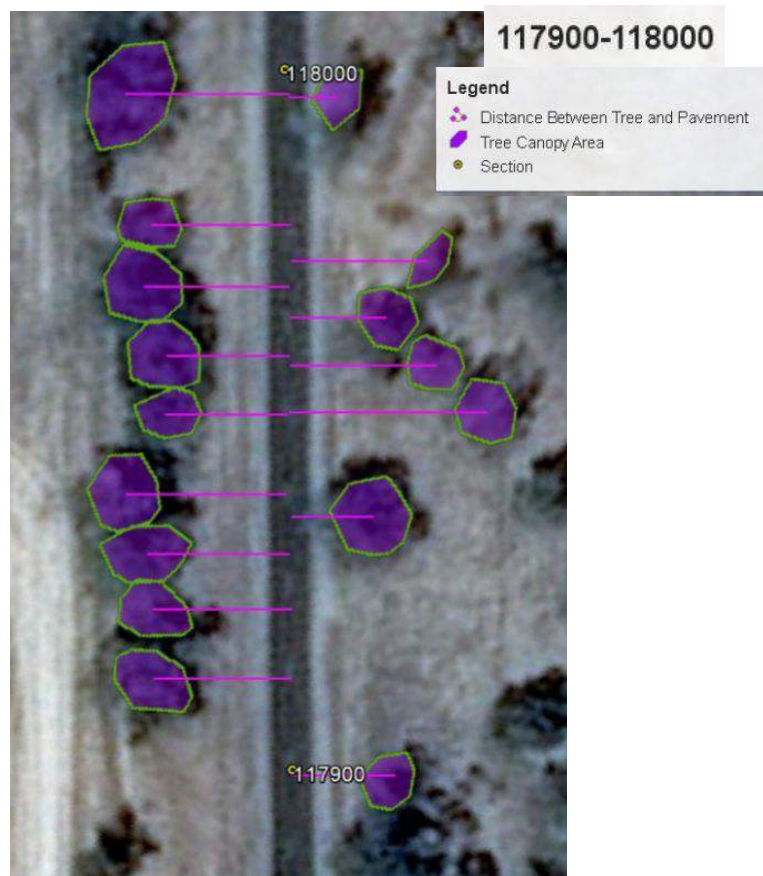
(b) ROADSIDE VEGETATION DATA COLLECTION

In order to determine the influence of roadside trees on pavement roughness, vegetation data was collected for the selected sections. This data included: (i) number of trees present on each side of the

pavement, (ii) location of individual trees along the pavement, (iii) offset distance between individual trees and centre-line of the pavement, (iv) canopy area of each tree, (v) height of each tree, and (vi) density rating of grass present in the shoulder. Figure 2 depicts some of the data collected for the roadside trees database.



(a)



(b)

Figure 2. Identification of tree data measurements (a) distance (offset) and height, and (b) an example of canopy area and distance (offset)

From satellite imagery, the number of trees present on each side of the pavement section has been tabulated. Only trees located within the road reserve were considered. All types of trees including mature trees, young trees, bushes and even trees with no canopy were considered. For large groups of trees, street view imagery was used to identify and tabulate these individual trees.

The offset distance between individual trees and center-line of the pavement were measured using the “Ruler” tab available in Google Earth Pro. Any two dimensional measurement is possible using this option and any desired scale can be chosen. The procedure of taking measurements in Google Earth Pro has been illustrated in Figure 3. In Figure 3 (a) the tree has been identified as DTLR104783 as it is located on left side of the pavement at a chainage of 104783 m and has an offset distance of 17.6 m from the centre-line of the pavement. In Figure 3 (b) the canopy area of tree (TR104783) has been measured using a polygon. Here, the area of the polygon measures 47.1 m².

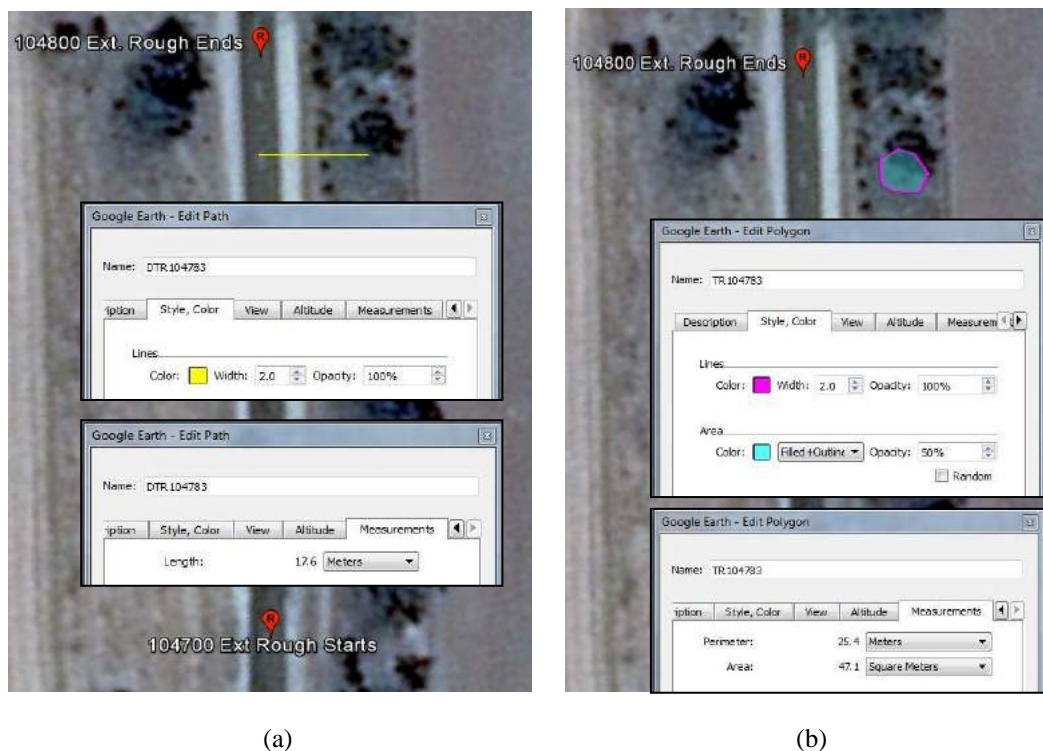


Figure 3. Example measurements (a) offset distance between tree and center-line of pavement, and (b) tree canopy area using polygon method

The measurement of tree height was performed using both satellite and street view type images in Google Earth Pro. First, satellite imagery was used to position a small polygon near the base of the tree (using 3D Polygon tab). A narrow or slender polygon is preferable as it is easier to position. The sides of the polygon then need to be extended to the ground using the scale, and the height of the polygon adjusted. The style and color of the polygon can be modified for better visibility depending on the image. Street view is then used to adjust or fine-tune the height (if required) of the polygon to match the height of the tree (see Figure 4). The accuracy of this approach may depend on the initial positioning of the polygon and visual adjustment of the height. The closer the position of the polygon to the base of the tree, the greater the level of accuracy will be achieved. Figure 4 (a) shows the positioning of the 3D polygon near the tree base and Figure 4 (b) shows the adjustment and fine tuning of established polygon to determine tree height. The height of the tree was measured as 9 m.

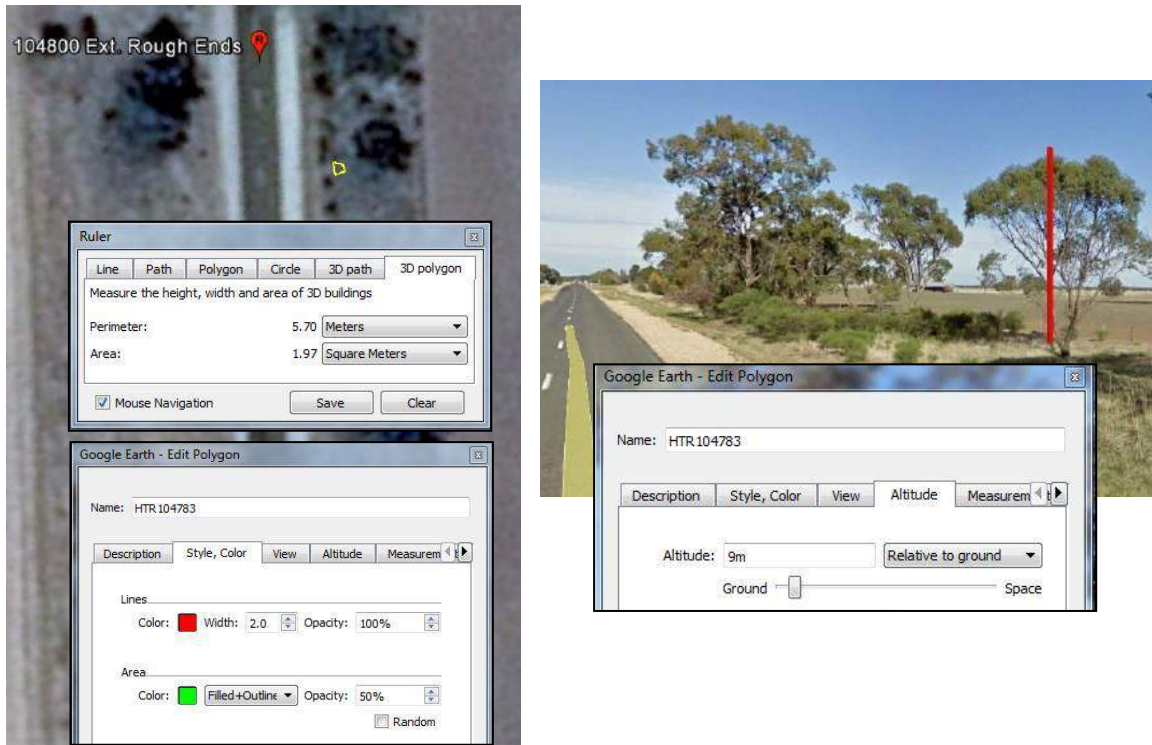


Figure 4 (a) Positioning of 3D polygon near the tree base, and (b) Adjustment and fine tuning of 3D polygon to measure the height

As the grass density within each pavement shoulder may influence pavement roughness, a rating system (using values between 0 and 5) was employed. A zero rating indicated no grass present while a rating of 5 indicated a shoulder fully occupied by grass. The rating system was necessary as it converted the approximate density of grass into a metric scale. The rating of shoulder vegetation was performed using Google Earth street view. The scrolling function within Google Earth street view allowed each shoulder to be rated with a reasonable degree of accuracy. Figure 5 presents an example of the density rating system. Here, the density rating for the left and right shoulders was recorded as 2 and 1 respectively. This is because the grass started to encroach into the left shoulder, while nearly no grass was present on the right shoulder.



Figure 5. An example of grass density rating for the left and right shoulder using Google Earth street view

DATA ANALYSIS AND RESULTS

As the objective of this study was to evaluate the influence of roadside vegetation (namely trees) on roughness levels, all roadside vegetation variables and road roughness indices were statistically evaluated. To achieve this, all roadside tree parameters were set as independent variables and the roughness indices were set as the dependent variable. Unfortunately, no statistical correlation was evident between the roadside tree variables and the individual roughness levels. This was most likely due to the fact that road pavements naturally deteriorate over time due to a number of different factors and pavement age was not considered in this analysis. Thus, to remove pavement age (or time factor) from this analysis, we evaluated the roadside tree and vegetation variables against roughness progression rates for both IRI and Butterworth waveband roughness indices instead. Roughness progression rates (RPRs) are average change in roughness level per year and represents the deterioration of pavement. The RPRs were divided into pre-maintenance and post maintenance rates.

The independent roadside tree and vegetation variables evaluated in this study were (i) Number of Trees (NoT), (ii) Average Distance between Tree and center-line of Pavement (AvDTrPv), (iii) Average Canopy Area (ACA), (iv) Average Tree Height (ATH), and (v) Average Grass Density (AGD) rating. Trees were considered up to a distance of 51.5 m from the center-line of the pavement, and tree height ranged from 0.5 to 24 m. In addition, the statistical analysis was performed in three modes. First, only the roadside tree population on the “near” side adjacent to the surveyed lane was considered. Then, only roadside trees on the far (opposite) side were evaluated. Finally, all roadside trees (both sides of the surveyed lane) were considered and evaluated with the lane roughness progression rates. The results of the statistical analysis using roughness progression rates for IRI and Butterworth indices (DOB3, DOB4, and DOB5) yielded much better correlations than roughness levels, which is presented in Tables 2 and 3. It is important to note that the statistical correlation between roadside trees and roughness progression rates was greatest when lane roughness indices were used rather than individual wheel paths indices.

Table 2. List of independent roadside vegetation variables and associated correlations with overall lane roughness progression rates (for IRI) as well as relevant prediction equations

		Ind. Variables	p	(r)	Prediction Equations
IRI	Near Side Trees Only	NoT	0.222	0.118	
		AvDTrPv	0.227	0.117	
		ACA	0.101	0.158	
		ATH	0.175	0.131	
		AGD	0.142	0.141	
	Far Side Trees Only	NoT	0.392	0.083	
		AvDTrPv	0.835	0.02	
		ACA	0.634	0.046	
		ATH	0.513	0.063	
		AGD	0.625	0.047	
	All Trees (Both Sides)	NoT	0.111	0.154	
		AvDTrPv	0.015	0.233	$RPR_{IRI,LANE} = 0.241 - 0.006AvDTrPv$
		ACA	0.198	0.124	
		ATH	0.02	0.223	$RPR_{IRI,LANE} = 0.219 - 0.012ATH$
		AGD	0.17	0.132	
	AvDTrPv, ATH	0.006	0.302	$RPR_{IRI,LANE} = 0.321 - 0.005AvDTrPv - 0.1ATH$	

Table 3. List of independent roadside vegetation variables and associated correlations with overall lane roughness progression rates (for DOB3, DOB4 and DOB5) as well as relevant prediction equations

		Ind. Variables	p	(r)	Prediction Equations
DOB3	Near Side Trees Only	NoT	0.202	0.123	
		AvDTrPv	0.155	0.137	
		ACA	0.095	0.161	
		ATH	0.095	0.161	
		AGD	0.288	0.103	
	Far Side Trees Only	NoT	0.398	0.082	
		AvDTrPv	0.599	0.051	
		ACA	0.646	0.045	
		ATH	0.851	0.018	
		AGD	0.526	0.061	
	All Trees (Both Sides)	NoT	0.104	0.157	
		AvDTrPv	0.015	0.233	$RPR_{DOB3,LANE} = 0.106 - 0.003AvDTrPv$
		ACA	0.087	0.165	
		ATH	0.009	0.251	$RPR_{DOB3,LANE} = 0.102 - 0.006ATH$
		AGD	0.249	0.111	
	AvDTrPv, ATH	0.003	0.32	$RPR_{DOB3,LANE} = 0.147 - 0.005AvDTrPv - 0.002ATH$	
DOB4	Near Side Trees Only	NoT	0.28	0.104	
		AvDTrPv	0.298	0.101	
		ACA	0.094	0.161	
		ATH	0.148	0.14	
		AGD	0.13	0.146	
	Far Side Trees Only	NoT	0.515	0.063	
		AvDTrPv	0.798	0.025	
		ACA	0.861	0.017	
		ATH	0.988	0.001	
		AGD	0.545	0.059	
	All Trees (Both Sides)	NoT	0.184	0.128	
		AvDTrPv	0.031	0.207	$RPR_{DOB4,LANE} = 0.13 - 0.003AvDTrPv$
		ACA	0.144	0.141	
		ATH	0.01	0.246	$RPR_{DOB4,LANE} = 0.132 - 0.007ATH$
		AGD	0.147	0.14	
	AvDTrPv, ATH	0.007	0.3	$RPR_{DOB4,LANE} = 0.18 - 0.002AvDTrPv - 0.006ATH$	
DOB5	Near Side Trees Only	NoT	0.314	0.097	
		AvDTrPv	0.417	0.079	
		ACA	0.124	0.148	
		ATH	0.227	0.117	
		AGD	0.055	0.184	
	Far Side Trees Only	NoT	0.404	0.081	
		AvDTrPv	0.694	0.038	
		ACA	0.598	0.051	
		ATH	0.498	0.066	
		AGD	0.671	0.041	
	All Trees (Both Sides)	NoT	0.157	0.136	
		AvDTrPv	0.029	0.209	$RPR_{DOB5,LANE} = 0.135 - 0.003AvDTrPv$
		ACA	0.252	0.111	
		ATH	0.014	0.234	$RPR_{DOB5,LANE} = 0.133 - 0.007ATH$
		AGD	0.092	0.162	
	AvDTrPv, ATH	0.008	0.294	$RPR_{DOB5,LANE} = 0.186 - 0.003AvDTrPv - 0.007ATH$	

The statistical results showed that when we consider only the near side tree population or far side tree population alone, the relationship with lane roughness progression rate is not significant for any of the independent vegetation variables. However, when the total tree population (on both sides) was considered,

the significance in correlation improved and became significant. The greatest correlations were recorded for AvDTrPv and ATH against the lane RPR. It was observed that in all cases, AvDTrPv and ATH produced the best correlation with lane RPR regardless of the type of indices used (IRI or Butterworth waveband indices). The Pearson r value for all these correlations was between 0.2 and 0.32, which indicated a small to medium effect (Field, 2009). But, these correlations were found as being significant with p values less than 0.05 achieved. This is a clear indication that a relationship exists between lane roughness progression rates and roadside tree variables like AvDTrPv and ATH. All prediction equations developed to predict RPR with respect to AvDTrPv and ATH have also been tabulated and recorded in Tables 2 and 3.

All RPR prediction equations based on AvDTrPv and ATH all show a negative relationship, which is common for both IRI and Butterworth waveband indices. These negative relationships indicate that: (i) the further the position of tree from pavement, the lower the expected roughness progression rate, and (ii) the greater the height of the tree, the lower the roughness progression rates. The first finding supports the current belief in the literature that the closer the tree the greater the moisture variation in the ground will occur and the greater the ground movement. However, the second finding goes against current literature as it is commonly believed that the bigger the tree, the more widespread and established the root system, and thus the greater the influence in moisture variation and the greater the ground movement. This was a surprising result, which could be explained by the fact that the smaller trees (being younger trees) need greater moisture to establish their growth cycles and cell development. Thus, the smaller trees contribute more to roughness progression within the nearby pavement.

CONCLUSIONS

The overall results obtained in this study proved that trees contribute to pavement roughness progression for those founded on expansive soils. A weak to medium correlation relationship was found to exist between roadside tree variables and roughness progression rates. The best correlations were found between roughness progression rates and (i) relative distance between the tree and center-line of the pavement, and (ii) the height of the trees. In both these cases the correlation was found to be negative and was common for both IRI and individual Butterworth waveband indices. Even though the correlations attained were not convincing, this might have occurred as other variables such as pavement age, annual average daily traffic (AADT) and pavement distress features such as cracking, rut depth, climate, topography and geological origin were not considered in this model. Moreover, many parts of the Borung highway have been found to be affected by Gilgai development, and the undulating pattern of such geological formation is expected to influence the growth of roughness progression rates. As a weak to medium relationship was established between tree variables and roughness development, the overall roughness progression rates could have resulted from the combined effect of many variables. Thus, in the future, it is intended to merge this database with the abovementioned “traditional” pavement deterioration variables to enhance current pavement deterioration models in areas of expansive soils with roadside tree plantations.

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PAPER TITLE	Asset Management to Highway: Soil-Cement Stabilization Technique		
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KEYWORDS:

Soil-cement Stabilization; Soft Ground Treatment; Roadbase.

ABSTRACT:

The North South Expressway section in Kedah was partly built on an overlying soft clay formation. The ground stabilization technique, namely stone column piles, was implemented to construct denser roadbase layer. However, consolidation and creeping processes had resulted in uneven settlement on treated layer thus undulating the pavement surface or creating an effect known as “mushroom”. The common pavement treatment of regulating, mill and pave to the undulated surface proves to be short term measures. In this paper, an innovative treatment method using alkaline compound was introduced to alleviate these issues at the Alor Setar Selatan Interchange ramps. Utilizing mostly alkaline earth elements, this technique recycles existing roadbase material by mixing it with cement additive to stimulate direct hydration processes to form a stiffer roadbase foundation. After curing it with adequate moisture content, upper layers of binder and wearing course are then applied. This treatment has effectively improvised roadbase compressive strength and bearing capacity.

Asset Management to Highway: Soil-Cement Stabilization Technique

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1. INTRODUCTION

The northern part of North-South Expressway that traverses the state of Kedah is mostly rested on the paddy field plain. Since its construction back in 1986, several locations have experienced uneven ground settlement and caused undulation, depression and cracks on the pavement surface. This condition may endanger the highway users during heavy rain due to hydroplaning effects as a result of water lying stagnant within the settlement area.

Undulation and uneven ground settlement on the highway surface is significantly influenced by soft ground condition with low compression and stiffness characteristics. Conventionally, for the ground stabilization technique, stone column piles were applied to construct denser road base layer. However, consolidation and creeping processes had resulted in uneven settlement on treated layer thus undulating the pavement surface or creating an effect known as “mushroom”. The highway pavement rehabilitation by applying the Cement-Bound Material (CBM) was introduced to extend the life span of the pavement structure by recycling the existing roadbase material (Chai et al., 2005). From this stabilization technique, the cement effect significantly improves the stiffness properties of unbound layer, thus providing better load transfer to the pavement foundation.

The factors that influence the physicochemical reactions of soil-cement hydration and the interactions of peat soil-cementation process are the amount of solid particles, the water-soil ratio, the quantity of binder, the presence of humic and/or fulvic acids, the soil pH and the amount of organic matter in the peat (Zulkifley et al., 2014). According to Wu et al. (2016), the applicability and the effectiveness of soil-cement stabilization technique in soft ground improvement are mainly evaluated by the compactability and strength performance.

2. DESIGN CRITERION

2.1 Existing Roadbase Identification

Prior to the determination of the design mix, samples of the existing roadbase was taken for identification and laboratory identification tests. The respective tests being conducted were Particle Size Distribution (PSD) for coarse and fine grained soils (BS 1377: Part 2: 1990: 9), moisture content determination (BS 1377: Part 2: 1990: 3) and Proctor test (BS 1377: Part 4: 1990) to determine the maximum dry density and optimum moisture content.

From the PSD qualification result, the existing roadbase material had mixed with subgrade material whereby a significant amount of silt and sand had been identified with the percentage of fine material (i.e. less than 0.8mm) not exceeding 10% (Figure 1), conforming to Arahan Teknik Jalan 5/85 (2013). While, the result for proctor test on the same existing roadbase was recorded 2.08 Mg/m³ of maximum dry density (Figure 2) which surpassed the minimum requirement of 1.3 Mg/m³ thus showing a very good material as a soil-cement stabilization medium. Hence, the preliminary stage for existing roadbase condition had found that additional fine material (i.e. quarry dust) shall be combined to improve the PSD curve grading. Furthermore, a minimum of 8% cement and 2% alkaline compound as the stabilizing agent shall act as a basis for the proposed mix design to display higher stiffness and strength properties.

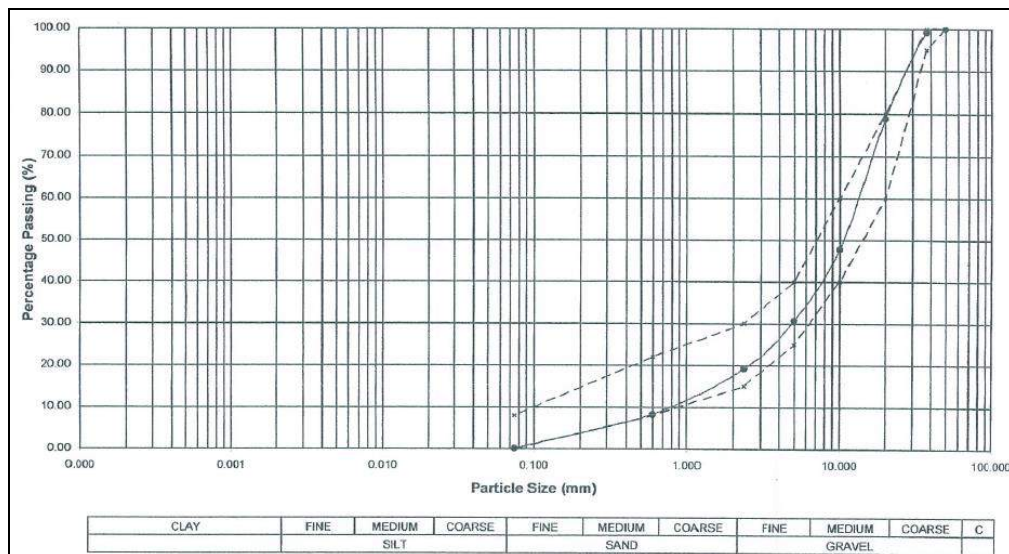


Figure 1. Grading characteristic for existing roadbase

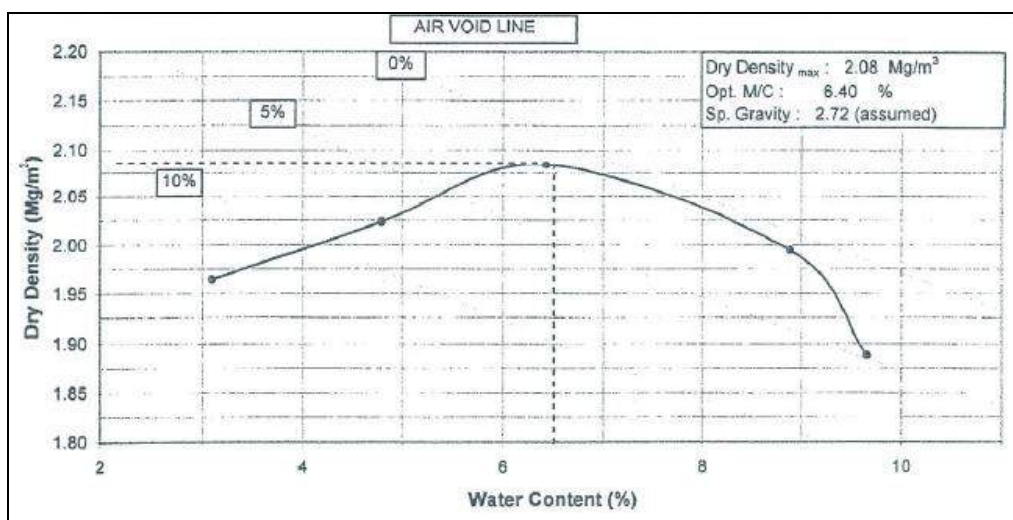


Figure 2. Proctor test result for existing roadbase

2.2 Design Application

The thickness of the layer to be treated by soil-cement stabilization is determined by the traffic category and equivalent standard axles as tabulated in Table 1. The stabilization thickness of 300 mm is determined in accordance with T4 traffic category. The recommended mix design then refers to Qualification test as per Soil Cement Specification, i.e. relationship between mix design and soil classification. This relationship then introduces the composition of Ordinary Portland Cement (OPC) mixed with alkaline compound that promotes cement hydration process, thus inhibiting the actions of fulvic acids and carbonic acids. Technically, this compound result in roadbase structural changes and the formation of minerals occurring during cement hydration greatly increases the compressive strength, the static and dynamic stiffness modulus, the tensile bending strength, besides stabilizing humus-rich soils (Suddath and Thompson, 1975; Wu et al., 2016).

Table 1. Relationship table between traffic categories with roadbase stabilization thickness (Arahan Teknik Jalan 5/85, 2013)

Traffic Category	Design Traffic (ESAL X 10 ⁶)	Depth Stabilization Thickness
T1	≤1.0	200 mm
T2	1.1 to 2.0	250 mm
T3	2.1 to 10.0	275 mm
T4	10.1 to 30.0	300 mm
T5	>30.0	≥300 mm

The amount of OPC and alkaline compound to be added to the mixture for stabilization was decided based on soil matrix classification (Table 2) that leads to the design of initial water content and amount of OPC and alkaline compound. This amount highly influences the rate of pozzolanic reaction that progressively depends on mineralogy of the soil and pH level. According to Little et al. (2009), sufficient high pH is maintained to solubilize silicates and aluminates from the clay matrix or fine silt soil. This reaction then forms calcium-silicate-hydrates and calcium aluminate-hydrates, a similar compound produced during the strength development in the hydration of Portland cement.

This non-binding material type of soil requires the amount of OPC and alkaline compound in the range of 140 kg/m³ to 180 kg/m³. For confirmation, the proposed mix design will be assessed by the seventh day Unconfined Compressive Strength (UCS) test (BS 1377: Part 7:1990) for bearing capacity.

Table 2. Soil matrix classification relation to mix design

	Soil Class		Initial Water Content	Normal Amount of OPC + Alkaline Compound
Non-Binding	GW	Well Graded gravels and gravel-sand mixture, little or no fine	From 0 to 15-20%	From 140 kg/m ³ to 180 kg/m ³
	SW	Well Graded sand and gravelly sand, little or no fine		
	GP	Poorly Graded gravels and gravel-sand mixtures, little or no fine		
	GM	Silty Gravels, gravel-sand-silt mixtures		
	GC	Clayey Gravels, gravel-sand-clay mixtures		
	SW	Well Graded Sand and gravelly sands, little or no fine		
Mixed Grains	SP	Poorly Graded Sand and gravelly sands, little or no fine	From 0 to 30-35%	From 160 kg/m ³ to 190 kg/m ³
	SM	Silty Sands, sand-silt mixtures		
	SC	Clayey Sands, sand clay mixtures		
Binding	ML	In organic silts, very fine sands, rocks, silty or clayey fine sand	From 0 to 15-30%	From 170 kg/m ³ to 220 kg/m ³
	CL	Inorganic clays of low to medium plasticity, gravelly/ sandy/ silty/ lean clays		
	OL	Organic silts and organic silty clays of low plasticity		
	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts		
	CH	Inorganic clays or high plasticity, fat clays		
	OH	Organic clays of medium to high plasticity		
Organic	Pt	Peat, muck and other highly organic soils	Import material recommended to be installed before stabilization	

2.3 Seventh Day Unconfined Compressive Strength (UCS) Test

This test was carried out to estimate and certify the bearing capacity of the proposed mix design and to choose the final mix design. The intended sample was prepared by mixing the existing material based on the ratio predetermined with 8% OPC and 2% alkaline compound. Cylinder of this mix of materials was prepared and cured for 7 days prior to the execution of the UCS test. Based on the results obtained (Table 3), the proposed mix design had surpassed the minimum strength of 2.5 MPa that has been set by JKR/SPJ/2008-S4, hence the ratio proportion for OPC and alkaline compound is satisfied.

Table 3. Summary results of UCS test after 7 days of curing

Sample No.	Age (days)	Maximum Load (kN)	Compressive Strength (MPa)
1	7	70.80	3.15
2	7	77.00	3.42
3	7	66.30	2.95

3. CONSTRUCTION APPLICATION

The pavement roadbase rehabilitation construction works by applying soil-cement stabilization technique was successfully completed at the respective ramp from Alor Setar Utara to Alor Star Selatan toll plaza. In this project works, the rehabilitation phases started from milling-off the existing pavement surface not exceeding 150 mm of thickness (Figure 3). Prior to stabilization works by mixing the existing roadbase material with OPC and alkaline compound (i.e. Geocrete), quarry dust was spread equally over the stabilized area (Figure 4). The mixing work was done by special rotary mixer machine, CAT RM500 that has the capability of ensuring a uniform blend of stabilization material with the set of 300 mm depth. Water was then spread uniformly over the mixed soil-cement-alkaline compound to start the chemical reaction for this stabilization process (Figure 5). Then, the mixture was compacted with 10 tonnes drum roller without vibration for two passes, followed by another compaction using three passes of vibration roller with the speed of not more than 3 km per hour (Figure 6).

Finally, before laying the bituminous layers, the curing process was performed by spraying water uniformly on the stabilized soil periodically for every three to four hours in three days. This is a process to ensure that no premature cracking is formed by controlling hydration during the stabilization process.



Figure 3. Milling process to scrape-off the existing asphalt layers



Figure 4. (a) Spread layer of quarry dust; and (b) positioning of cement and alkaline compound packaging to ensure an evenly distribution over the stabilized area



Figure 5. Mixing process of mixed soil-OPC-alkaline compound by special rotary mixer machine, CAT RM500



Figure 6. Compaction process by a 10 tonnes drum roller without and with vibration

4. POST-CONSTRUCTION RESULTS

Two types of monitoring test procedures were conducted after the completion of construction works, i.e. UCS test after 28 days and Light Falling Weight Deflectometer (LFWD) test. As tabulated in Table 4, results confirmed that the application of soil-cement stabilization technique has achieved the expected strength and the project objective by complying with the requirement greater than 2.5 MPa.

Table 4. Summary for UCS test results after 28 days

Sample No.	Diameter (mm)	Length (mm)	Sample Mass (g)	Density (kg/m ³)	Maximum Force (kN)	UCS (MPa)
1	94.0	323	1338.9	2113	73.0	10.5
2	94.0	318	1378.9	2094	105.0	15.1
3	94.0	334	1346.4	2098	106.0	15.3

LFWD tests were performed according to ASTM E2835 - 11(2015) to monitor the bearing capacity and the strength behaviour of unbound pavement material by using Light Drop Weight Tester ZFG2000 (Figure 7). This quick and cost effective method is to evaluate the dynamic deflection modulus, E_{vd} in MN/m². This modulus is an index for the bearing capacity of roadbase layer, and calculated from the measured settlement(s) which equivalent to the results obtained from static plate bearing test (BS 1377 Part 9: 1990) recorded for existing pavement; i.e. 46.03 MN/m².

Figure 8 shows an illustration for the summary average results of LFWD up to 28 days after the construction had been completed. For comparison purposes, the data of the 1st, 3rd, 7th and 28th days after stabilization were compared again with the result of existing roadbase surface. The readings recorded after stabilization on the 1st, 3rd, 7th and 28th days age were 109.08 MN/m², 140.45 MN/m², 159.46 MN/m² and 182.39 MN/m² respectively. It evidently shows an increase of 237% from the 1st day up to 396% on the 28th day, a significant improvement in roadbase bearing capacity index which was achieved in the post construction.



Figure 7. In-situ practical of Light Falling Weight Deflectometer (LFWD) test

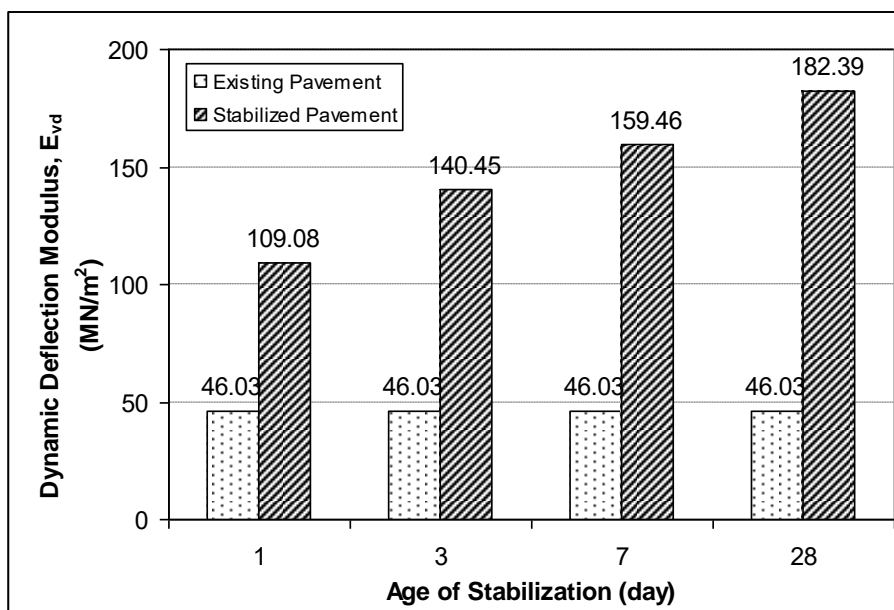


Figure 8. Summary of result for dynamic deflection modulus with respect to age of stabilization

5. CONCLUSIONS

The application of soil-cement stabilization technique with the incorporation of quarry dust and alkaline compound (e.g. Geocrete) with designated proportion has resulted in high value bearing capacity and is more effective than the conventional regulating during the mill and pave of the undulated surface of highway pavement. In tandem with soil-cement-alkaline stabilization design approach, a high performance of roadbase in term of compressive strength and high resilient modulus can be expected as a solution for unstable pavement.

From the mix design of 8% and 2% for OPC and alkaline compound respectively, the average results of 13.63 MPa for compressive strength and 396% increment in bearing capacity as compared to existing roadbase reading of LFDW for after 28 days were achieved. Subsequently, this stabilization technique can lessen the frequency of maintenance cycles, which makes it an effective and practical engineering solution especially for highway maintenance.

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PAPER TITLE	Removal method for ceiling board of the Hanshin Expressway's tunnels		
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KEYWORDS:

Hanshin Expressway
ceiling board of tunnel
removal works
Kobe Nagata tunnel
Shin-Kobe tunnel

ABSTRACT:

In December 2012, a traffic accident caused by fall of the ceiling board occurred inside a tunnel in Japan. We checked the tunnel in the Hanshin Expressway network where a similar type of ceiling board was installed. As the result of safety evaluation, we made a judgement that the ceiling board would be able to serve sufficiently for some more years. However, we decided to remove the ceiling board in consideration of risks assumed in the course of operation and maintenance. In order to minimize inconvenience to users, the removal works was carried out during a limited days. This paper introduces the outline of the removal works.

Removal method for ceiling board of the Hanshin Expressway's tunnels

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1 INTRODUCTION

In December 2012, a traffic accident caused by fall of the ceiling board occurred inside a tunnel in Japan. Following this incident in Sasago tunnel, we checked the tunnel in the Hanshin Expressway network where a similar type of ceiling board was installed. As the result of safety evaluation, we made a judgement that the ceiling board would be able to serve sufficiently for some more years.

However, we decided to remove the ceiling board in consideration of risks assumed in the course of operation and maintenance. In order to minimize inconvenience to users, the removal works was carried out during a limited days. This paper presents the methods of removal of the ceiling boards.

2 KOBE NAGATA TUNNEL AND SHIN-KOBE TUNNEL

Hanshin Expressway Co. Ltd. is in charge of the administration of expressways of 259.1 km in total length (Figure 1), which consists of elevated structures of 209.1 km, underground structures of 28.1 km and others of 21.9 km. Of these, Kobe Nagata Tunnel and Shin-Kobe Tunnel (northbound) had ceiling boards installed.

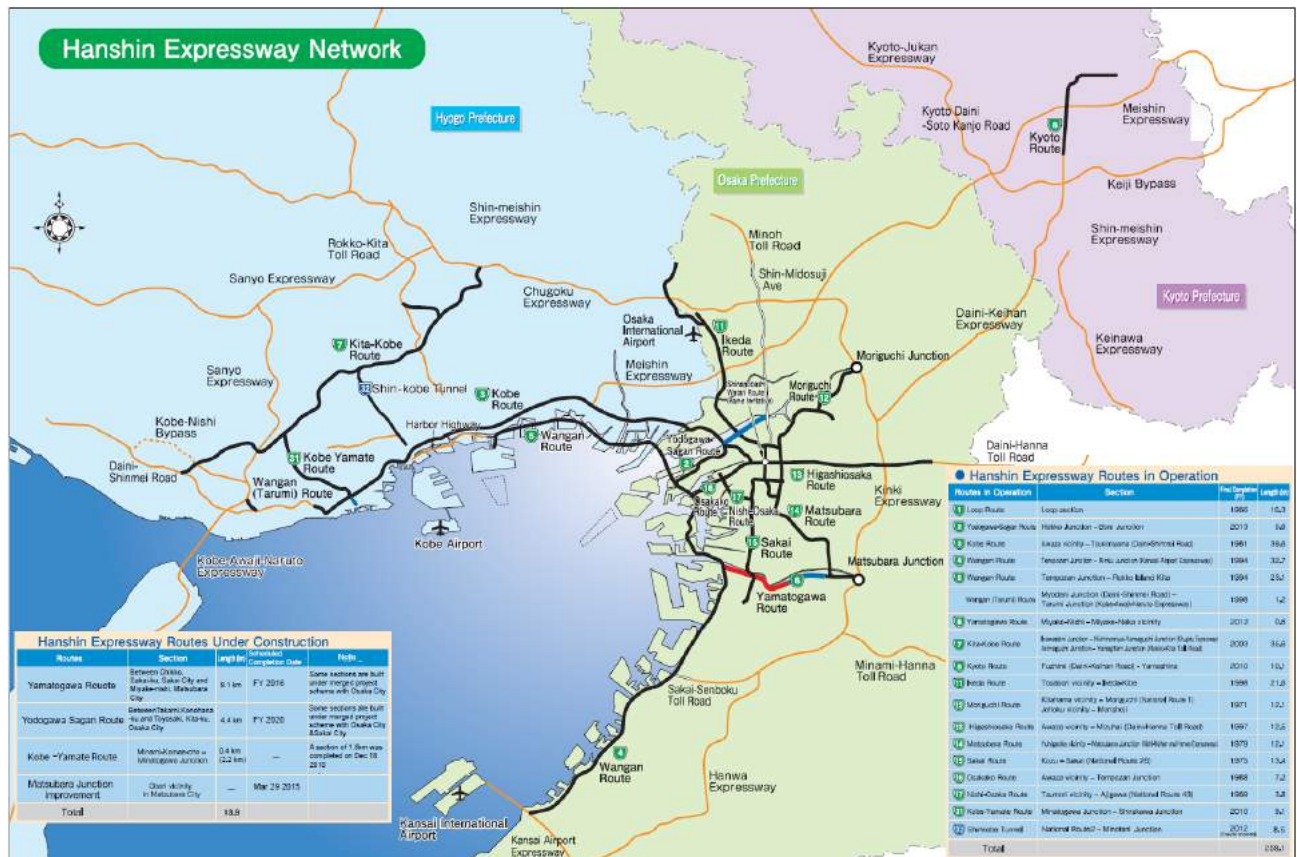


Figure 1. Hanshin Expressway network.

Shin-Kobe Tunnel (northbound) is a two-lane one-way tunnel with a length of 6,910 m. When it was first brought into service in 1976, it was a two-way tunnel with one lane for each direction, which is one factor that led to the employment of a transverse ventilation system, and ceiling boards were installed at the top of the tunnel cross section to ensure space for supply and discharge of air. Subsequently in 1988, the southbound tunnel was opened and the respective tunnels are now in service as one-way northbound and southbound tunnels.

Kobe Nagata Tunnel is composed of two tunnels of 3,906 m northbound and 3,364 m southbound, each of which is a two-lane one-way tunnel. For the Tunnel, a transverse ventilation system was employed from the first construction and the structure included suspended ceiling boards to ensure space for supply and discharge of air.

3 STRUCTURE OF CEILING BOARDS OF KOBE NAGATA TUNNEL

The ceiling boards of Kobe Nagata Tunnel were structurally supported by metal fittings suspended from the tunnel lining using post-construction anchor bolts and the ceiling boards were installed by inserting at the lower end of the metal fittings (Figure 2). For these ceiling boards, extruded cement panels containing asbestos (non-scattering) were mainly used. For preventing falling of the ceiling boards, a backup structure using fall prevention wires was installed.

On these metal fittings for suspension at the center (hereafter referred to as "central metal fittings"), ALC panels were installed as partitions between the air supply duct and air discharge duct. The partitions were installed on the central metal fittings by using M12 hexagon head bolts.

At the side ends of the ceiling boards, mortar was poured for filling the gap between the lining and ceiling boards.

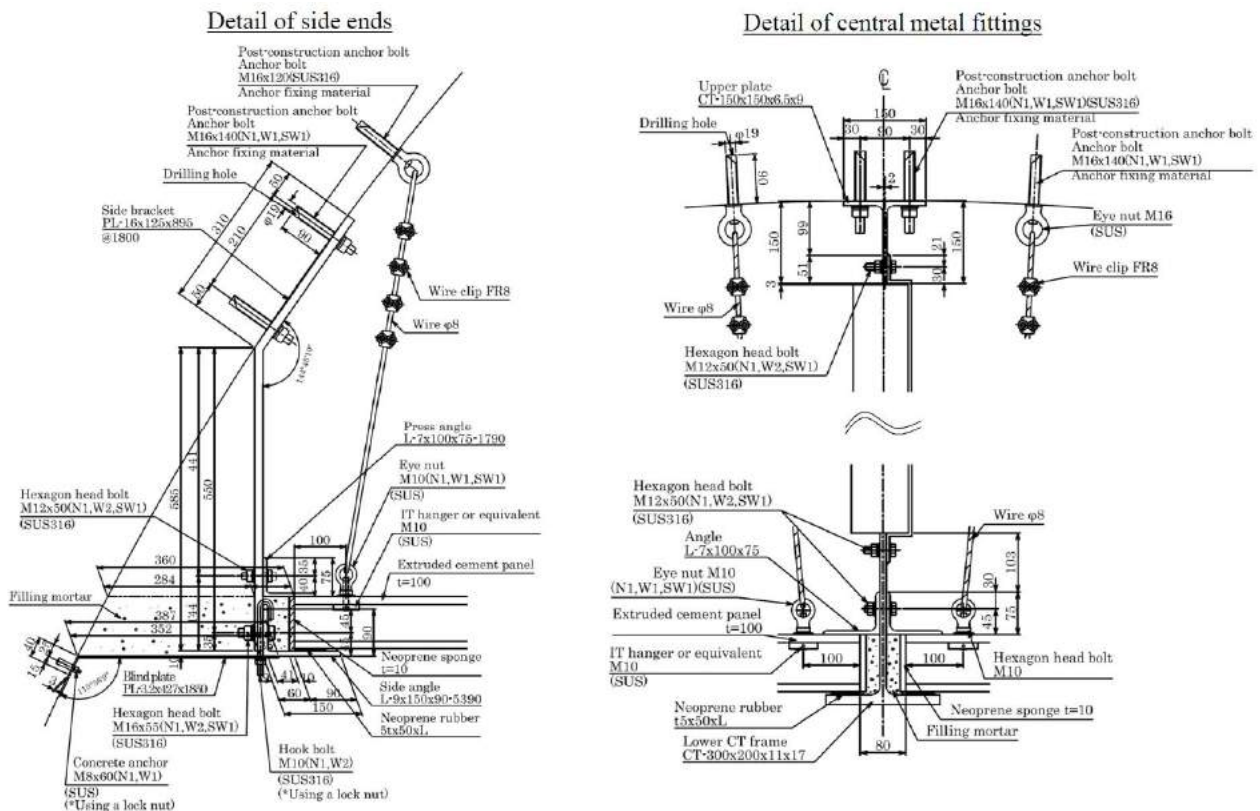


Figure 2. Structure of ceiling boards of Kobe Nagata Tunnel.

4 STRUCTURE OF CEILING BOARDS OF SHIN-KOBE TUNNEL

Unlike the suspended structure using post-construction anchor bolts of Sasago Tunnel, the ceiling boards of Shin-Kobe Tunnel have a structure self-supported by arched reinforced concrete ($t = 150$ mm) as shown in Figure 2 and the inside was divided by partitions into the air supply and discharge spaces. The emergency stopping lane had the structure supported by the suspension members embedded in the lining, as shown in Figure 3.

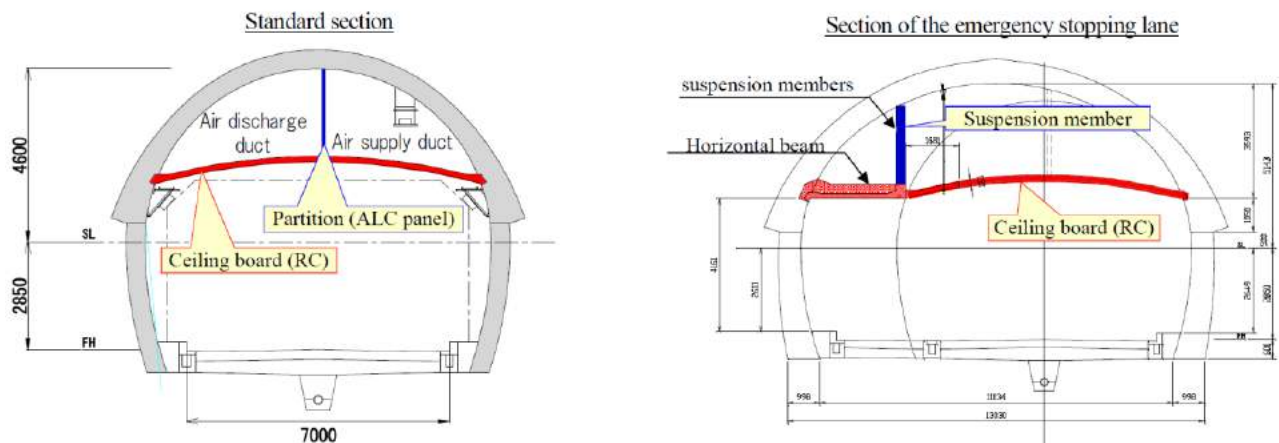


Figure 3. Structure of ceiling boards of Shin-Kobe Tunnel.

5 CEILING BOARD REMOVAL METHOD FOR KOBE NAGATA TUNNEL

Plans for the removal of the ceiling boards of Kobe Nagata Tunnel were made mainly including complete closure of the Tunnel for all days. Closure for 14 days was required for completing the removal of all ceiling boards.

For removing the ceiling boards, the works were conducted by assigning the operations to different work teams mainly for the ceiling boards, partitions and metal fittings for suspension (central and end) in this order to complete the removal. The following describes the procedure.

(1) Removal of ceiling boards

Ceiling boards were removed by using a handling machine equipped with a special device on the backhoe. This device is capable of rotating the tips of the arms that grip an object, which can be used to avoid the mounting metal fittings to remove ceiling boards as shown in Figure 4. The ceiling boards were installed separately for the driving lane and passing lane sides bordering the central metal fittings and, to maintain the stability of the ceiling boards during the removal, two boards were removed each time while alternating the sides. The ceiling boards removed by the handling machine were transferred onto a forklift for temporary placement and carried outside.

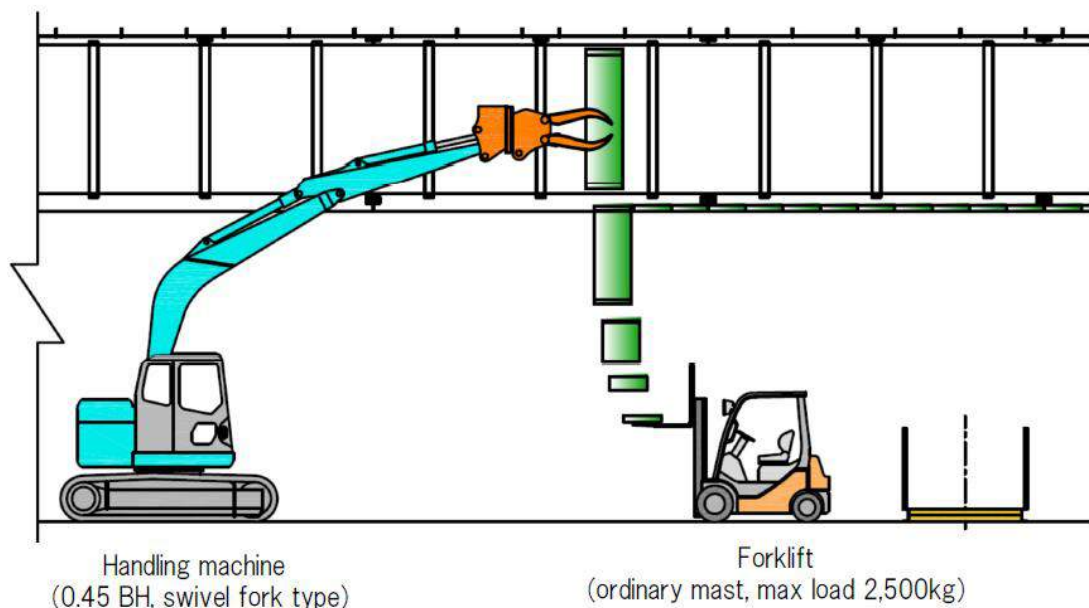


Figure 4. Removal by using handling machine.

(2) Removal of central metal fittings and partitions

After the removal of the ceiling boards, the partitions were removed by using a high place work vehicle to remove the mounting bolts of the partitions (ALC panels) and bringing the ALC panels down into the widened bucket of a wheel loader. Then the metal fittings for suspension (central) were removed.

For removal of the metal fittings for suspension (central), a jig for removing the central metal fittings was mounted on the forklift and, after removing the central metal fittings, a crane was used to lift them from the forklift to temporarily place them on the road surface, where the central metal fittings were disassembled. In view of the overall work process, some of the partitions were removed inside the ceiling boards before the removal of the ceiling boards (Figure 5 and Figure 6).

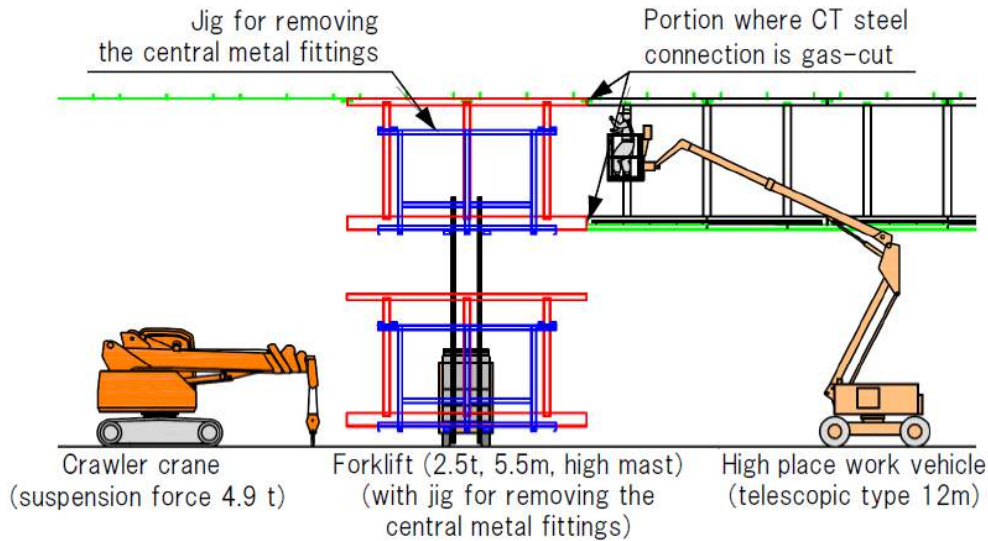


Figure 5. Removal of the central metal fittings.

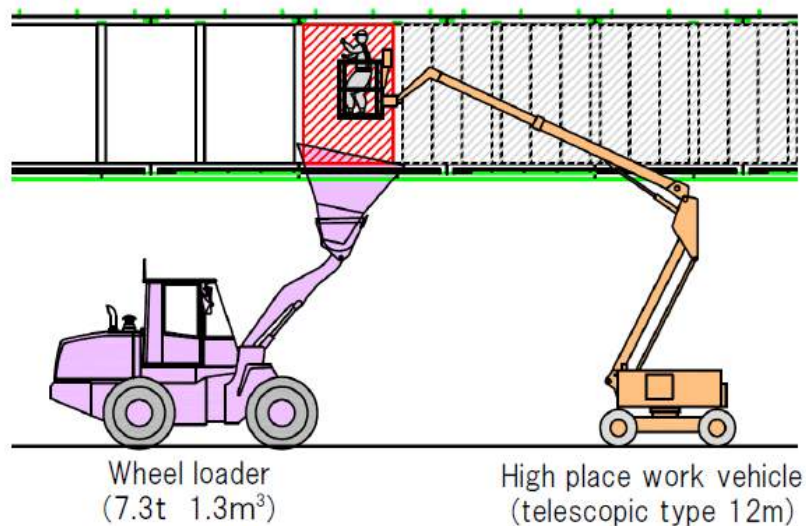


Figure 6. Removal of partitions.

(3) Removal of metal fittings for suspension (side ends)

The pedestals at the both side ends were found to be filled with mortar, and a standard approach based upon this plan was designed to removal of the filling mortar by complete chipping. However, it was also revealed that the filling mortar was shrinkage-compensating type and its removal by complete chipping was difficult. Accordingly, the removal method was changed to simultaneous removal of the filling mortar and steel plates by using a special jig mounted on the forklift to grip the filling mortar and steel plates together.

5 CEILING BOARD REMOVAL METHOD FOR SHIN-KOBE TUNNEL

Ceiling board removal works by complete closure were preferable also for Shin-Kobe Tunnel (northbound). However, unlike Kobe Nagata Tunnel, it is on a principal road used by buses on regular routes and works by complete closure was assumed to cause significant impact on the users. For this reason, the removal of the ceiling boards of Shin-Kobe Tunnel was planned based on implementation of works by closure only during the nighttime and completion of the removal of all ceiling boards required nighttime closure for 40 days.

The hours of the nighttime closure were determined to be 6 hours and 30 minutes from 23:30, which is after the last bus on a regular route passes, to 6:30 in the next morning before the first bus passes. It included the time for preparation and clearing up for lane opening (lifting of the closure) and the time that could be used for the actual removal works was very short at about 4 to 5 hours. Accordingly, how efficiently the removal cycle could be scheduled was an important point. The following describes the removal procedure.

(1) Support and cutting of ceiling boards

The ceiling boards were cut in the transverse direction in advance from the inside of the ceiling boards. Then, a multi-axle carriage was placed at the location of the ceiling board removal, the deck lift was jacked up to support the ceiling boards at the center and the ceiling boards were cut in the longitudinal direction by using a concrete cutter for detaching the central part (Figure 7).

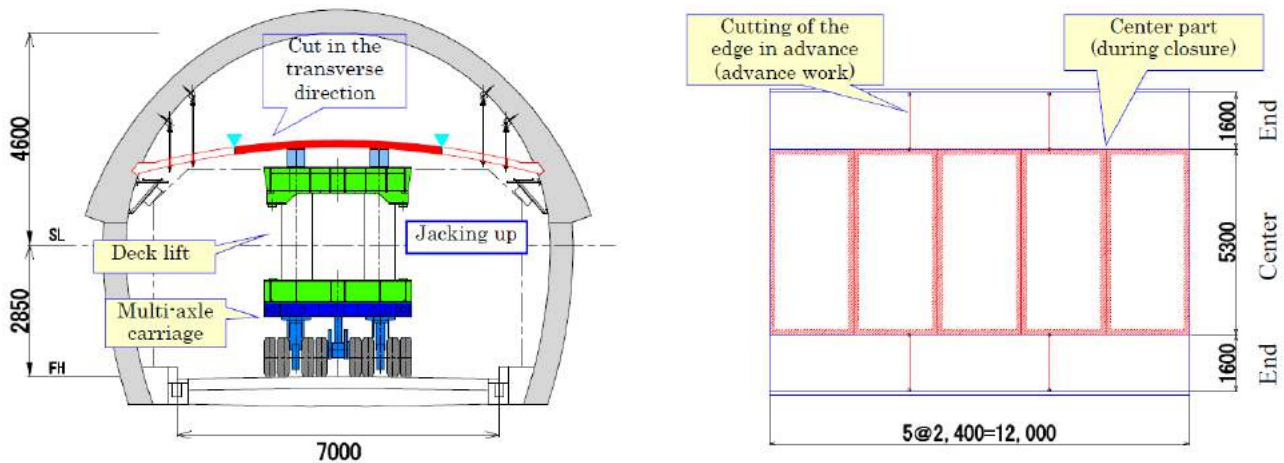


Figure 7. Ceiling board cutting and support by using multi-axle carriage.

(2) Jacking down and moving of multi-axle carriage

The maximum length of a ceiling board that can be supported by the multi-axle carriage at one time is 12 m. The first span to be removed, after its cutting was finished, was placed on the deck, and jack-downed. After then, the carriage was moved to the next span (Figure 8). The next 12 m of the board was supported in such a way that it was placed on top of the ceiling board removed first and cut in the same way. This process was repeated three times and a stack consisting of three ceiling boards each 12 meters, or 36 meters in total was loaded onto the carriage.

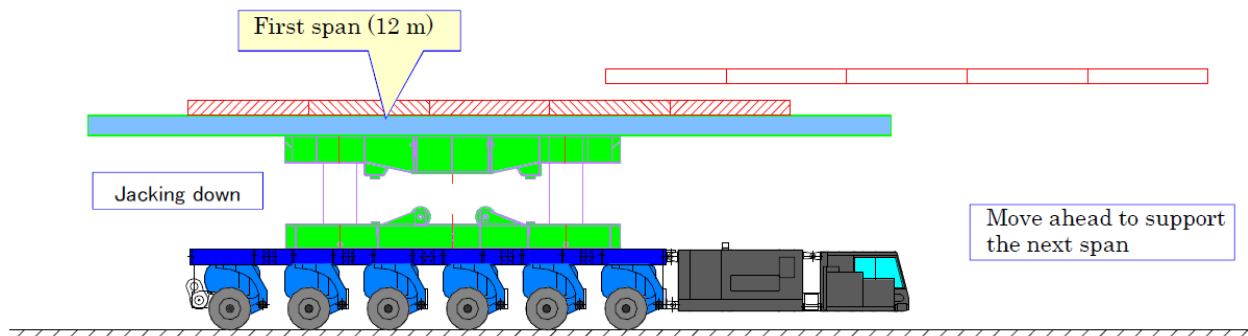


Figure 8. Moving of multi-axle carriage.

(3) Transshipment of ceiling boards from multi-axle carriage to trailer

The multi-axle carriage loaded with the ceiling boards having a total length of 36 m (12 m ceiling boards in three tiers) was moved to the transshipment area and the electric hoist installed on the crown in advance was used to transfer the boards to the trailer for carrying out (Figure 9). The ceiling board, after it had been cut in the traversal direction into panels, were grouped in units of three panels each having the size of 5.4 m x 2.4 m; then this unit, in a state suspended, was turned 90 degrees, and loaded onto the trailer for carrying.

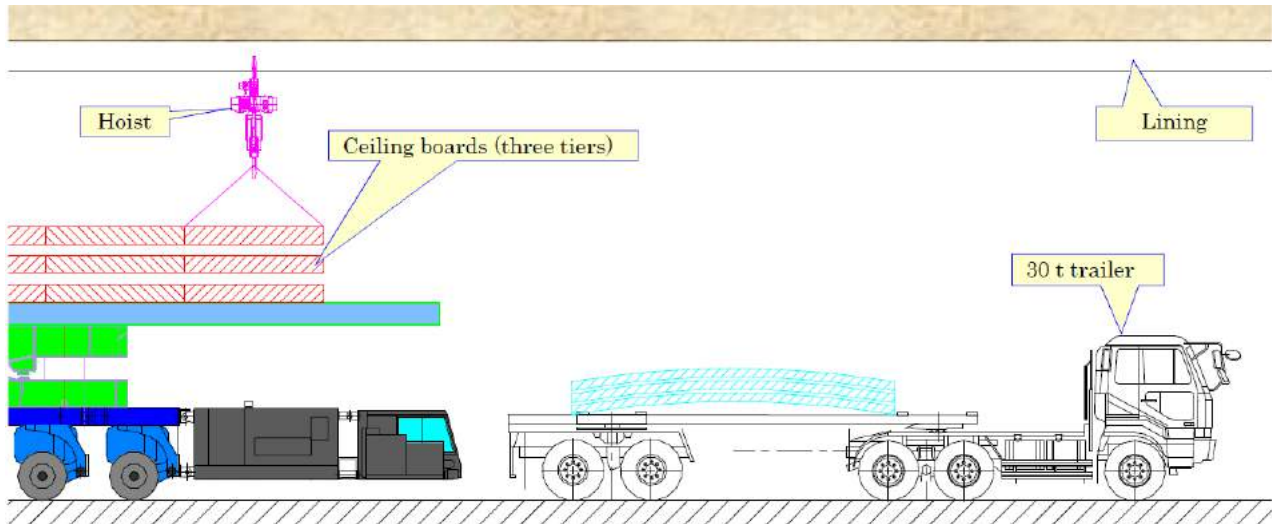


Figure 9. Transshipment of ceiling boards.

(4) Removal of ceiling board side ends

The remaining side ends of the ceiling boards were removed by using a forklift. On the forklift, a special platform was installed and, with a ceiling board end supported from the bottom, the fall prevention (suspending) metal fittings were cut for removal (Figure 10). By carrying out the end removal in parallel with the removal of the central part using the multi-axle carriage, the cycle time was reduced.

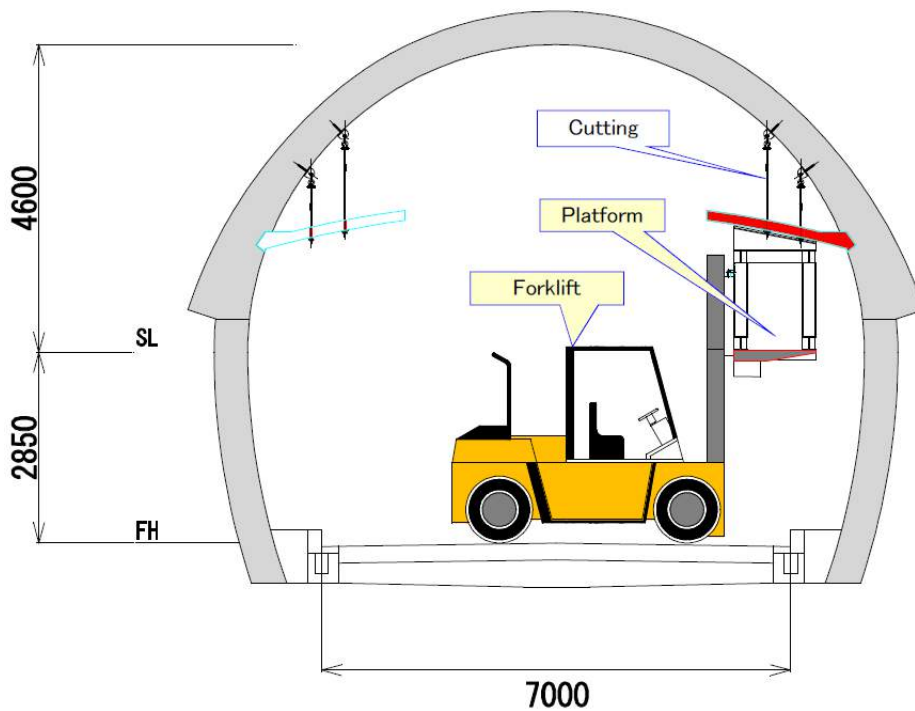


Figure 10. Removal of ceiling board side ends.

6 CONCLUSIONS

Of the tunnels on Hanshin Expressway, Shin-Kobe Tunnel and Kobe Nagata Tunnel had ceiling boards installed and the removal of the ceiling boards was completed in 2016 for Kobe Nagata Tunnel and in 2014 for Shin-Kobe Tunnel (northbound).

For the removal of the ceiling boards, it was necessary to take into account the structure and material of the respective ceiling boards and the restrictions on the work time in view of the impact on the road users. To deal with these issues, we selected appropriate methods with the various conditions taken into consideration and studied removal methods capable of realizing an efficient cycle time of the removal works, which resulted in the successful completion of the ceiling board removal as initially scheduled.

These ceiling board removal works, which were determined in view of long-term safety, leads to the safety, security and comfort of Hanshin Expressway.

2nd IRF Asia Regional Congress & Exhibition
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PAPER TITLE	EFFECT OF LONG-TERM AGING ON PROPERTIES OF APHALT		
TRACK			
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asphalt pavement, aging, FT-IR, stiffness, Dynamic Shear Rheometer

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ABSTRACT:

The properties of asphalt are important in the characteristics of pavement used for long periods. Damage to asphalt pavement is caused by fatigue due to cyclic traffic loads and hardening of asphalt due to oxidative aging. East Nippon Expressway Co., Ltd. has carried out excavation investigation of asphalt pavement on the expressways. In this study, we have collected sample blocks from asphalt pavement under long-term use from expressways. Ten samples were obtained from the following four locations: On the Joshin-Etsu Expressway between the Nagano and Suzakanagano East interchanges (both away from and toward Tokyo), on the Nagano expressway between the Toyoshina and Omi interchanges (away from Tokyo), and on the Nagano expressway between the Omi and Kousyoku interchanges (away from Tokyo). These samples are recorded about the traffic volume and repair history. The samples were then classified based on the damage level, years of service, wheel path location, asphalt mixture type, and other parameters.

This study investigates the effects of traffic and environmental factors on the aging of the asphalt. The objectives of this research were (1) to achieve a better understanding of the aged state of the asphalt mixtures in pavements after long-term use and (2) to simulate the aged state in laboratory tests in order to identify the factors affecting aging. We surveyed the asphalt extracted from the surface course to base course in each blocks. Characterizations were performed on the extracted asphalts to determine how the specimens had aged. Next, the same characterizations were performed on the asphalt, which were subjected to accelerated aging under various conditions.

The results revealed that aging affects not only the surface course of asphalt pavement but also the base course, and the penetration of asphalt in porous asphalt mixture decreased more rapidly than that of dense-graded asphalt mixture.

Moreover Stiffness of asphalt was calculated with a nomograph and measured using a dynamic shear rheometer (DSR). The findings showed a very high correlation between stiffness calculated by nomograph and complex modulus measured by DSR regardless of the loading time and measurement temperature, as well as a very high correlation between complex modulus and loss modulus regardless of loading time, temperature, type of binder, field sample and accelerated aging sample. The relationship between damaged asphalt pavement and the rheological properties of asphalt is discussed.

Effect of Long-term Aging on Properties of Asphalt

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1 INTRODUCTION

Damage to asphalt pavement is caused by fatigue resulting from cyclic traffic loads and asphalt hardening due to oxidative aging (Kasahara et al. 1975; Iijima et al. 1982; Steiner et al. 2015). Previous research (Hofko et al. 2015; Yumura et al. 1980; Abe et al. 1985) has shown that oxidative aging progresses not only on the surface, but also in the downward direction. However, there have been no previous studies presenting detailed surveys of the binder course (or lower courses).

In this study, we extracted sample blocks from asphalt pavement under long-term use from expressways that have extensive traffic volume and repair history records. The samples were then classified based on the damage level, years of service, wheel path locations, asphalt mixture type, and other parameters. Next, we examined asphalt extracted from the surface course, binder course, and asphalt stabilized base course of the each block. Moreover, asphalt stiffness was calculated with a nomograph and measured using a dynamic shear rheometer (DSR). The findings showed a very high correlation between the stiffness calculated by nomograph and the complex modulus measured by DSR, regardless of the loading time and measurement temperature, as well as a very high correlation between complex modulus and loss modulus, regardless of loading time, temperature, binder type, field sample, and accelerated aging sample.

This report presents the results of the investigations described above. The study was carried out as a part of a survey conducted by East Nippon Expressway Co., Ltd. (Takahashi et al. 2013, 2014).

2 OBJECTIVE AND METHODS

The objectives of this research were (1) to achieve a better understanding of the aged state of various asphalt mixtures in pavements after long-term use, and (2) to simulate age states in laboratory tests in order to identify factors that affect aging.

Characterizations were performed on the extracted asphalts in order to determine how the specimens had aged. Next, the same characterizations were performed on the asphalt samples that were subjected to accelerated aging under various conditions.

3 RESULTS OF THE FIELD INVESTIGATION

3.1 TEST LOCATIONS

Ten samples were obtained from the following four locations: the Joshin-Etsu Expressway between the Nagano and Suzakanagano East interchanges (both away from and toward Tokyo), the Nagano expressway between the Toyoshina and Omi interchanges (away from Tokyo), and the Nagano expressway between the Omi and Kousyoku interchanges (away from Tokyo) (Takahashi et al. 2013). Table 1 provides a summary of the sites from which the samples were taken.

Both the damaged and undamaged portions indicated in the table were distinguished by the cracking present on the surface course. The traffic volume on the expressways is approximately 14,000–15,000 vehicles/day in each direction, and large vehicles account for 19%–22% of this total. The section of the Joshin-Etsu Expressway between the Suzakanagano East interchange and the Nagano interchange was opened in November 1995, while the sections on the Nagano expressway were opened in March 1993. Subsequently, repairs were made on the Nagano expressway to the surface, binder, or asphalt stabilized base course of these roads. Dense graded asphalt mixtures (DGA) or porous asphalt mixtures (PA) were used, as shown in the table 1.

Table 1. Summary of the investigation sites

Site	#121		#72				#39		Obasute	
Location	Nagano IC – Suzakanagano east IC		Suzakanagano east IC – Nagano IC				Toyoshina IC – Omi IC		Omi IC – Kousyoku IC	
Type of surface	Dense-graded		Dense-graded				Porous	Dense-graded	Porous	Porous
Open	March 1995		March 1995				March 1993		March 1993	
Repair	-		-				2005 ^{a)}	1998 ^{a)}	2005 ^{b)}	
No.	1	2	3	4	5	6	7	8	9	10
Location	Non-wheel path	Wheel path	Wheel path	Wheel path	Wheel path	Non-wheel path	Wheel path	Wheel path	Wheel path	Wheel path
Damage Level ^{c)}	A	A	A	D	C	C	D	D	A	D

a) Surface course and binder course were repaired.

b) Surface course, binder course, and a portion of asphalt stabilized base course were repaired.

c) A: non-damaged, C: slight cracking, D: much cracking on the surface of the pavement.

3.2 DESCRIPTION OF SPECIMENS

Block samples (45 cm × 45 cm × 18 cm) of asphalt mixtures were retrieved from the expressways. The asphalt mixtures were then sliced into sections with the dimensions shown in Figure 1(a) representing the surface, binder, and asphalt stabilized base courses. Asphalt samples were then extracted from each section.

The specimens taken from undamaged expressway sections were well formed, but the specimens retrieved from damaged pavement tended to fragment during the cutting process. Additionally, some specimens showed cracking in the asphalt stabilized base course layer, as shown in Figure 1(b).

According to the construction records, straight-run asphalt (stAs) was used for the asphalt mixtures. However, the asphalt used in the repaired sections was not recorded. Tests for the presence of a modifier were conducted in compliance with “Quantitative Test Method A058 for SBR Incorporated in Polymer-Modified Asphalts” published in the Book of Standards for Surveys and Tests of Pavement. Absorption at 965 cm⁻¹, which is typical for styrene-butadiene-styrene (SBS) block copolymer, was observed in all of the PA samples. Additionally, the softening temperature was around 80°C, which provides further evidence that high content polymer-modified asphalt (Type H) had been used.

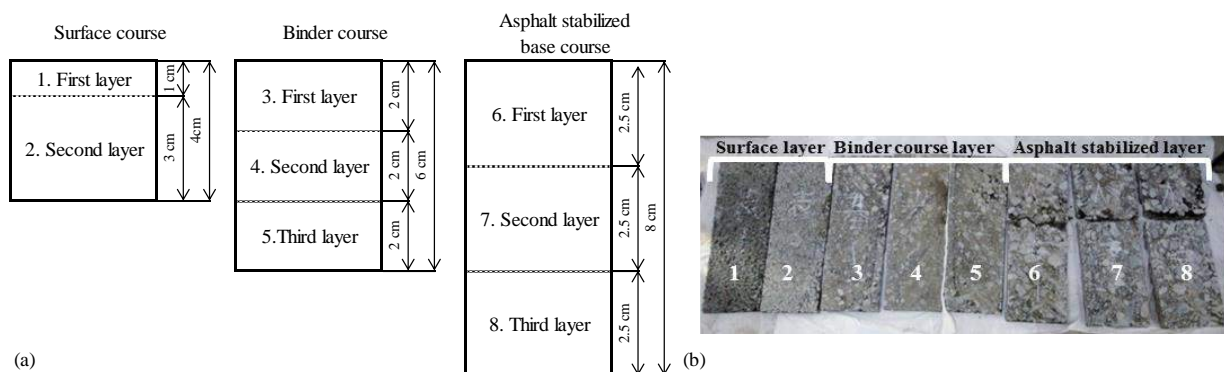


Figure 1. (a) Sliced layers of the samples (b) Cracking in the asphalt stabilized base course layer

3.3 EVALUATION METHOD

Asphalt was extracted from the blocks using Abson method (ASTM 1856-95a) in Table 2. The properties test summarized in Table 3 were conducted. Infrared spectroscopic analysis followed “The Handbook of Pavement investigation and examination method (2007)”. Infrared spectroscopic analysis was conducted by attenuated total

reflectance Fourier transform infrared (ATR-FTIR) spectroscopy (Shimadzu), which enabled direct observation of the asphalt without the use of solvents. Asphalt aging was previously investigated over the range of wavelengths described in Puello et al. 2008. In this comparative investigation, we focused on the increase in absorbance at the carbonyl peak (1700 cm^{-1}) that is expected with the progression of oxidative aging using the approach outlined in Figure 2.

Table 2. Absorbance test conditions

Items	Contents
Flow value of inactive gas at 135°C	200ml/min
Flow value of inactive gas at 160°C	1600ml/min
Hold time at 160°C	30min
Extractant	n- propyl bromide

Table 3. Characterizations performed in this study

Test	Temperature	Method
Penetration	25°C	ASTM D5
Softening point	-	ASTM D36
Ductility	15°C	ASTM D113
Infrared spectroscopic analysis	-	A058

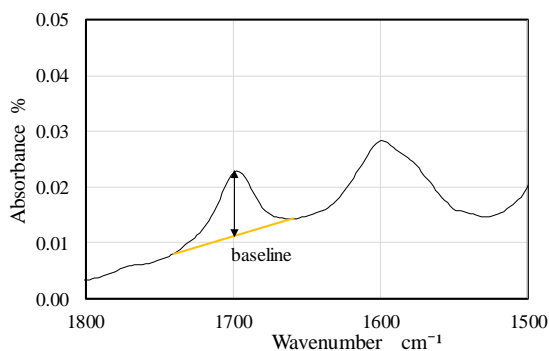


Figure 2. Method for calculating absorbance using carbonyl absorption at 1700 cm^{-1}

3.4 TEST RESULTS

3.4.1 PENETRATION OF ASPHALT IN THE DEPTH DIRECTION

Figures 3(a)–3(d) show the results of penetration tests from the surface to the asphalt stabilized base course taken from each investigation site. Since there were no clear distinctions from the first to third layers for any of the binder course and asphalt stabilized base course samples, the averages of the first, second, and third layers are shown. The undamaged and damaged portions are indicated by solid and dashed lines, respectively, while the DGA and PA are indicated with solid and open symbols, respectively.

For site #121, the first layer penetration in the surface course was the lowest value for all layers, and the level of penetration was higher as the distance from the surface of the block increased.

For site #72, cracking was found to have occurred at the bottom surface of all blocks. Low-level penetration was found in the asphalt stabilized layer, as well as the surface course. The level of asphalt penetration in the damaged section (No. 4 in the figure) was especially notable, although low-level penetration was found in all layers.

The surface course from investigation site #39 was classified into 7-year-old PA and 14-year-old DGA. Both sets were taken from the damaged portions of wheel paths. Penetration into the 7-year-old PA (No. 7 in the figure) was clearly lower than that into the 14-year-old DGA (No. 8 in the figure).

The surface course in the Obasute site was replaced with PA during repair work in 2005. Specimens were taken from both undamaged and damaged portions. While no cracking was found in the upper surface, both the undamaged and damaged specimens showed cracking at the bottom surface. Penetration was lower at the upper surface, and increased with depth, as seen at the site #121.

Focusing on the damage level, it was found that when the asphalt pavement was undamaged, asphalt penetration was lower at the surface level, and increased with depth from the block surface. The DGA and PA results were similar. When the damage was seen in the asphalt pavement, no difference in penetration was seen between the surface course and the asphalt stabilized base course. It is presumed that the damage occurring in asphalt pavement caused the oxidative aging progression.

Regarding differences between wheel path and non-wheel paths locations, non-wheel path penetration was found to be lower than the levels found in wheel paths, as described in previous papers (Kasahara et al. 1975; Tateishi et al. 1998).

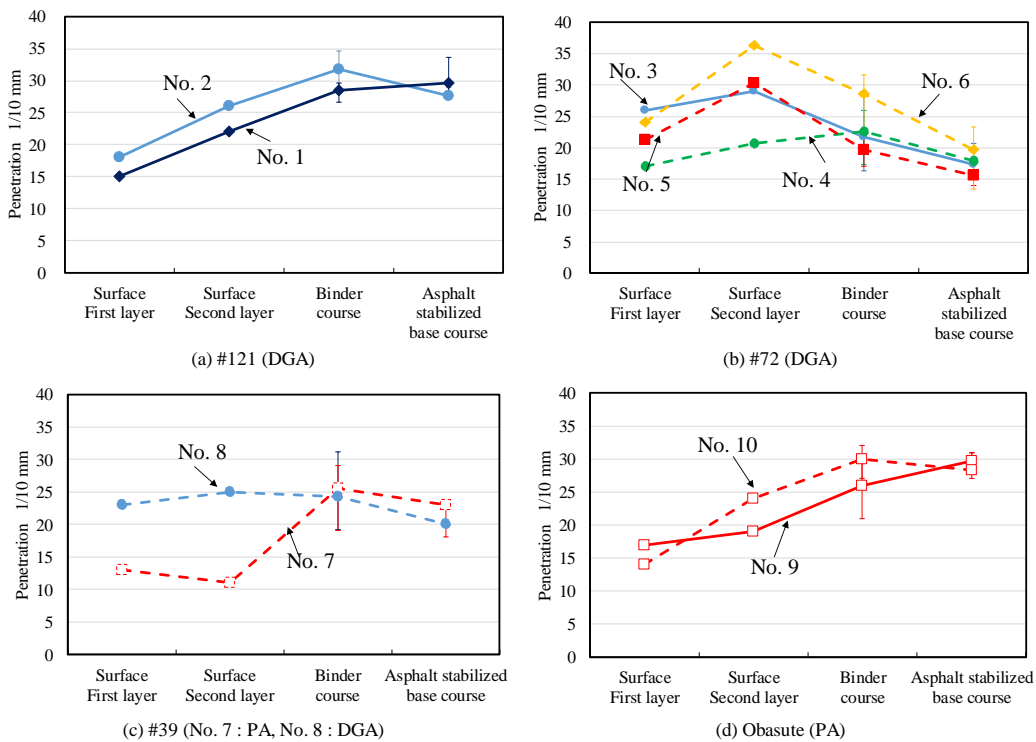


Figure 3. Penetration of asphalt extracted from surface to asphalt stabilized base course in each investigation site

3.4.2 VARIATIONS IN PENETRATION WITH LENGTH OF SERVICE

Figure 4(a) provides the results of the tests and shows comparisons between years of service and penetration of the surface course. Penetration of asphalt extracted from PA with 7 years of service was 11–24 (1/10 mm), and that of DGA with 19 years of service was 13–29 (1/10 mm). Although there was a more than 10-year difference in the service periods, these specimens showed similar penetration levels. The asphalt including PA tends to deteriorate more rapidly in comparison with that of DGA.

Figure 4(b) shows the relationship between length of service and penetration in the binder course and asphalt stabilized base course. As expected, penetration into the undamaged pavements was not significantly lower, although it was clearly lower in those of the damaged pavements. This indicates that the progression of asphalt aging is affected by the presence of damage to the asphalt mixture.

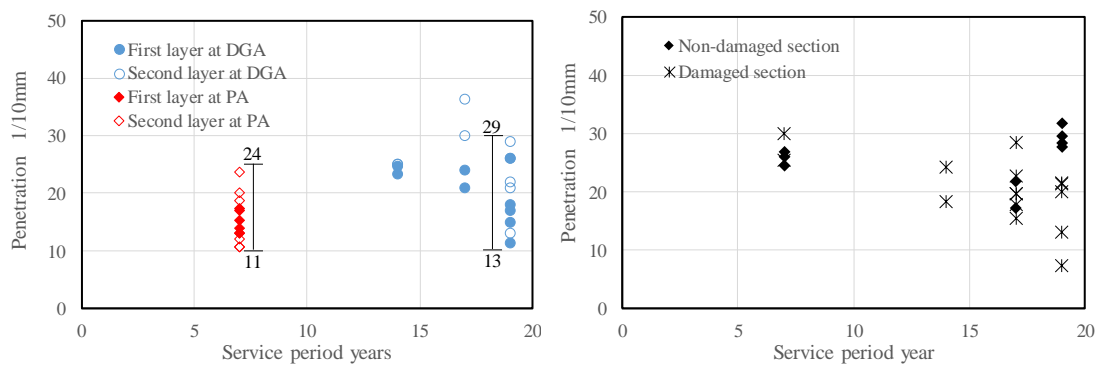


Figure 4. Relationship between Service period and penetration (a) surface course and (b) binder course and asphalt stabilized base course.

3.4.3 RELATIONSHIP BETWEEN PENETRATION AND OTHER PROPERTIES

Figure 5 shows information on absorbance by infrared spectroscopic analysis, ductility, softening point, and penetration. First, the infrared spectroscopic analysis clearly shows that the absorbance increased as the penetration decreased, which indicates that the asphalt hardened as oxidative aging progressed. The absorbance of TypeH extracted from PA was lower than that of stAs for the same penetration values. It was also found that the service period for the PA was shorter than that for the stAs, which suggests that the service period has an impact on absorbance.

The softening point increased and ductility decreased as the penetration decreased. The ductility of TypeH was found to be somewhat higher than that of stAs with identical penetration, which suggests that the modifiers included in TypeH maintain the ductile properties of asphalt.

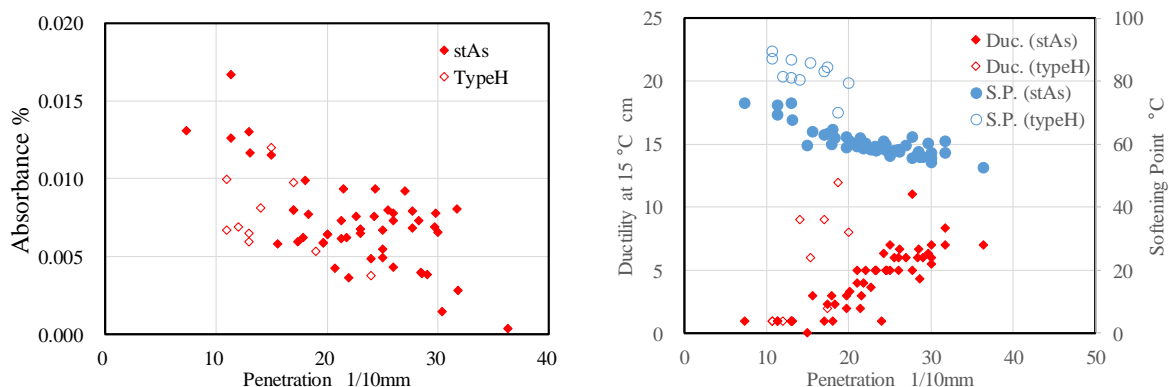


Figure 5. Variation in penetration with (a) absorbance and (b) ductility and softening point.

4 EVALUATION OF RHEOLOGICAL PROPERTIES OF ASPHALT

4.1 ACCELERATED AGING TESTS OF ASPHALT

Next, to determine the viscoelastic properties of field-aged asphalt, test specimens were examined over a temperature range of room temperature to the high-temperature region. The influence of aging was observed in laboratory tests, specimens subjected to accelerated aging were compared with asphalt taken from expressways, and methods for evaluating the viscoelastic properties of aged asphalt were investigated.

An accelerated aging test was conducted using stAs. After a rolling thin film oven test (RTFOT: ASTM D 2872), specimens were aged for 20, 40, and 60 hours with a pressurized aging vessel (PAV: ASTM D 6521).

4.2 METHODS OF EVALUATION

The penetration test and the softening point test were used to estimate the stiffness of the specimens taken from pavement, and the DSR test was used to measure each modulus of the asphalts. Table 4 shows the DSR test conditions.

Van der Poel proposed a nomograph for estimating asphalt stiffness using penetration, a softening point penetration index (PI), and any available loading time and temperature information (Van der Poel 1954). Himeno presented software he had developed in “Estimation of S_{bit} and S_{mix} using Van der Poel's nomograph and Heukelom's equation ($S_{bit}S_{mix}$)”, based on Van der Poel's nomograph, in which asphalt stiffness was calculated upon input of each property. That software was used to calculate the asphalt stiffness in this study.

Table 4. DSR test conditions

Items	Contents
Test Temperature	20°C, 40°C, 60°C
Frequency	0.1-100rad/s
Radius of plate	8mm
Thickness	1mm
Strain	10%

4.3 RESULTS OF PROPERTY TESTS

4.3.1 Results of accelerated aging test

Table 5 presents the test results. It can be seen that accelerated aging reduced the penetration and raised the softening point, and that the PI of stAs was increased by these changes. Additionally, while ductility did not change significantly after the RTFOT, it was found to fall quickly after the PAV test.

Figure 6 shows how the value for S_{bit} of stAs, calculated by the program using penetration, softening point, and PI, varied with several parameters identified in the DSR test. A significantly high correlation was found between S_{bit} and the complex modulus indicated by the DSR test, and the relationship is linear on a log-log graph (Figure 6(a)). Examination of the relationships among each modulus shows that these all fell with an increase in the measured temperature or a lengthening of load time. Comparing the complex modulus, loss modulus, and storage modulus, we see that a storage modulus decreased most (Figure 6(b)).

Table 5. Physical properties of stAs after accelerated aging

Test			Original	RTFOT	PAV20h	PAV40h	PAV60h
Penetration	25°C	1/10mm	61	42	30	25	18
Softening Point		°C	48.0	51	57	60	62.5
Ductility	15°C	cm	100+	100+	7	5	4
PI		-	-1.3	-1.3	-0.7	-0.5	-0.6

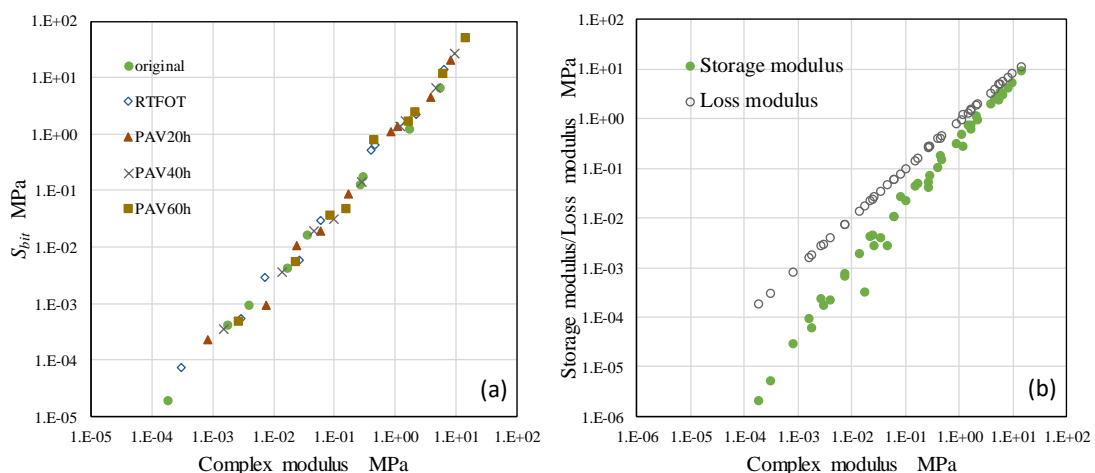


Figure 6. S_{bit} , complex modulus, storage modulus, loss modulus and phase angle of straight asphalt after accelerated aging

4.3.2 Comparison between field-aged asphalt and accelerated-aged asphalt

Figure 7 shows the values for the stiffness of the extracted asphalts predicted by the nomograph and each modulus found by the DSR test. Here, the parameters were abstracted from the DSR results under loading times of 0.1,

1, and 10 s, and the results were compared with the stiffness results provided by the nomograph. The parameters in the figure representing asphalt extracted from visibly damaged specimens were measured at 20°C after loading for 0.1 s.

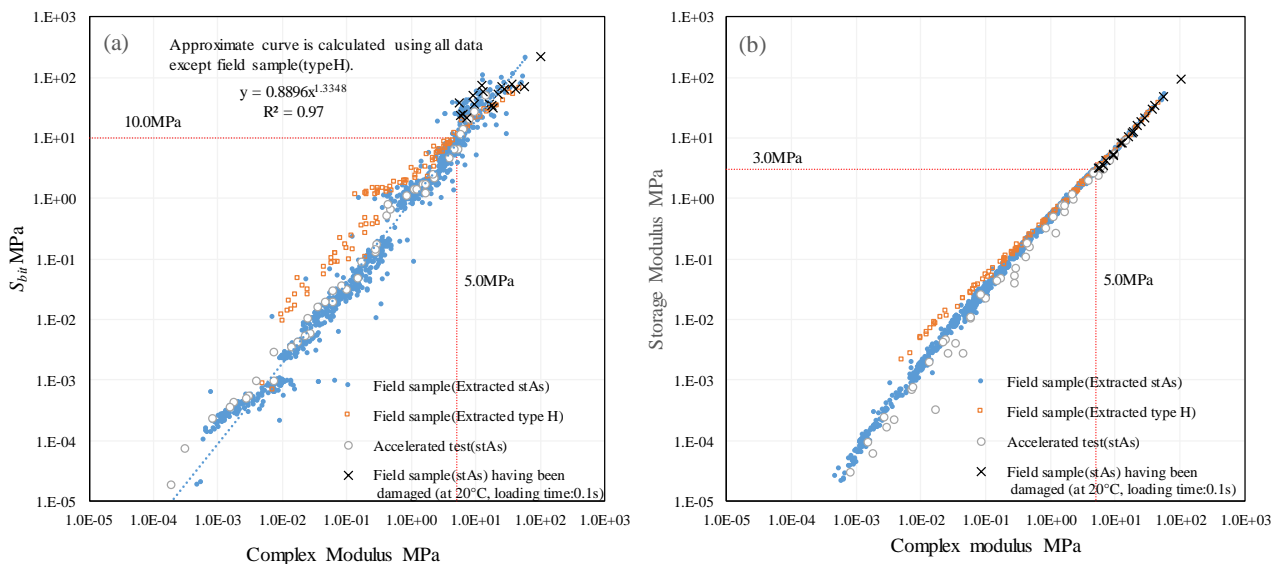
By the results except type H, the complex modulus indicated by the DSR test showed excellent correlation with the stiffness of the extracted asphalt estimated using the nomograph, regardless of use age, load time, or test temperature. From these results, for stAs, the stiffness can be estimated using the nomograph with penetration and softening point. We can see that this represents the viscoelastic properties of asphalt with high accuracy.

These specimens had been in use for 20 years and comprised different asphalts, but they provided results that were nearly identical to the stAs specimens used in the laboratory accelerated aging test. These findings suggest that asphalts subjected to accelerated aging in the laboratory can provide some estimates of the stiffness and changes in stiffness occurring after aging in the field. However, the stiffness found the nomograph did not match the results for complex modulus measured by the DSR test on a one-to-one basis. This discrepancy is probably for two reasons: the stiffness predicted by the nomograph is based on the penetration and the softening point, that is, observations in static tests, while the DSR test is a dynamic test; and the nomograph estimate is based on two temperature measurements, while the DSR test is conducted at numerous different temperatures.

Polymer modified asphalts have high complex modulus at high temperatures, so they exhibit behaviors different from those of straight asphalts. In the future, accelerated aging tests must be conducted with polymer modified asphalts in order to obtain a better understanding of the behaviors of these mixes as well.

Examination of the relationship between the storage modulus and the complex modulus revealed slight differences in the specimens subjected to accelerated aging in the laboratory and the specimens extracted from roadways. In contrast, the same comparison between the loss modulus and the complex modulus revealed a linear relationship between the two on a log-log chart, regardless of changes in test temperature, loading time, or material, that is, accelerated-aged asphalt versus road-aged asphalt.

The stiffness, complex modulus, storage modulus, and loss modulus of damaged asphalt were examined at the test temperature of 20°C and a loading time of 0.1 s. The stiffness was at least 10 MPa, the complex modulus and loss modulus were at least 5 MPa, and the storage modulus was at least 3 MPa. It is especially notable (see Figure 7(c)) that the relationship between complex modulus and loss modulus can be viewed as a straight line in the materials subjected to accelerated aging in the laboratory, as well as in the asphalt and the modified Type H asphalt extracted from roadways, and thus can be treated as a single index for the degree of aging. The Strategic Highway Research Program (SHRP) has set a standard of 5 MPa or less for the asphalt subjected to accelerated aging of loss modulus $|G^*| \cdot \sin \delta$; the test conditions in the SHRP differ from those in the present study, but their results have generally been the same.



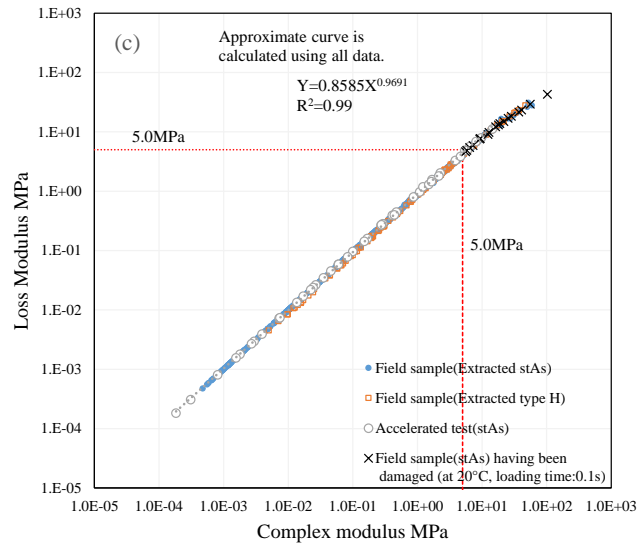


Figure 7. S_{bit} , complex modulus, storage modulus and loss modulus of field-aged asphalt compared to asphalt after accelerated aging

5 SUMMARY

The following findings were obtained in this study:

- Aging decreases the rheological properties in the interior courses of asphalt, as well as the surface course. The degree of aging depends on the type of the asphalt mixture and the extent of damage.
- A study of the relationships between penetration, ductility, and softening point indicated that the ductility decreases and the softening point increases as penetration decreases. Additionally, Type H used for PA has higher ductility than stAs, even though its penetration is lower.
- Type H used for PA with 7 years of service and DGA with 19 years of service showed the same level of penetration, even though penetration between stAs and Type H were different at the original construction.
- In order to simulate the properties of actual aged asphalt pavement, it is necessary to consider the composition of the asphalt mixture.
- The stiffness of asphalt subjected to accelerated aging in the laboratory and that of field-aged asphalt show similar trends, in spite of loading time and test temperature differences. This suggests that the stiffness of field-aged asphalt can be reproduced by accelerated aging.
- There is an extremely high correlation between the complex modulus obtained in the DSR test and the asphalt stiffness estimated using the nomograph.
- The complex modulus obtained in the DSR test was found to show a linear relationship with the loss modulus on a log-log graph, regardless of test temperature, load time, asphalt type, and source of specimens, whether laboratory specimens subjected to accelerated aging or specimens from roadways.
- The stiffness of damaged asphalt was at least 10 MPa, the complex modulus and loss modulus were at least 5 MPa, and the storage modulus was at least 3 MPa. Since the complex modulus showed a relationship with loss modulus that could be considered linear, regardless of whether the specimens had been subjected to accelerated aging in the laboratory or recovered from roadways, whether stAs or Type H asphalt, this relationship can be expressed as a single index of the degree of aging.

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PAPER TITLE	<i>New Integrated System to support the London Streets Tunnels Operations Control Centre (LSTOC)</i>		
TRACK	Intelligent Transportation Systems		
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KEYWORDS:

“**HORUS**” is a SCADA (Supervisory Control And Data Acquisition) product developed by Indra that not only integrates different devices and systems into a complete software application to monitor and control interurban traffic and tunnels, but also manages the equipment maintenance.

“**London Streets Tunnels Operations Centre (LSTOC)**” means the London Streets Tunnels Operation Centre which is responsible for the management of traffic and safety of the London road tunnels and designated approach roads

“**Minimum Operating Requirements (MOR)**” means, in respect of each Tunnel, the lowest number of End-Point Devices connected to the System which the Users need to be able to view and/or control in order to keep a Tunnel open

“**Rule**” is a set of fundamental rules for the control of End-Point Devices which cannot be contravened by either a User or the System, primarily to ensure the safety of road users

“**Sequence**” is a series of End-Point Device actions which must be implemented in a defined order with appropriate time delays and confirmation points;

ABSTRACT:

In 2015, HORUS was the new integrated system to support the London Streets Tunnels Operations Control Centre (LSTOC).

The HORUS platform is **the solution to modernize the technology for the control and operation** of the 12 road tunnels in London and 90 km of approach roads. These tunnels are critical for the mobility across London, some of which are more than 100 years old and others which have special requirements (A2 Eltham, A12 Eastway, George Green and GreenMan, A13 Limehouse Link and East India Dock, A101 Rotherhithe, A102 Blackwall Northbound and Southbound, Hanger Lane, Upper Thames Street and A406 Fore Street Tunnels).

The **HORUS** solution provides the LSTOC operator with real-time information, helping in decision-making and facilitating quick and accurate management of everything that happens in the tunnels. Due to the **intelligent incident management system** operation is simple and predictable, even in exceptional situations, as the system guides the operator through the correct response in each case.

By collecting current and historical traffic data, the Horus solution also facilitates the **analysis, reporting** and consolidation of information for decision making regarding mobility.

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Therefore, the new system:

- provides a unique solution
- integrates multiple traffic control systems
- manages emergencies to keep the city moving safely and efficiently
- helps to manage maintenance services
- provides the real-time information about incidents in the tunnels and minimise the impact on London's streets
- is designed to incorporate new systems in the future

The aim object of this paper is to provide you an introduction for the solution and the operational management.

Firstly, It includes specific sections to introduce you in the project:

- A leader in technology and innovation (Who is Indra?, What does it do?, Our experience)
 - Our client (What is Transport for London?, What does it do?, What TfL is requesting?)
- The transport and traffic solution (HORUS. Transport and Traffic solution, How did Indra do it?)

Secondly, It provides specific sections to introduction you in the general solution:

- The system architecture and integrations
- Main features of the platform (all in one philosophy, flexibility, scalability, customization)
- The advanced incident management system (IMS) and asset management
- Expert systems (Rules , sequences and MOR)
- Configuration tools
- Reporting, validation and simulation tools

And finally, the remaining sections introduces you the conclusions and acknowledgements.

In summary, Indra provides a single and intelligent traffic system with state-of-the-art technology to manage the city's tunnels in an integrated manner.

New Integrated System to support the London Streets Tunnels Operations Control Centre (LSTOC)

Olga ALVAREZ MOLINA

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1 INTRODUCTION

Indra is delighted to have been accepted to this congress and we are confident that we are able to contribute our knowledge and years of technical expertise to all of the participants.

Throughout this paper we will have the chance to share our experiences. In particular, we will present the reader with the technical solution for the London Streets Tunnels Operation Centre (LSTOC) project.

In the following chapters, the reader will have the opportunity to enjoy this approach to the project. Indra hopes that the reader finds this paper useful.

2 INDRA, A LEADER IN TECHNOLOGY AND INNOVATION

Who is Indra?

Indra is a global consulting, technology, innovation and talent company headquartered in Madrid. It is on the cutting edge of high value-added solutions and services for the Transport and Traffic, Energy and Industry, Public Administration and Healthcare, Financial Services, Security and Defence and Telecom and Media sectors. The multinational is one of the top European companies in its sector in terms of Research & Development & Innovation.

Additional company information about Indra can be found by following the links below:

<http://www.indracompany.com>

<http://www.indracompany.com/en/tunnels>

What does it do?

The company **operates in 128 countries, with more than 42,000 employees worldwide** focusing on developing innovative solutions that meet the needs of the most demanding clients.

Our experience

The Transport and Traffic sector accounts for 23% of the company's overall business and is the largest, and fastest growing, area within Indra. Our priority is to design more secure, ecological, economical and efficient technologies and we supply solutions and services for Air Traffic Management (ATM); Communications, Navigation and Surveillance (CNS); Airports; Rail Traffic Management; Intelligent Traffic Systems (ITS); Vessel Traffic Management (VTMS); CAD/AVL Systems; Ticketing and Tolling.

We are proud to boast that more than 100 cities in the world rely on Indra for the management, security and development of their transport networks and that we have supplied 1,200 air traffic management facilities in more than 90 countries.

HORUS. Transport and Traffic solution

Specifically in the Road Tunnel Management field, we are a world leader and our Horus platform has been used in more than 50 large traffic infrastructure projects around the world over the last 10 years, about half of these being complex multi-tunnel environments.

More than 30 countries control the traffic on their highways, tunnels, airports and roads with Indra's systems.

3 BACKGROUND INFORMATION

What is Transport for London?

Transport for London (TfL) is the integrated body responsible for London's transport system.

What does it do?

TfL is responsible for London's network of principal road routes, for various rail networks including the London Underground, London Overground, Docklands Light Railway and TfL Rail, for London's trams, buses and taxis, for cycling provision, and for river services. It is accountable for both the planning and management of transport services across London.

What is London Streets Tunnels Operations Centre (LSTOC)?

LSTOC is responsible for the management of traffic and safety of the London road tunnels and designated approach roads.

The key LSTOC responsibilities are:

- Monitor and control tunnel environments and systems
- Control Traffic Using the Tunnels and Approaches
- Co-ordinate "First Response" Incident Management
- Co-ordinate Internal Communication, Incident Logging and Report System Management

4 GENERAL PURPOSE OF THE LSTOC PROJECT

HORUS is the new integrated system to support the London Streets Tunnels Operations Control Centre (LSTOC).

What was TfL requesting?

Before the migration, LSTOC used multiple control systems to operate and control safety and traffic control systems, communicate with road users, emergency and maintenance services people and to manage safety and minimize disruption.

According to this, the new system should be able to provide:

- A fully integrated solution
- A multi-system interface
- A multi-operator platform
- A solution that monitors and controls all the existing sub-systems and end-point devices in the 12 urban tunnels
- A solution that monitors and controls all the existing sub-systems and end-point devices in the LTRACS traffic systems on 90km of Road Networks.
- Primary and disaster recovery operator user interfaces in two separate central London facilities.
- Hosting in two high quality data centers.



Figure 1. HORUS platform

What does Indra provide?

The HORUS platform is able to carry out the following functions:

- Tunnels' monitoring and control
- Asset Management and Fault Handling
- Incident and Alarm management

- Performance Reporting
- Decision Support
- Integrated Human Machine Interface (HMI)

Therefore, the upgrade solution for the LSTOC is based on the **HORUS platform**. This new management system has the following technical capabilities:

- A new software platform with a single interface that communicates with, monitors and controls all the existing systems in the 12 tunnels and 90km of surface roads operated by TfL from the London Streets Tunnels Operations Control Centre (LSTOC).
- An open, highly flexible and configurable solution, which can incorporate any new system.
- A high level of automation of operations, which provides the operator with real-time information, helping in decision-making and facilitating quick and accurate management of everything that happens in the tunnels.
- An intelligent incident management system whose operation is simple and predictable, even in exceptional situations, as the system guides the operator through the correct response in each case.
- By collecting current and historical traffic data, the Horus solution also facilitates the analysis, reporting and consolidation of information for decision-making in terms of mobility.

How does Indra do it?

The HORUS platform is able to monitor and control the following sub-systems:

- Public Address and Emergency Telephones
- Road traffic signs, signals and barriers
- Over-height vehicle detection
- Tunnel closure system
- Tunnel emergency signage system
- Integration with other SCADA systems (floodgate, electrical supplies, etc)
- CCTV system (including video wall)
- Video Automatic Incident Detection (VAID) system
- Drainage system
- Pump system
- Fire detection
- Tunnel ventilation system
- Tunnel lighting system
- Fire detections and suppression system

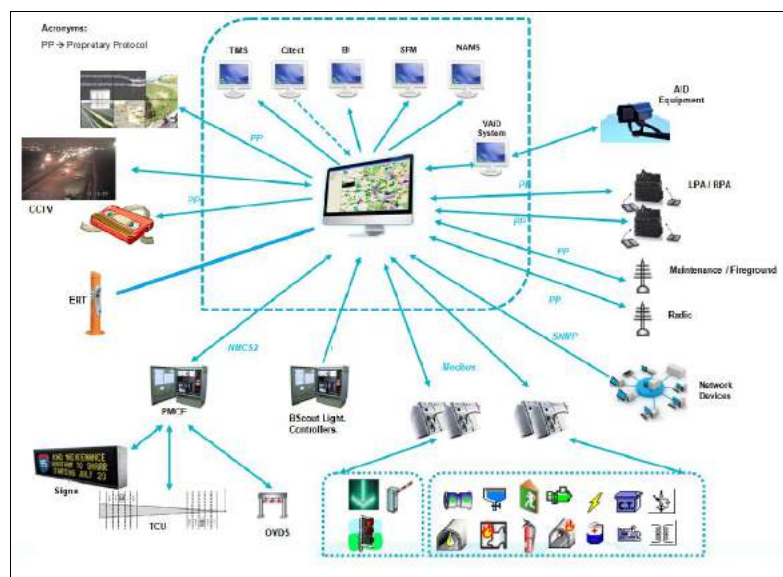


Figure 2. System & Integration Protocols

The HORUS platform is able to exchange data with external systems like

- TMS means "Traffic Information Management Systems"
- NAMS means "Asset Management System for Highway, Drainage and Electrical Assets on the Transport for London Road Network"
- SFM means "Site Fault Management"

5 TECHNICAL SOLUTIONS & INTEGRATIONS

Flexibility & scalability

The HORUS system has been designed as a **modular, distributed application**, its tasks executed in distributed systems allowing each module to be expanded or reconfigured without affecting the others.

As a result of this modularity, the large library of standard interfaces and the integration tools available, the system is highly expandable. This has allowed many of our customers to seamlessly grow the system adding new tunnels to their network or new features to the system.

Systems Architecture

The HORUS application uses a distributed process architecture that allows the system to scale in performance and functionality. The process communications are service oriented and performed through the Active Interface Communications library JMS.

HORUS Platform: “all in one” philosophy

The HORUS Platform has been designed with an “All-in-one” philosophy, aiming to simplify the work to be performed by the operator, both in their daily routine and in emergency situations. To achieve this goal the system has been designed to:

- Be consistent and easy to use in terms of system operation.
- Use summary windows and present the relevant information, highlighting where the operator’s attention is required.
- Unify tools (Alarms, Sequences, summary windows...) of different types of elements, to avoid having to navigate between multiple windows and simplifying complex tasks.

The HORUS platform is designed for the consistent representation and management of traffic and tunnels’ elements, regardless of the type of equipment. The aim is that ,once the operator knows how to manage one type of equipment, they will know the basics of how to manage all system equipment. This consistent representation of any item ensures the operator will easily see, with just a glance, the status of any item and without the need to consult multiple detailed information screens. The navigation has been optimised to minimise the number of steps required to perform the basic actions of daily work. It is also a highly integrated platform allowing a single user interface to manage all the systems under the operators’ control.

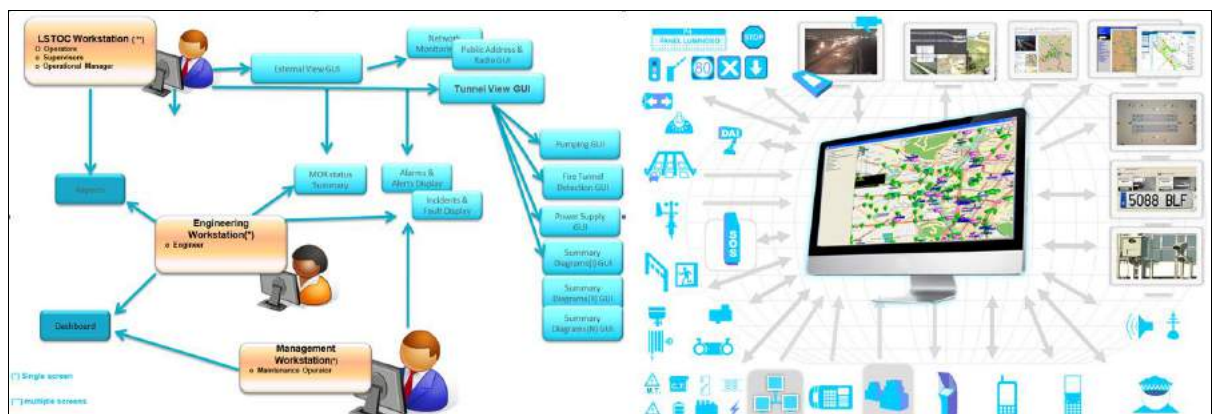


Figure 3. HORUS Platform: “all in one” philosophy

Customization & Configuration tools

The HORUS system is fully customizable. The HORUS system allows configuration of parameter settings related with alarms, procedures, incidents, resources, user profiles, road maps, etc.

The administrator could modify existing equipment, add new equipment, new equipment types and even new tunnels into the platform.

6 THE ADVANCED INCIDENT MANAGEMENT SYSTEM (IMS) AND ASSET MANAGEMENT

Advanced incident management system (IMS)

One of the major strengths of HORUS is its powerful and flexible Incident Management System (HORUS IMS), which coordinates all subsystems and performs an integral management of the incident and incident recording. HORUS goes beyond the required functionality through a high level subsystem that automates the detection of operational incidents and maintenance incidents, and automates the response to these, guiding the users through those actions that the system cannot do automatically.

This HORUS IMS subsystem carries out complex analysis and management of concurrent situations handling multiple alarms intelligently. It manages the response to an incident based on the severity of the incident, its impact on the network impact and if there are other active incidents nearby.

HORUS IMS helps LSTOC maximize security and minimize risks; reduce reaction time to a road incident; guarantee a consistent response time and ensure that the incident management follows the operation manual.

Reporting tools (IMS)

All the actions, automatic or manual are registered in the system. It is possible to query this information through specific reports.

Workflow and Incident life Cycle

An incident has the following workflow:

- **Detection:** An incident can be automatically generated through an alarm sent by equipment via the Control Centre Software or manually by the user. At this stage, and to facilitate the verification of the incident, the IMS will automatically reposition a camera located near the incident, and show this on the relevant control centre monitor.
- **Confirmation:** When an incident is automatically detected, the user must confirm or discard it. In order to verify the incident, the user can use the live feed image situated in the monitor which the system has previously repositioned along with any other relevant information which will be presented to the operator.
- **Response:** On confirming the Incident, the IMS will provide the operator with a series of optional and mandatory actions. These will be used in order to resolve the Incident as defined in the Operation Manual. These actions can be automatic or manual:
- **Automatic actions:** Many of the actions defined in the Operation Manual can be automatically performed through the IMS, these include actions such as: signalling, lighting, public address, Ventilation, etc. or any other action implemented in the IMS. Some of these may require operator intervention to authorize the execution.
- **Manual actions:** All those reserved for the user because they are not automated.
- **Restoration:** After the resolution of the incident, it is necessary to restore the initial conditions of the road. The IMS is also capable of automating this restoration.
- **Incident Reporting:** The IMS allows information and documentation to be attached to the incidents file. As the Incident is managed the required data is collected and stored in the database.

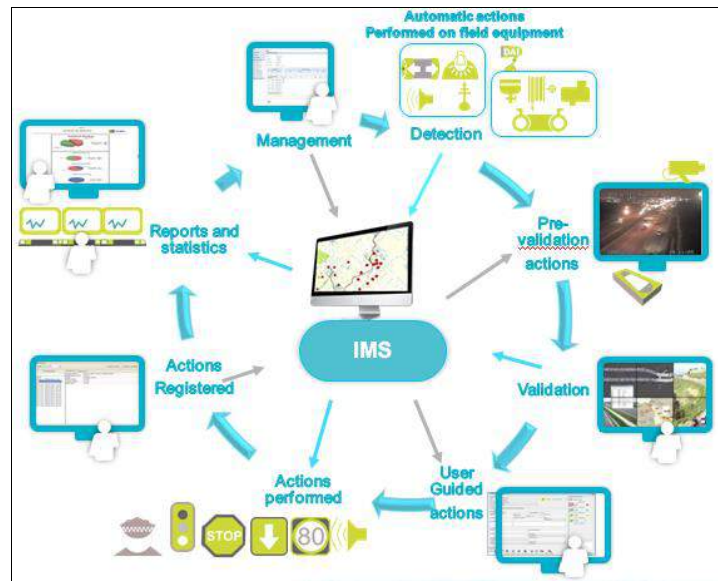


Figure 5. IMS: Management for the whole Incident life Cycle

Benefits of using the IMS

The benefits of using the IMS are many:

- Maximize security and minimize risks
- Reduce reaction time towards road incidents.
- Guarantee that the response time towards one same incident in different periods of time is homogenous and follows the Operation Manual.
- Facilitate the Operation of Control Centre
- Register all operations carried out, both normal operations described in the road management plan, and those carried out to solve an incident as described in the emergency plan.
- Allow the information registered to be used, not only at a statistical level, but also based in actions carried out during the resolution of a specific incident.
- Provide a high level of service. The high level execution of the objectives is essential to provide all the road users with an adequate level of comfort and their subsequent satisfaction.
- Optimize costs - both those derived from the operation itself and those that come from having the road shut down.

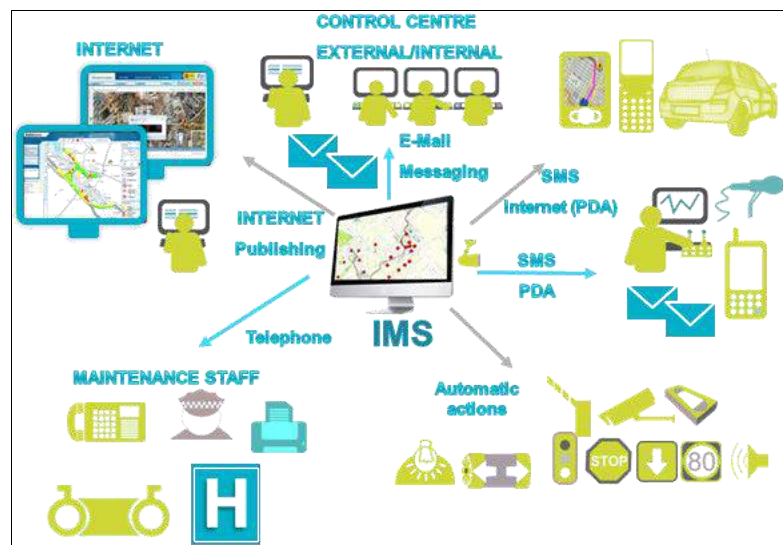


Figure 6. IMS: Automatic management of Stakeholders & end-point-devices

Asset Management

The HORUS platform, being an incident management platform, has implicit facilities which can be used for asset management without the need for additional software for TfL.

Using the power of the HORUS platform, the operators can control and monitor the facilities, and analyse all the information that flows within the system, included information coming from other systems like SFM or NAMS or the MOR tracking.

This module manages the whole life cycle of asset faults (i.e. detection, monitoring of the actions taken and their resolution). These faults are treated as maintenance incidents, taking advantage of the benefits of the incident management features of the HORUS IMS.

As a result, this tool is able to reduce failure detection time improving the response to the faults, with the aim of ensuring the optimum maintenance conditions of both facilities and equipment.

Reporting tools

Horus also has a powerful tool for reporting. New reports can be added or modified to fit client requirements. The system is designed to know what has happened and what action has been taken, as well as information that will assist in applying responsibilities, detection of procedural failures, detection of unsafe points on the road, procedures for improvement or evaluating the suitability and number of resources. The system also allows extensive customization of the forms for reports.

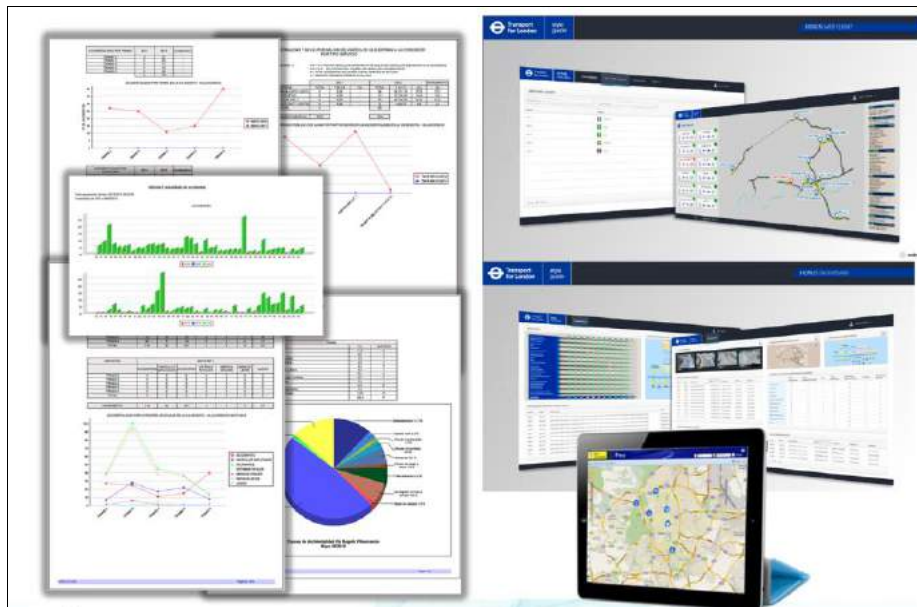


Figure 4. HORUS Platform: Report System & Configuration tools

7 EXPERT SYSTEM

At this point, we would like to highlight the trend of automating scheduled actions using business logic. In this way, the HORUS Platform has a set of tools to increase the reliability of operations carried out on the devices, automating actions and guiding the operation in order to have a safer environment road and tunnel operation.

There are three expert systems available in LSTOC:

- the rules management
- the sequences management
- the MOR management

All these modules are configurable and they allow subsequent reconfigurations of the system in order to guarantee its scalability.

Rules management

The rules management is a set of fundamental rules for the control of End-Point Devices which cannot be contravened by either a User or the System, primarily to ensure the safety of road users.

This tool provides LSTOC with autonomy and LSTOC is therefore capable of automatically managing systems such as:

- ventilation system,
- lighting system,
- energy savings system,
- roadway management,
- detour notifications, etc

In particular, the **traffic rules** are available for evaluation of traffic conditions. For example:

- Logic required and condition for evaluation
- Logic required for developing device management scenarios of the traffic system

After this validation, the traffic rules are able to set signaling in matrix signal gantries or carriageway.

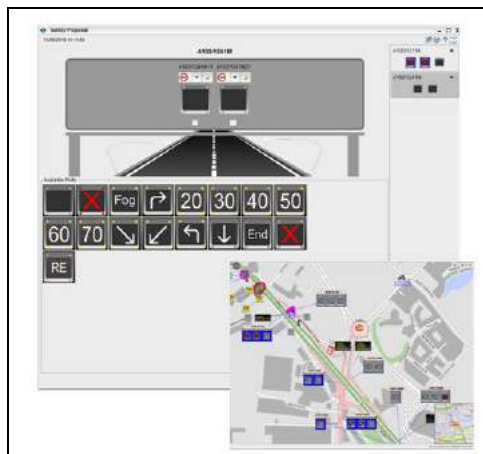


Figure 7. Traffic Rules

In addition, and before the activation of the signaling, there is a proposal layer. This proposal summary is updated with the aspect of the primary signals and the aspects of secondary signals affected.



Figure 8. Off Proposal layer vs. On Proposal layer

Interlock rules

The HORUS system disposes of interlock rules that do not allow the incompatibles states or aspects of the signals. For example, on the reversible lane the HORUS system defines incompatibilities between two signals which have a special location relationship. In order to manage the reversible lane when there are two signals on the same location, one in one direction and the other in the opposite direction, the system blocks the lane open aspect in the two signals at the same time. This incompatible command orders are blocked and it shows a message indicating that this command is not executed because of non-compliance with the interlock rules.

Sequences management

A sequence is a series of End-Point Device actions which must be implemented in a defined order with appropriate time delays and confirmation points. In this context, a sequence is a set of instructions capable of changing the state of part of the traffic system.

In LSTOC, the new system provides TFL with a library of available orders to perform an action over any type of device:

- VMS,
- Traffic Lights,
- Fans, ...

The new system provides TFL with a library of commands or logical types of orders. For example, these are available for use:

- Send warning messages to users.
- Ask confirmation to a user.
- Wait a fixed time.
- Execute another plan, ...

HORUS has a powerful graphic tool that allows the user to define new business logic by creating sequences with automatic, manual or scheduled execution. This execution of complex sequences is transparent to the user.

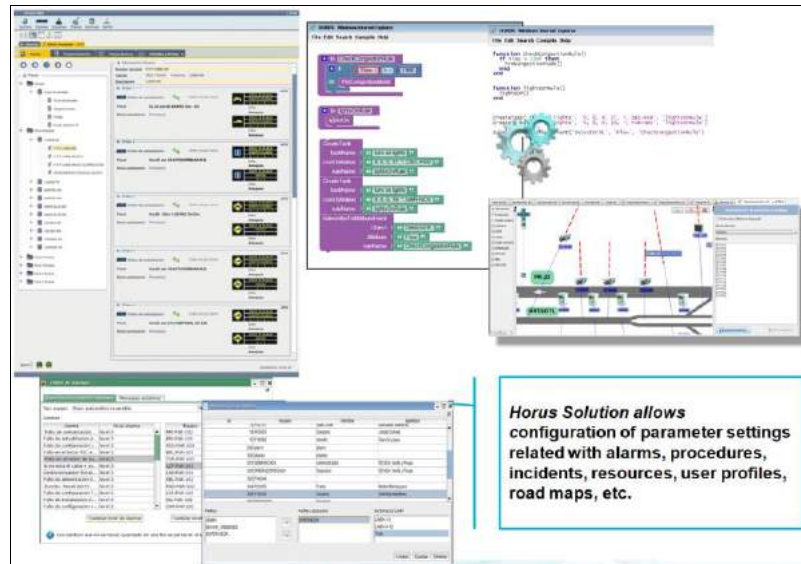


Figure 9. Expert System

MOR Fault management & diagnostics

The Minimum Operating Requirements (MOR) means, in respect of each Tunnel, the lowest number of End-Point Devices connected to the System which the Users need to be able to view and/or control in order to keep a Tunnel open.

The new system, HORUS platform, detects faults of end-point devices, either because the device reports its own failure or by applying detection thresholds or correlation alerts.

The MOR can be loaded into HORUS in order to anticipate a MOR breach and initiate tunnel closure before MOR limits are surpassed.

According to this, the new system HORUS:

- manages the whole life cycle of asset faults
- carries out statistical studies of faults
- it filters false faults and fault avalanches
- has implemented a rating matrix whose criticality conditions are: 25,50,75 or 100%
- has access to all the information:
 - state
 - alarms
 - parameter data
 - events



Figure 10. MOR Fault management & diagnostics

8 SIMULATION TOOLS

FAT Testing & pilot availability

The main objective of the simulation environment in the HORUS platform is to enable the practice of the available tools and to become familiar with the environment and devices installed in tunnels and roads.

In particular, for training / test purposes, the platform has been used for TFL as:

- Platform of training and continuous improvement (Training).
- FAT and Test bench of the control system (Test).

The tool allows simulating the conditions to be controlled and managed in a tunnel. The architecture consists of a model that recreates the installations and on which the operators will be trained.

For the second functionality, the tool allows the performance of offline tests. The following tasks have been performed with TFL:

- Conceptualization Validation: On the environment, it allows the validation of incidents, automatisms, sequences and logic associated with automatic actions.
- Functionality Validation: On the same environment it allows the validation of the state logic, alarm generation, faults and malfunctions.

And finally, the tool is also used for internal TFL validation of new system configurations and version validation after the changes have been made from the preproduction environment during the current operation phase.



Figure 11. FAT Testing & pilot availability

9 CONCLUSIONS

The HORUS platform is the solution to modernize the technology for the control and operation of the 12 road tunnels in London and 90 km of surface roads operated by TfL from the London Streets Tunnels Operations Control Centre (LSTOC).

The updated system has the following functionalities:

- Design, installation, commissioning and maintenance of a new hardware and software platform with a single interface that communicates with, monitors and controls all the existing systems in the 12 tunnels
- As an open, highly flexible and configurable solution, it can incorporate any new system.
- With a high level of automation of operations, the solution provides the operator with real-time information, helping in decision-making and facilitating quick and accurate management of everything that happens in the tunnels. Due to the intelligent incident management system, operation is simple and predictable, even in exceptional situations, as the system guides the operator through the correct response in each case.
- By collecting current and historical traffic data, the Horus solution also facilitates the analysis, reporting and consolidation of information for decision making regarding mobility.

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KEYWORDS:

“Integrated”

“Prasarana”

“Multi-modal”

“Transit”

“Indra”

ABSTRACT:

In 2017-2018 Indra will unveil the new Kuala Lumpur Integrated Control Centre, from where all operation, control and management of the Monorail and two Light Rapid Transit lines will take place. These three transport lines are currently managed from three different control centres; the new Integrated Control Centre will allow their operation from one location and will involve the integration of various different systems, including but not limited to Traffic Management Systems, SCADA for Energy Remote Control, SCADA for Fixed Facilities, Communications, CCTV and Access Control Systems. Furthermore, the new Integrated Control Centre will have the capacity and capability of integrating the operation of Bus Rapid Transit, High-Speed trains and Mass Rapid Transit systems in the future.

Indra previously achieved the successful integration of Prasarana's ticketing system; through the Automatic Fare Collection (AFC) all rail networks now use the MyRapid Card as a common ticketing mode.

KL Integrated Control Centre - Monorail, LRT, BRT, High Speed and Mass Rapid Transit systems

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1. INTRODUCTION

In 2017-2018 Indra will unveil the new Kuala Lumpur Integrated Control Centre (ICC), from where all operation, control and management of the Monorail and two Light Rapid Transit (LRT) lines will take place. This paper will firstly introduce the background and context of the ICC, including a brief description of Prasarana, the owner of the ICC, and Indra, the contractor for this project. The main features and benefits of the ICC will then be outlined and explained. Finally, this paper will conclude with a summary of what has been presented and with the relevancy of the ICC to the Transportation industry in Asia.

2. BACKGROUND

The ICC was commissioned by Prasarana (Prasarana Malaysia Berhad), a 100% government-owned company. Established in 1998 and operational since 2002, Prasarana is the asset owner and operator of Malaysia's two LRT networks and the KL Monorail, in addition to the bus services of RapidKL, RapidPenang and RapidKuantan. Prasarana has been undertaking a modernisation and expansion of its multi-modal transport offering in the past years, with for example the fleet expansion of the KL Monorail and RapidBus, and the line extensions of the Light Rapid Transit lines of KL.

Indra was awarded the ICC project in 2012, after Prasarana saw the need for a control centre from which the operations of its 3 metro lines in Kuala Lumpur could be managed. This innovative and ambitious project consolidates Indra as one of Prasarana's technological partners. Indra has previously successfully integrated Prasarana's ticketing system, so that through the Automatic Fare Collection, all rail networks now use the MyRapid Card as a common ticketing mode.

Indra is a multinational leader in consulting and technology, present in 138 countries and with a strong presence in Asia Pacific. It is now present in India, Bangladesh, China, Vietnam, the Philippines, Indonesia, Thailand, Australia and of course, in Malaysia. Indra offers technological solutions and services in various different sectors, from Transport & Traffic to Security & Defence, passing through Energy & Industry and Public Administration.

Within the Transport & Traffic division, the offering varies from urban traffic systems, airport management, traffic control systems for highways, tunnels and toll systems and port and waterways management. Indra has been an active player in the global railroad traffic industry for many years now, with projects in Saudi Arabia (Mecca-Medina High-Speed Train), Spain, Morocco, Lithuania, China and Brazil for example, and subway systems projects in Madrid, Mumbai, Delhi, Shanghai, Saint Louis and Cairo, to name but a few. The company has one of the most advanced railroad traffic control systems in the world, and has developed an integration platform which allows integrating systems from many different providers into one single control centre. For example, in Lithuania Indra has integrated interlockings from Siemens, Bombardier, AZD, Ex-Soviet technology and electromechanical in one Traffic Management System.

In short, Indra provides integrated solutions for software, systems, control rooms, data centres, crisis management and training rooms in rail environment.

The main aim of the ICC is to integrate the operations of the Ampang and Kelana Jaya lines, as well as the Monorail Line of Kuala Lumpur. The operating control centres of these 3 lines will remain as back-ups to the ICC. Indra will also be responsible for the implementation of a new data centre in the ICC which will support the control centre's activities and provide all the technology required for this.

However, it will also have the capability and capacity of integrating the operations of more transport lines. Prasarana and Indra foresee that the ICC will integrate the Mass Rapid Transit (MRT) and KTM Komuter lines in the future, and will have the potential for the planned High-Speed Rail from KL to Singapore to be operated from it. Furthermore, the operation of non-rail transportation systems, such as the bus network of Kuala Lumpur, could also be carried out from the ICC.

3. ICC INTEGRATED SYSTEMS

Focusing on the current aim of this project, the technological solution designed and deployed by Indra will include the integration of the following main systems:

Traffic Management System

This will include the Scheduling Subsystem and the Real-Time Subsystem, which will jointly organise the train timetables and regulate, control and monitor the train traffic in real-time. Indra's system is more than a control system; it is a complete operation management system from traffic planning to vehicle scheduling including statistics and key performance indicators monitoring. Indra has previously integrated systems in other projects from different providers such as Thales, Invensys, Siemens and Bombardier.

SCADA for Fixed Facilities

Indra's SCADA for Fixed Facilities system (HORUS) is designed to control/monitor the fixed facilities installed in the existing Kelana Jaya Line, Ampang Line and Monorail Line, providing a common graphical interface. In addition, it can centralise the alarms received and collected from all the lines.

SCADA for Energy Remote Control

It involves the control and management of all elements related to power supply for both the traction train and the operation of other equipment or facilities that exist along the way.

Security and CCTV Monitoring

The integrated CCTV System will provide a platform designed to integrate multiple unconnected security applications and devices and control them through one comprehensive and unique user interface. It will collect and correlate events from existing different security devices and information systems to empower Prasarana's personnel to identify and proactively resolve situations.

The CCTV system in the ICC will integrate the systems of 2 different providers into one user-friendly interface created for Prasarana's operators.

Radio and Telecommunication systems

Indra's technological solution to integrate the different radio and telecommunication systems in the 3 different lines is the iConnect Rail Voice Coordination System. iConnect Rail will enable an operator-train and operator-station communications as well as communication through the Administrative and public Telephony. The iConnect Rail system designed for Prasarana will enable a dispatcher operator to access the different communication systems currently operating in each line (QS2, TETRA and WIFI) from one single dispatcher. Indra's iConnect Dispatcher which will be deployed in the ICC will integrate the QS2 systems of 2 different providers, the TETRA communication system and the WIFI system, into one easy-to-use, sleek interface.

Card Access System (CAS)

The ICC CAS will be integrated with the new Prasarana CAS Infrastructure as a common CAS Solution. Operators in the ICC will be able to monitor and control the CAS in any of the 3 lines as well as in the ICC.

On top of the integration of these systems, Indra will implement the following systems in the ICC in order to improve and enhance the performance of the three metro lines:

Topology Management System

This system will allow modelling different versions of the network infrastructure and performing capacity analysis of the network for the different modelled versions. TOPMS is a simulator system which will enable the optimisation of rail traffic capacity, both for rolling stock and personnel.

Remote Monitoring System

It allows monitoring of railway components in real time within a working framework with web access. This data can be shown in 3 different views: geographical, synoptic, and time-distance view.

Dashboard and Decision Support System

This system makes use of Business Intelligence technology to collect and make an effective use of information in order to improve business effectiveness. It can be defined as a set of methodologies, processes, architectures and technologies that change raw data into meaningful and useful information. This information will then be used by Prasarana to gain more effective strategic, tactical, and operational insights and better decision-making.

All of these operational systems will be supported by the Authentication and Authorisation Infrastructure, which will control and verify the user permissions to enter the ICC systems and offer a Single Sign-On platform to access the integrated applications. Many of the systems mentioned above will be accessed from a common human-machine interface: the ICC launcher. Users in the ICC will be able to log-in to use different systems from this interface. Based on their roles and permissions (administrators or users) they will be able to access different functions and the train line they are operating.

The Equipment Control and Monitoring System will monitor and supervise the integrated architecture in the ICC. Furthermore, the ICC will be integrated with Prasarana's existing Automatic Fare Collection systems (provided by Indra) in order to collect data on revenues and ridership.

The following figures show some examples of the systems that will be integrated in the ICC and what these will look like.

Indra's iConnect Dispatcher which will be deployed in the ICC will integrate the QS2 systems of 2 different providers, the TETRA communication system and WIFI, into one easy-to-use, sleek interface.



Figure 1. iConnect Rail Dispatcher Frontal View



Figure 2. iConnect Rail Dispatcher Rear View

As another example, the CCTV system in the ICC will integrate the systems of 2 different providers into one user-friendly interface created for Prasarana's operators. This will look similar to the figure below, which shows some of the different functionalities that will be available for the operators of the 3 lines:

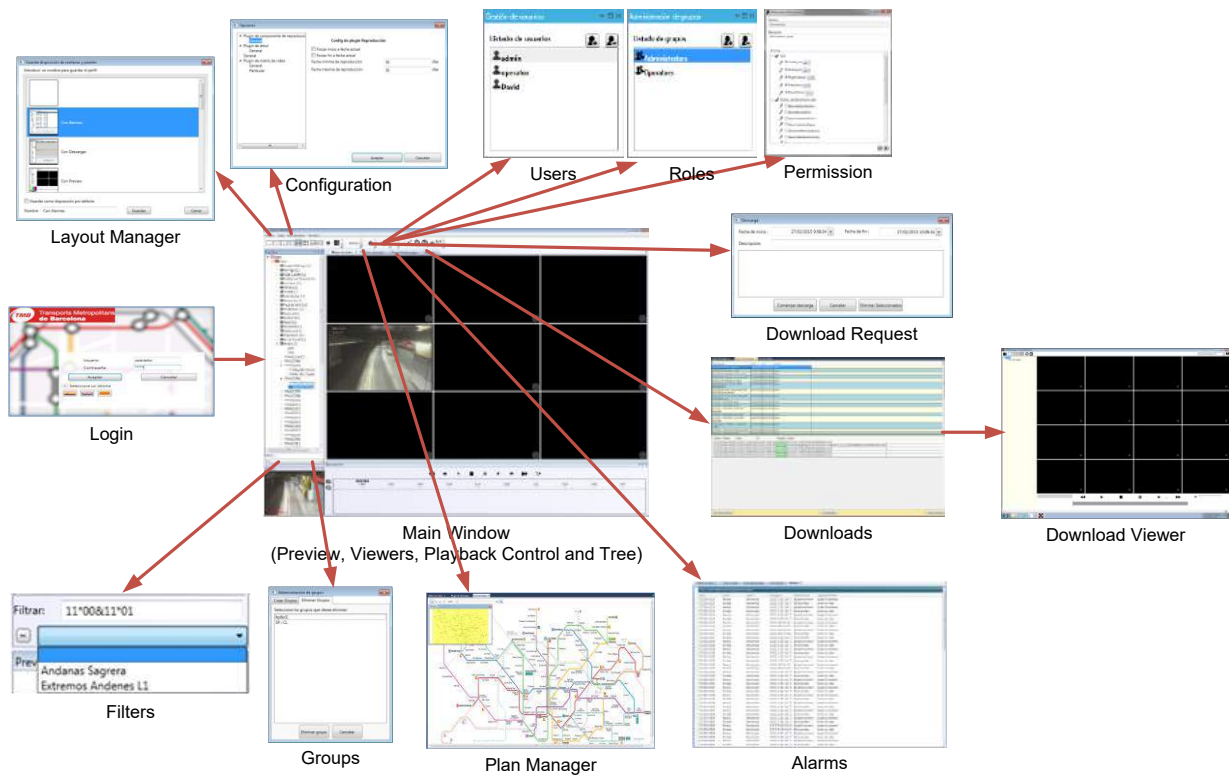


Figure 3. Indra Integrated CCTV system – example

Many of these systems will be accessed from a common human-machine interface: the ICC launcher. (The images shown below are a render of the ICC launcher and are subject to change according to the system’s final look & feel designed for Prasarana).



Figure 4. ICC launcher initial screen

Users in the ICC will be able to log-in to use different systems from this interface. Based on their roles and permissions (administrators or users) they will be able to access different functions and the train line they are operating.

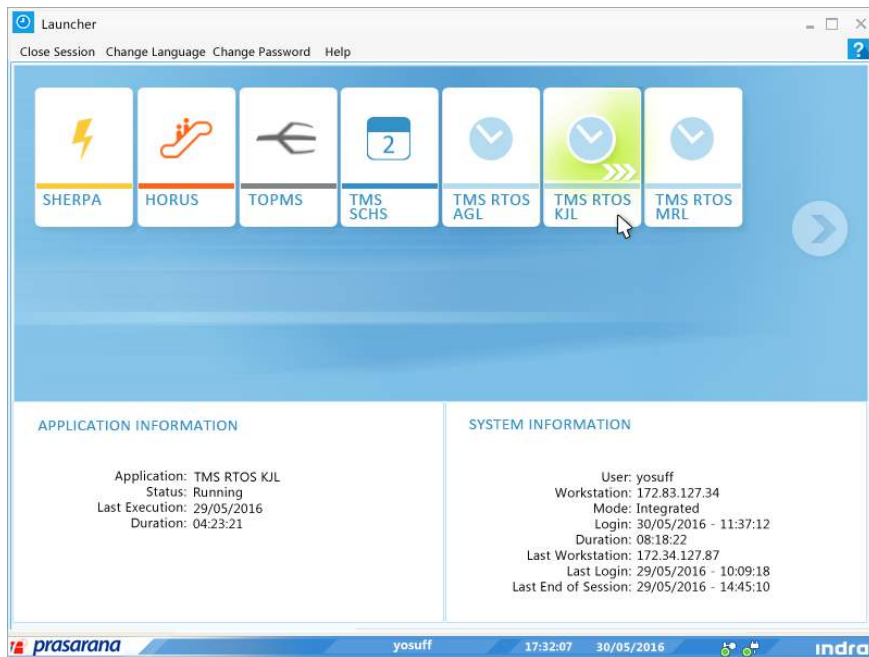


Figure 5. Launcher operational screen

In summary, Indra’s technological solution for the ICC will create an open framework for the integration of any third party technology. However, what will differentiate the ICC from other control centres will be the fact that 3 different metro lines will be operated in the same control centre, under the same roof, metres apart from each other. And herein lies the ICC’s strength and Indra’s specialty: the integration, into one common working environment, of the systems of different providers with different features and functionalities.

In fact, the integration in the ICC is especially complex since Prasarana is currently undergoing the extension of the Ampang and Kelana Jaya lines, and the fleet expansion of the Monorail line. These works imply the re-haul of many of the existing systems and the involvement of many new parties and providers into the integration required for the successful operation of the ICC.

4. ICC BUILDING

The building of the ICC, also awarded to Indra and located in Cheras, KL, has been designed to become a symbol of Prasarana and even Kuala Lumpur. It will hold a control centre with an initial 22 operating posts, with the capacity of expanding this number in the future to include more operating posts based on the additional transport lines integrated within the ICC. The control centre will also hold a continuous videowall where the operation of the lines will be displayed, also with future expansion capacity.

The control room has been ergonomically designed to provide an environment which blends functionality with comfort and ease of use.

Furthermore, the ICC will include a Crisis Management room with a direct view to the control centre, Security and CCTV Monitoring Room, Operation and Maintenance Planning Centre and Press Conference Room.

The ICC will be connected to the Operating Control Centres of the Kelana Jaya and Ampang lines, and Monorail, through a communication link, also provided by Indra. This communication link will include fiber optic cabling as well as the digital transmission backbone network.

5. IMPACT OF ICC

Operations from the ICC will allow Prasarana to benefit from more control in real time of its transportation systems, achieving increased efficiency levels and reducing operation costs by integrating the management of the different lines into only one centre. Indra's reliability in this field will bring about the highest level of automation avoiding human errors and increasing safety and punctuality.

On the other hand, Prasarana's customers and users of its transport systems will enjoy a higher quality and safer service, where interconnection between the different lines will be favoured. The integration of the ICC systems will aid Prasarana in its strategic decision-making, which will allow it to obtain service-level indicators with the aim of adjusting this to the real demand of KL's citizens, increasing performance and reducing costs.

6. CONCLUSION

Prasarana's Integrated Control Centre has been designed to become a landmark of Kuala Lumpur city and will offer Indra's cutting edge technology for the integration of the operation of 3 different transportation lines. Indra will provide the ICC with a solution which will allow the integration of all the elements of control and management of the metro systems and a higher degree of automation.

The potential of the ICC lies in its versatility and ability to integrate different public transportation systems under one roof, including buses and high-speed lines, in order for each one, Prasarana, and the general public, to benefit from the synergies and knowledge transfer created.

In view of all this, it becomes clear that the ICC will become a turning point in the way that the operations of the public transportation system in KL, and the rest of Asia in the future, are handled. Indra is proud to be at the forefront of such prominent technology and the leader of this influential multi-modal project, with far-reaching implications for the Intelligent Transportation Systems industry worldwide.

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PAPER TITLE	Geocells for Affordable Low Volume Pavements		
TRACK			
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KEYWORDS:

Pavement, access, flexible, concrete, affordable

ABSTRACT:

Developing countries are often faced with a need to extend paved road networks into remote rural areas. Paved roads significantly enhance accessibility but also come with higher capital and maintenance costs than unpaved roads. One novel solution to address the need for paving roads with low traffic volumes are concrete geocell pavements. Consisting of an interlocking set of unreinforced concrete blocks formed by a thin high density polyethylene lattice, the resulting flexible concrete surface reduces the need for granular pavement layers, resulting in significantly lower construction and maintenance costs. The objective of this paper is to present this novel pavement system and an application in the Pacific Islands. The paper summarizes the theory behind geocell pavements, presenting a simplified design method which takes into account two parameters of traffic loading and subgrade stiffness. Construction methodology is then discussed including equipment and material supply issues. Finally a case study presents successful application of geocell pavements to a road rehabilitation project in Kiribati. Conclusions reveal that concrete geocell pavements offer an affordable alternative to conventional bituminous surfacing and should be considered in the design of rural road projects.

Geocells for Affordable Low Volume Pavements

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1 INTRODUCTION

1.1 BACKGROUND

Developing countries are often faced with a need to extend paved road networks into remote rural areas. Paved roads significantly enhance accessibility and quality of life, providing all weather access and reducing the burden of routine maintenance inherent with unsealed roads. However an issue often faced is the capital cost of conventional pavement technology. Typically consisting of layers of aggregate surfaced with either concrete, asphalt or chip seal, constructing and maintaining these designs is costly and requires significant maintenance capability. As a result, traffic volumes need to be high enough to justify the investment, which can be rare in rural areas where traffic is light.

One novel solution to address the need for paving roads with low traffic volumes are concrete geocell pavements. Consisting of flexible interlocking concrete blocks, they can make extensive use of local labor, while reducing the need for granular pavement layers. Road maintenance is greatly reduced compared with other types of surfacing. Used in Africa, Australia and now in the Pacific, these geocells should be considered in any low volume road project as a cost effective way of delivering improved accessibility.

1.2 DEVELOPMENT NEED

Governments often request support from development institutions for improving accessibility for their rural communities which despite trends of urbanization still represent approximately 46% of the world's population, some 3.4 billion people (United Nations, 2015). Based on surveys of accessibility, the Rural Access Index (RAI) estimates that 1 billion of these rural dwellers are without access to reliable all-weather transport (World Bank, 2013).

Strong links have been drawn between low RAI ratings and the failure to achieve millennium development goals including poverty, maternal mortality and gender equity. Rural areas are also the source of inequality when compared with urban dwellers who have access to the services of larger settlements. By providing all weather access with a smooth running surface, rural dwellers are provided access to economic opportunities. For example, they can sell agricultural produce consistently and with less damage, and also access essential services such as healthcare and education.

While the needs are clear, the investments in rural roads need to be economically justified, which can often cause tension between Governments and Donor organizations. When traffic volumes are low the savings in road user costs do not justify the investment in conventional pavement technology. In a number of countries roads which were previously paved with bituminous surfaces have deteriorated to unpaved condition due to unsustainable maintenance costs.

The main advantage offered by geocell pavements is that they provide this access at lower cost, making extension of the sealed road network into rural areas more economically viable, as well as offering reduced long-term maintenance costs.

1.3 GEOCELL TECHNOLOGY

Geocell pavements consist of an interlocking set of unreinforced concrete blocks formed by thin plastic lattice, with the resulting composite structure serving as a flexible but impermeable pavement surface. Ranging from 75 to 150 mm thick with plan dimensions from 150 mm to 300 mm square, the cells are shaped by a high density polyethylene (HDPE) sacrificial formwork which remains in the final pavement. This plastic mold forms concrete blocks into an interlocking shape, which allows mechanical transfer of load from one block to those surrounding it, and thus provides the surfacing with an enhanced ability to resist loads—refer

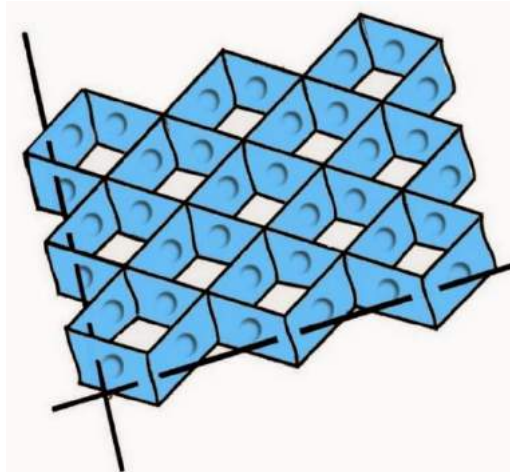


Figure 1. Diagram showing expanded geocell formwork (Hyson Geocells)

Geocells differ from conventional block paving due to a mechanical keying which occurs as a result of the distorted face of the cells. Protrusions in one block key in to cavities in adjacent blocks, creating a joint which is better able to transfer loads. This interlocking effect means that geocell pavements are effective in spreading load and can do without the aggregate base course and subbase layers of traditional pavements, relying instead on load transfer between blocks. Doing away with these layers gives a lower cost solution for low volume roads which makes extending sealed networks more affordable.

While geocells with aggregate or concrete fill have a range of applications for example as waterproof channel liners, and for slope stabilization, they have not been widely used for pavements outside South Africa where they were developed despite offering significant benefits.

The objective of this paper is to present this novel pavement system and an application in the Pacific Islands. Firstly the paper will summarize the theory behind geocells pavements before describing the design approach and construction methodology. Finally a case study will be presented describing the application of geocell pavements on a project in the Republic of Kiribati, an equatorial Pacific Island.

2 GEOCELL PAVEMENT SYSTEM

2.1 ENGINEERING THEORY

A deeper understanding of the application of Portland cement concrete filled geocells can be formed by considering the engineering theory which underlies the technology. The use of flexible concrete pavements formed by geocells was documented by Visser and Hall (1999). Their initial approach required base and subbase layer works, with the resulting requirement for extensive roller compaction effort proving to be a constraint in some developing countries where such equipment is scarce. In subsequent work (Visser and Hall 2003), their approach evolved to include the consideration of remote rural areas with a structural design approach developed to be congruent of equipment, terrain and labour conditions.

In this work they stated that a pavement's primary purpose is to protect underlying weak layers of soil, which can be composed of fine grained or clayey soils. According to mechanistic design methods of South Africa (Theyse et al. 1996) and Australia and New Zealand (Austroads, 2008), the critical pavement design parameter is the vertical compressive strain of the subgrade, or the amount the underlying soil deforms in the direction of the applied traffic load.

By constructing instrumented experimental sections from concrete filled geocells, Visser and Hall's analysis revealed that stiffness of the geocell pavements depended on the stiffness of the subgrade layers. Therefore increased subgrade stiffness resulted in increased geocell layer stiffness, while thicker cells also increased stiffness. Note that due to the inherent flexibility of the concrete geocell layer (especially thin ones), the cells themselves provide little additional stiffness. Instead, stiffness gains come from two areas. Firstly the lock-up which occurs during large deflections, where the load on a single block is distributed to surrounding blocks, with this effect more prominent on thicker geocells. A linear relationship was subsequently identified between geocell thickness and the logarithm of geocell stiffness. Secondly, the impermeable geocell layer enhances pavement stiffness by preventing saturation of underlying subgrade which can become plastic when moisture content is high. Once protected these soils are much more able to resist traffic loads.

Using a transfer function provided by Theyse et al (1996) which relates the number of repetitions of an equivalent standard 80 kN axle (ESA) load for a rut depth of 20mm to the allowable vertical compressive strain, equation (1) was developed.

$$\text{Allowable vertical compressive strain } (\mu\epsilon) = 10^{\frac{-\log_{10}(\text{number of ESA repetitions})+36.7}{10}} \quad (1)$$

Finally, using ELSYM5 linear elastic layer software, the vertical compressive strains at the top of the subgrade layer were calculated, with results presented in Figure 2 for a range of geocell pavement thicknesses.

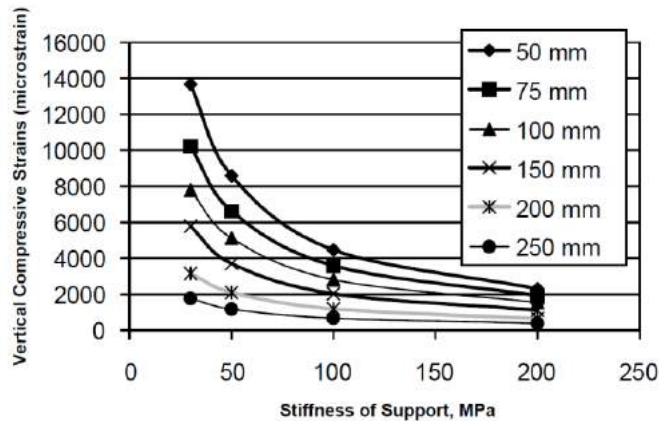


Figure 2. Computed vertical compressive strains applied by an ESA load on subgrade for a range of geocell thicknesses (Visser & Hall, 2003)

This theory provides a sound basis for design of concrete geocells pavements by following the methodology described below.

2.2 DESIGN METHODOLOGY

Perhaps the most valuable aspect of the Visser and Hall (2003) work is the simplified design approach they presented. The approach takes into account inputs of traffic and loading and subgrade stiffness, and provides practitioners with guidance on a geocell pavement layer thickness.

To apply the theory they developed, the first step is to calculate the pavement traffic loading using conventional methodology including traffic counts and or estimation. Average daily traffic should be adjusted to yearly traffic, then

annual traffic growth added for each year, before summing the traffic over the design life of the pavement. Any traffic loading must be converted to ESA loading, which can then be used in equation (1) to arrive at an allowable vertical compressive strain.

The next step is to determine subgrade stiffness. In many cases in developing countries, this information is not readily available, with a dearth of information for basic subgrade material properties like California bearing ratio (CBR). Fortunately, an estimate of in situ CBR can be obtained using a dynamic cone penetrometer (DCP), a ubiquitous piece of soil test equipment. The Shell relationship (2) (Heukelom & Klomp, 1964) can then be used to determine subgrade stiffness, noting that there is some variation in the factor used. Visser & Hall (2003) suggested that where subgrade conditions are poor (CBR < 3), a factor of 3 can be used, while for CBR > 10 a factor of 10 is a good starting point. A linear relationship between CBR and factor can be assumed between these values.

$$\text{Subgrade stiffness (MPa)} = Z \times \text{CBR} \quad (2)$$

Where $1 \leq Z \leq 100$

As a guide: $Z = 3$ for $\text{CBR} < 3$, $Z = \text{CBR}$ for $3 \leq Z \leq 10$, $Z = 10$ for $\text{CBR} > 10$

Armed with these two inputs of allowable vertical strain and subgrade stiffness, practitioners can use Figure 2 to calculate the required geocells pavement thickness. By drawing a vertical line up from corresponding support stiffness value, and noting where it intercepts with a horizontal line matching the allowable compressive strain, a point can be plotted. Identifying the charted line which lies closest to this point from the legend, the required cell thickness is provided. If a conservative design is required, the thickness corresponding with the line below the plotted point can be selected.

For example shown in Figure 3, for an allowable strain of 6000 $\mu\epsilon$ and a subgrade stiffness of 50 MPa, a geocell thickness of 75mm is required (or 100mm for a conservative design).

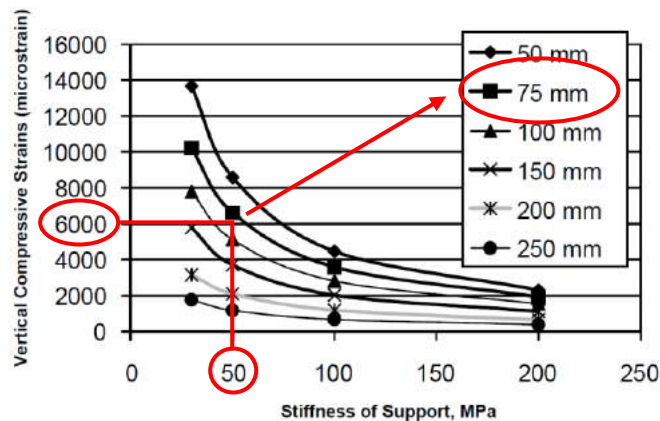


Figure 3. Application of Figure 2 to example case (Subgrade stiffness 50 MPa, Allowable vertical strain 6000 $\mu\epsilon$)

This simple design methodology can be readily applied and thus makes the approach accessible for the developing country context where the resources for complex pavement modeling are limited.

2.3 CONSTRUCTION METHODOLOGY

With the design thickness established, discussion can now turn to the construction methodology. Much of the physical work required can be completed by either labor or equipment based methods, for example edge beam or geocell or tuck-in areas can be excavated by hand, or with mechanical excavators. This can provide work for laborers which assist with poverty reduction goals. Geocells also have the advantage of being able to be constructed in very constrained alignments—including curves—where large paving equipment would have difficulties in operating or in densely populated urban areas which prohibit street closures.

The first step to construct the geocells pavement is preparation of the subgrade. This can include filling in low spots with select fill, and shaping the surface to achieve fall. Excavation for edge beams is also required. These beams provide horizontal restraint to the geocells and prevent water ingress. Wooden formwork is required to form the beams, and is removed after concrete has reached sufficient strength. Secondly, it is necessary for the subgrade to be compacted, using either small rollers (refer Figure 4) or hand operated plate compactors.



Figure 4. Compaction of subgrade with mechanical roller. Note edge beam formwork in place.

Thirdly the HDPE geocell formwork can be anchored in place and tensioned with steel reinforcing pegs, then expanded to the full width and length of the road section (refer Figure 5). Once expanded and tensioned, the geocells will be ready for placement of high-slump concrete. This is mixed, poured and spread into the cells before broom finishing and curing. The modular nature of the geocells means it can be constructed using small mixers, or alternatively for large areas if concrete agitator trucks are available.



Figure 5. Geocell formwork expanded and tensioned and ready for placement of concrete.

After curing of concrete, edge beam formwork can be removed and edges of pavement backfilled, with pavement usable as soon as concrete strength of 15 MPa is reached, probably within one week, although the mix design would have a strength of 30 MPa for durability.

2.4 MATERIALS REQUIRED

In order to construct a geocell pavements, a range of materials are required. Perhaps the most difficult to source is the HDPE geocell formwork. At present, only one supplier produces the thin (0.2mm thick) formwork which the theory described above is applicable for, but as the technology is adopted more widely other suppliers are more likely to produce equivalent products. Conventional materials required include Portland cement from local suppliers if available,

or imported if no such supply exists. The sand and aggregate required for the concrete should be supplied from sustainable local sources, and can be of a quality which might otherwise be unsuitable as pavement or surfacing materials. A target concrete mix strength of 30MPa is typically specified, and the workability required for placing in the geocells means that a high slump mix of 120 to 150mm is needed. This slump can be difficult for contractors to achieve and a superplasticizer can assist with achieving a sufficiently workable mix.

2.5 PERFORMANCE AND MAINTENANCE

Given that geo-cells rely on the subgrade layer for strength, they are typically less able to withstand the heavy loads and traffic of traditional designs. Despite this, vehicle loads are transferred to surrounding blocks in a way which doesn't occur with traditional block paving, which results in good pavement performance despite the absence of granular pavement layers. Thin geocells pavements are clearly well suited to applications where traffic loading is light. For a 20mm rut to form in a 100mm thick geocell pavement (assuming a subgrade CBR of 10) it would take 2000 ESA passes which is equivalent to a typical truck loading, but 124 million passes for a small vehicle with a 50 kN axle load. Thicker geocells should therefore be used for pavements with truck traffic, as well as considering granular base course and subbase layers. By applying the theory described above with proper supporting pavement layers the system has been successfully used for container terminal pavements where axle loads of 100 kN are applied by heavy loading equipment.

Because geocells are typically installed without granular pavement layers, the subgrade is less protected from excessive loads, and this can lead to a significantly shorter pavement life compared with granular pavements. This problem may be exacerbated where high water tables or weak subgrades exist, the latter may require stabilizing which may offset any cost savings from omission of granular layers.

The modular nature of geocells also allows for individual blocks to be removed for pothole repair and service installation without the more extensive reinstatement required for traditional pavements. Geocell pavements also have benefits in difficult geographic and climatic conditions. In mountainous terrain, the steepness of longitudinal drainage channels can lead to scour and eventual pavement failure, however geocells can be placed with drainage integrated into the road surfacing at center or edge of the road. This effectively armors the channel and prevents a common failure mode. Performance of geocells has not been observed in situations where frost heave is an issue, as application has so far been restricted to warm climates.

In thin thickness and with no granular support layers, geocell pavements are well suited to light traffic and difficult terrain and spatial conditions for both rural and urban roads, offering an affordable pavement solution in these applications.

2.6 CASE STUDY

Geocells have recently been used on the Kiribati's Road Rehabilitation Project (KRRP) being undertaken by the Government of Kiribati with support of the World Bank. With a development objective of improving the condition of South Tarawa's roads and strengthening road financing and maintenance capacity (World Bank, 2011), the ambitious project is due for completion in November 2016. Included in the project is rehabilitation of 6.8 km of pothole ridden feeder roads with geocell pavements – refer Figure 6. As a remote coral atoll, material supply was an issue for the project with some pavement materials needing to be imported by barge from Fiji, some 2000km of open ocean away. In this context, 5m wide geocell pavements of 75mm thickness cost AU\$140,000 per kilometer, 72% of the cost for chip seal surfacing, and 53% of asphalt.



Figure 6. Before (left) and after (right) on a KRRP project feeder road in Kiribati

While these pavements have only been installed for a short time, pavements in South Africa have demonstrated service lives of more than 10 years without periodic maintenance. That said, additional research is required to document the long-term performance of geocells and establish whole of life costs (including maintenance) to allow for better comparison with traditional pavements.

While the contractor in Kiribati noted long lead times for geocell formwork and issues with getting a concrete mix with a sufficiently high slump, the geocell feeder roads are now considered one of the project's success stories. No complaints were received from adjacent residents despite the need for complete road closure to vehicles until the geocell pavement had cured. The geocell pavements have significantly enhanced the amenity and accessibility of the communities they serve. The success in Kiribati prompted geocells to be considered for paving sections of road in the neighboring Pacific nations of Samoa, Tuvalu and Papua New Guinea.

3 CONCLUSIONS

Geocell pavements represent an attractive solution for paving low volume roads. With significantly lower costs compared with traditional pavements due to a reduced need for granular pavement layers, and the potential for a labor intensive construction approach which assists with poverty alleviation goals, the benefits are numerous.

While the rewards of increased accessibility are clear, expanding the sealed network can often lead to capital and maintenance costs which are difficult to justify economically. Geocell pavements represent a novel way of addressing the need for all weather accessibility at a significantly lower initial construction cost as well as reduced routine and periodic maintenance requirements. All projects involving low volume roads should consider concrete geocells as a pavement option.

4 ACKNOWLEDGEMENTS

This paper was prepared with the guidance and review input of Dr Alex Visser and Dr Chris Bennett who have both supported the wider adoption of geocells pavements on development road projects in the Pacific.

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PAPER TITLE	Status of Electronic Toll Collection (ETC) and the Development of a New ETC-based Traffic Information Service in Korea		
TRACK	Intelligent Transportation Systems		
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KEYWORDS:

Electronic Toll Collection, ETC, Expressway, Traffic Information, Vehicle Trajectory, DSRC, Section-based Detection

ABSTRACT:

Hi-pass system is the brand name of the electronic toll collection (ETC) system of the Korea Expressway Corporation. Since 2004, the Hi-pass system has been employed to collect toll fares on expressways throughout the country. Recently, the share of the vehicles equipped with the onboard unit (OBU) for the Hi-pass system exceeded 65% of the total registered vehicles in Korea. In addition, the Hi-pass system has been used to collect data on the section travel time of traveling vehicles equipped with the OBU; this was achieved by installing antennas along the main lines of expressways. The antennas can read the ID of passing vehicles with OBUs by means of dedicated short-range communication (DSRC). This study was initiated to utilize the travel time data collected using DSRC to develop a new ETC-based traffic information service. First, an algorithm to estimate the path travel time using ETC-based individual vehicle trajectories was developed in this study. This algorithm was used to estimate the path travel times between two major cities in Korea. Second, the behaviors of vehicles parked in service areas were analyzed using the travel time data collected using the DSRC.

Current Status of Electronic Toll Collection (ETC) and the Development of a New ETC-based Traffic Information Service in Korea

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1 INTRODUCTION

Most of the expressways in Korea have closed-type toll collection system(TCS). In such a system, the driver pays toll fare based on the distance traveled on expressways. The Korea Expressway Corporation (KEC) is the biggest government-run company operating expressways in Korea. The KEC is using a TCS for collecting toll fare. For the convenience of drivers using expressways, KEC has employed an ETC system referred to as “High-pass” system since 2004. This Hi-pass system is now the brand name of KEC's ETC system. The share of the vehicles equipped with onboard unit (OBU) for the Hi-pass system exceeds 65% of the total registered vehicles in Korea. In addition, the share has been continuously increasing since 4.5 ton or heavier trucks were allowed to use the Hi-pass system and since low-cost OBUs were made available for the Hi-pass system. Accordingly, the share of vehicles with OBUs is expected to reach 80% of the total registered vehicles in the near future.

The KEC developed the Freeway Traffic Management System (FTMS) for better traffic operations. In the FTMS, the basic traffic data, including speed and volume, are collected through a vehicle detection system (VDS), which consists of loop detectors and image sensors. The VDS is utilized to collect point-based traffic data. However, because of the limitations of point-based traffic data, an additional step is performed to estimate section-based traffic information, including a segment travel time, using the point-based traffic data. In general, the values estimated by means of point detection are usually less precise than the directly observed values obtained via section detection. Hence, the KEC decided to use the data collected through the TCS and the Hi-pass system in order to increase the accuracy of traffic data for segments or routes.

Using the data collected from the TCS, the travel time between major cities was estimated; this was possible because the TCS records the entry tollgate, exit tollgate, entry time, and exit time of every trip via the expressway network operated in the closed-type TCS. However, the travel time determined based on the TCS data have some shortcomings. When the TCS data are used, the actual route of an individual trip cannot be identified. This is because the TCS only records the entry tollgate, exit tollgate, entry time, and exit time even if multiple routes exist between an entry tollgate and an exit tollgate. In addition, screening the vehicles parked or rested at a service area is difficult, and this may increase the estimated travel time as an outlier. Moreover, the available sample size dramatically decreases in the case of a long distance trip. Therefore, acquiring sufficient sample size and guaranteeing the reliability of the estimated travel time are technically difficult, especially in the case of long-distance trips.

In order to overcome the aforementioned shortcomings, the Hi-pass system was used to collect data on the section travel time of traveling vehicles equipped with OBUs. For this purpose, Hi-pass system's antennas were installed along the main lines of expressways. The antennas can read the ID of passing vehicles with OBUs by means of dedicated short-range communication (DSRC).

This study was initiated to utilize the travel time data collected using the DSRC in order to develop a new ETC-based traffic information service. To this end, first, an algorithm to estimate the path travel time using the ETC-based individual vehicle trajectories was developed in this study. Using the algorithm, the path travel times between two major cities in Korea were estimated. In addition, the behaviors of vehicles parked in service areas were analyzed using the travel time data collected via DSRC.

2 LITERATURE REVIEW

Dixon and Rilett (2002) and Chun et al. (2005) suggested that a travel trajectory may be applied to verify the relationships among links. In addition, they claimed that the trajectory data might be useful for estimating travel speed in a section. According to a survey conducted by Guillaume, the traditional data collection method employing sensors on the road is necessary but not sufficient. Travel trajectories obtained using the Global Positioning System (GPS) or cellular phones have become more important in the development of Intelligent Transport System. Such trajectories can provide significant information such as traffic conditions and alternative routes to users (Guillaume 2008).

Park et al. (2009) and Im et al. (2009) proposed the concept of calculating the representative value for link travel time using vehicle trajectories based on wireless communication. However, the trajectory data applied in these surveys were simulation data. Hence, there were limitations in the application of this concept to real systems. Lee et al. (2011) surveyed the establishment of the dynamic origin-destination (OD) using vehicle trajectory; OD was expected to facilitate data collection during the deployment of SMART Highway's¹ Wireless Access in Vehicular Environment (WAVE) communication. However, as in the case of Park et al.'s research, this survey too had a shortcoming in that simulation data were used.

Jing et al. (2010) developed the optimal route algorithm for designated departure time using 30,000 probe cars with GPS in Beijing. The field test results showed an average travel time reduction of 16%. Kwon and Pravin (2005) developed a method for computing partial trip trajectory and estimating real-time OD based on FasTrak ETC system.

3 CURRENT STATUS OF ELECTRONIC COLLECTION SYSTEM IN KOREA

The primary function of the Hi-pass system is to enable payment without stopping at a tollgate. The KEC uses these payment data for generating traffic information. The Hi-pass system provides data on travel time and frequency between the entry and exit tollgates. However, the derived travel time differs from the actual travel time owing to route diversity and service area's rested vehicles. The Hi-pass utilization ratio is approximately 64% of the total expressway trips, and this value corresponds to 70 million trips annually. The KEC currently operates the Hi-pass system for 709 of 970 entry lanes and 428 of 1412 exit lanes nationwide. As shown in figure 1, two types of OBUs—infra red (IR) and radio frequency (RF) OBUs—are adopted. When a Hi-pass OBU-equipped car enters the entry tollgate, the TCS will issue the entry statement. This statement includes the entry tollgate number, time, and vehicle class. The exit statement will be issued when this car passes the exit tollgate. The exit statement comprises the exit tollgate number, time, travel distance, travel time, and fare in addition to the elements mentioned in the entry statement.

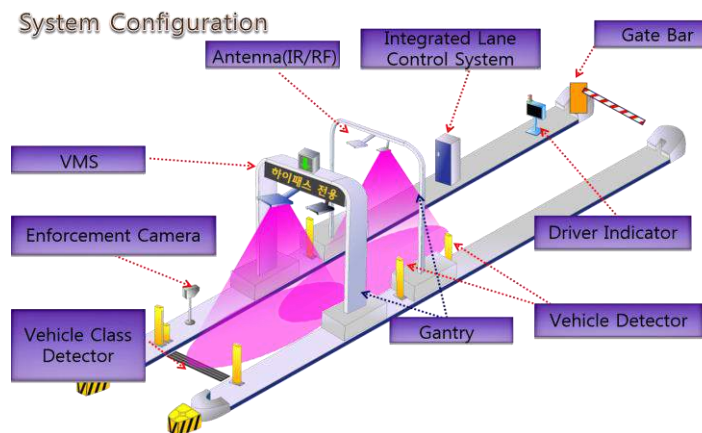


Figure 1. Configuration of the KEC Hi-pass system

The Hi-pass traffic information system is derived from the Hi-pass system. This system comprises roadside equipment (RSE) and RF-type OBUs. This system generates the segment travel time of the main line by means of the RF-type OBU. Each RSE collects the encrypted OBU ID, communication time, OBU type, and vehicle class. The traffic information center first searches for a specific OBU ID in the data collected by numerous RSEs within the indicated time duration. If the same OBU ID is detected at different RSEs, the trip is considered a partial trip. Then, the center searches for other partial trips with the same OBU ID until no more partial trips are found. Thereafter, the center estimates the section and path travel time from each trajectory. Figure 2 shows the configuration of the Hi-pass traffic information system.

¹ SMART Highway is the brand name of the KEC's super highway that includes C-ITS service.

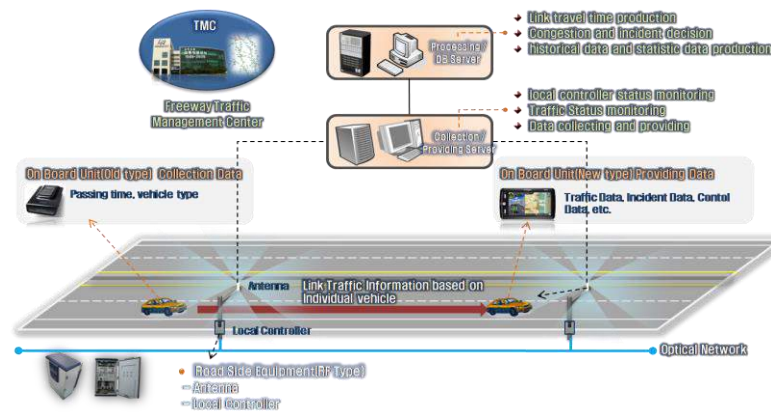


Figure 2. Configuration and function of the Hi-pass traffic information system

Data on each criterion such as segment and path are collected. Finally, the representative travel time will be determined by screening each trip's travel time. At present, the KEC operates 967 RSEs nationwide.

4 ALGORITHM DEVELOPMENT FOR TRAVEL TRAJECTORY

If the obtained data are perfect, the travel time and trajectory can be produced easily. However, actual data always have imperfections. Hence, additional preprocessing is necessary to guarantee reliability of the obtained value. The imperfections reason may be attributed to various reasons such as operation failure of RSEs, interruption by OBU-equipped cars on frontage roads beside expressways, the OBU power getting turned off when driving, duplicate communication, and overstaying of vehicles in service or parking area. Table 1 lists these abnormal data cases.

Table 1. Abnormal data cases and their influence on information

Abnormal data case	Influence on traffic information
Re-detection of an exit vehicle on an expressway	Abnormal travel time
Overstaying of a vehicle in service or parking area	Abnormal travel time
Interruption by an OBU-equipped car on a frontage road beside an expressway	Reversal and discrete travel trajectory
Turning off of OBU power when driving	Reversal and discrete travel trajectory
Operation failure or breakdown of an RSE	Discrete travel trajectory

When these imperfection factors cannot be filtered out, they may cause various abnormalities in the traffic information, such as excessive travel time, re-detection of a vehicle that has already exited the expressway, and a momentary reversal progression on the opposite side of the trajectory. Thus, the travel trajectory generation algorithm developed in this study reflects various abnormalities.

Sorting each RSE ID by communication time with same OBU ID is a simple method to create travel trajectories. Nevertheless, some problems may occur. First, the TCS and Hi-pass traffic information system operate independently. Therefore, mutual data transaction between the systems is impossible. Therefore, the whole travel time for the entry tollgate-mainline-exit tollgate route cannot be determined in the current system environment of the KEC. Hence, the vehicle that exited through the Interchange (IC) and re-entered from the same IC on the same day cannot be distinguished for each trip and trajectory. Second, detection of the re-entered car by the simple sorting method is impossible because of missing data and communication failure.

Trajectory tracking is the process of connecting points that are revealed as the target vehicle's detected locations. Using the section-based detection method, the traffic information center gathers the individual vehicle's location, and then combines every vehicle's detected location by matching the license plate or specific ID structure in the upstream and downstream data. This is a transparent process to check whether the target vehicle sequentially passed along the pre-defined RSE's spatial and temporal arrangement. The target vehicle's OBU ID may be missed in a certain section even though the ID exists in the RSE's communication log for the upstream and downstream sections. This means that the target vehicle exited, detoured, and re-entered the expressway. Therefore, trajectory's continuity has to be terminated and re-created.

The simplest method to establish a travel trajectory, which only comprises the communicated RSEs according to the communication time, frequently leads to discontinuities in the trajectory owing to various reasons. In this study, the concept of “Conzone”, which does not include entry and exit ICs in the section, was designed to solve these problems. If a vehicle has one RSE communication log at least once within a Conzone, the vehicle has passed the whole section of the Conzone. This concept expands the required basic spatial unit from point-based to section-based units. Figure 3 shows the relation between Conzone and RSE composition.

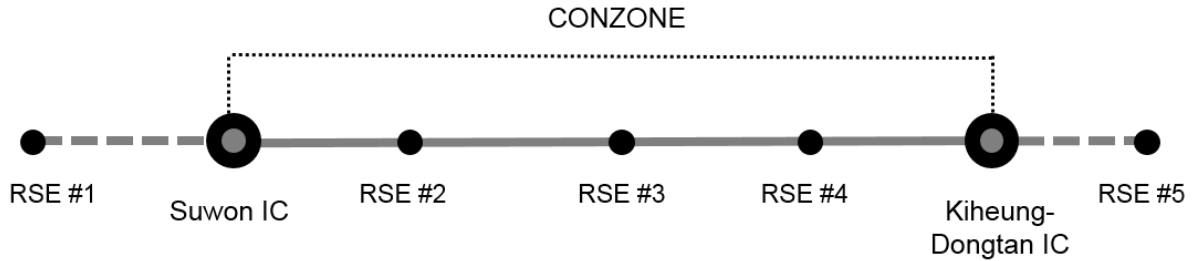


Figure 3. Expansion of the tracking of spatial units and the concept of Conzone

As can be seen from figure 3, if RSE #1, 2, and 5 have the target vehicle's communication log, but RSE #3 and 4 do not, the trajectory will be terminated at RSE #2 and restarted at RSE #5 in the RSE-based tracking method. Consequently, two discrete trajectories will be created. On the contrary, as the basic spatial unit on Conzone, the trajectory in the above case will be continuous from RSE #1 to #5 because three Conzones (upstream of Suwon IC, Suwon IC-Kiheungdongtan IC, and downstream of Kiheungdongtan IC) can satisfy the minimum requirement of at least one communication log within one Conzone.

The time coverage of the developed algorithm for matching OBU IDs is from D-1 to D+1 day to minimize the loss of late departure and early arrival of vehicles of a target day (e.g., a vehicle departs at 11 P.M. on day D-1 or arrives at 2 A.M. on day D+1). The travel sequence for the commuting or regular route bus, which has to be considered to detect repetitive trips along the same routes, is also introduced. The feature of the RSE that one RSE simultaneously contacts both up-bound and down-bound vehicles cannot be used to assign the direction for the collected data. Further, the reversal progression step is avoided by checking the RSE sequential master data. Table 2 presents the travel trajectory algorithm for filtering abnormal patterns in the collected data.

Table 2. Abnormal data cases and their influence on information

Step	Description
1	Get 72 h (D-1, D, D+1) data including those for the target day
2	Remove duplicate data in 30 min
3	Classify by vehicle type
4	Select by origin and destination
5	Sort RSE data by communication time with RSE for each vehicle
6	Compute travel time and travel sequence for each vehicle
7	Check direction of RSE data
8	Match RSE attribute Conzone master data
9	Connect Conzones that were matched in the previous step

The developed algorithm was implemented by a database query to produce a travel trajectory. The data comprise all RSE data from September 17 to 19, 2013. In this survey, the analysis range is set as long distance trips between major cities. The analysis results show that the average ideal trajectory ratio of total actual trips confirmed by the TCS data among particular OD is approximately 42%. Table 3 lists the analysis results.

Table 3. Analysis results of ideal trajectory's sample size and detected ratio

Origin	Destin.	Date	Total trips ² (vehicles)	Cars passing OD ³ (vehicles)	Ideal trajectories ⁴ (vehicles)	Ideal trajectory ratio
Seoul	Gangreung	Sep. 17	740	613	253	34.2%
		Sep. 18	753	632	182	24.2%
		Sep. 19	684	561	170	24.9%
	Gwangju	Sep. 17	2,748	2,478	1,272	46.3%
		Sep. 18	2,902	2,521	937	32.3%
		Sep. 19	1,628	1,504	700	43.0%
	Daegu	Sep. 17	2,906	2,581	1,173	40.4%
		Sep. 18	2,783	2,310	972	34.9%
		Sep. 19	1,604	1,401	627	39.1%
	Daejeon	Sep. 17	4,280	3,931	2,709	63.3%
		Sep. 18	5,579	4,982	3,162	56.7%
		Sep. 19	3,658	3,348	1,956	53.5%
	Busan	Sep. 17	436	370	170	39.0%
		Sep. 18	415	337	148	35.7%
		Sep. 19	216	184	70	32.4%

5 TRAJECTORY ANALYSIS RESULTS

In this survey, the long-distance path travel time and path distribution ratio within the complete trajectory were determined. The confidence interval method is often applied to screen outliers in travel time pre-processing (Ki et al., 2010). In this survey, the confidence interval is set as 1σ . The target route was from the Seoul tollgate to Daegu tollgate; there are numerous routes connecting these tollgates, and samples can be easily obtained.

Figure 3 presents the network diagram of the paths between Seoul and Daegu tollgates. Path 1 is a single Gyeongbu line. Path 2 is a combination of lines along Gyeongbu-Yeongdong-Jungbunaeryuk-Gyeongbu. The last line is a combination of lines along Gyeongbu-Cheongwonsangju-Jungbunaeryuk-Gyeongbu. The TCS will produce the representative travel time between Seoul and Daegu as path #1 in KEC's system environment in spite of the existence of other paths.

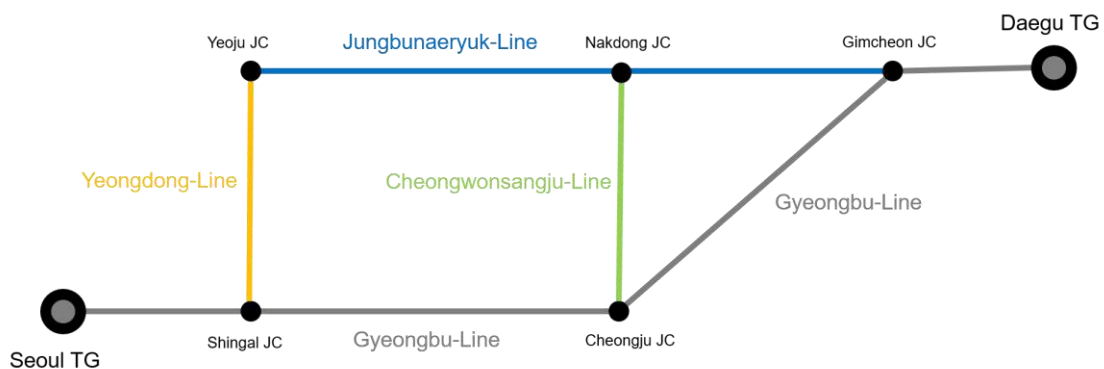


Figure 4. Network diagram showing the paths between Seoul TG and Daegu TG

Table 4 lists an analysis example corresponding to September 17, 2013. The distribution ratio is the ratio of the number of vehicles along each path to the total number of sample vehicles. Congestion length is the summation of the section congestion length of each path.⁵ The congestion ratio is the ratio of the total congestion length of each path and the path length.

² The number of total vehicles between the origin and destination. This is a true value based on the TCS.

³ The number of total vehicles that are detected at the origin and destination RSEs. This value does not mean that each vehicle has a perfect trajectory.

⁴ The number of vehicles that have ideal trajectories, and were detected at the origin and destination.

⁵ KEC's congestion criteria is the Conzone's spatial mean speed below than 40km/h.

Table 4. Travel time analysis example of Seoul–Daegu on Sep. 17, 2013 (partial example)

Time	Travel time (min)				Distribution ratio			Congestion length (km)			Congestion ratio		
	#1	#2	#3	TCS	#1	#2	#3	#1	#2	#3	#1	#2	#3
0	162	171	195	210	17%	63%	21%	0	0	0	0%	0%	0%
1	151	188	166	191	14%	71%	14%	0	0	0	0%	0%	0%
2	152	163	0	201	17%	83%	0%	0	0	0	0%	0%	0%
3	0	183	243	191	0%	67%	33%	1	0	1	0%	0%	0%
4	255	175	183	183	24%	65%	12%	0	1	0	0%	0%	0%
5	165	181	195	181	12%	85%	4%	0	0	0	0%	0%	0%
6	175	178	208	176	21%	59%	21%	0	0	0	0%	0%	0%
7	253	191	183	183	26%	48%	26%	0	0	0	0%	0%	0%
8	195	206	181	178	31%	49%	20%	0	1	0	0%	0%	0%
9	187	184	175	181	26%	66%	8%	0	1	0	0%	0%	0%
10	214	221	195	188	20%	54%	25%	0	0	0	0%	0%	0%
11	232	271	207	228	38%	56%	6%	2	1	2	1%	1%	1%
12	210	256	192	234	44%	34%	21%	1	3	1	0%	1%	0%
13	204	349	253	217	60%	8%	32%	1	1	1	0%	0%	0%
14	232	329	314	239	62%	25%	13%	2	4	5	1%	2%	2%
15	241	332	242	268	54%	34%	12%	11	13	18	4%	5%	7%
16	264	347	324	254	64%	24%	12%	15	17	22	6%	7%	8%
17	261	351	276	274	66%	27%	7%	28	19	32	11%	8%	12%
18	251	358	277	287	58%	30%	12%	38	26	37	14%	11%	14%
19	270	390	284	295	58%	27%	15%	37	37	38	14%	16%	15%
20	280	323	278	296	63%	25%	12%	27	32	30	10%	13%	12%
21	253	332	256	288	51%	30%	19%	21	29	24	8%	12%	9%
22	224	319	320	283	62%	23%	15%	18	27	21	7%	11%	8%
23	233	304	247	305	52%	32%	16%	12	25	18	5%	11%	7%

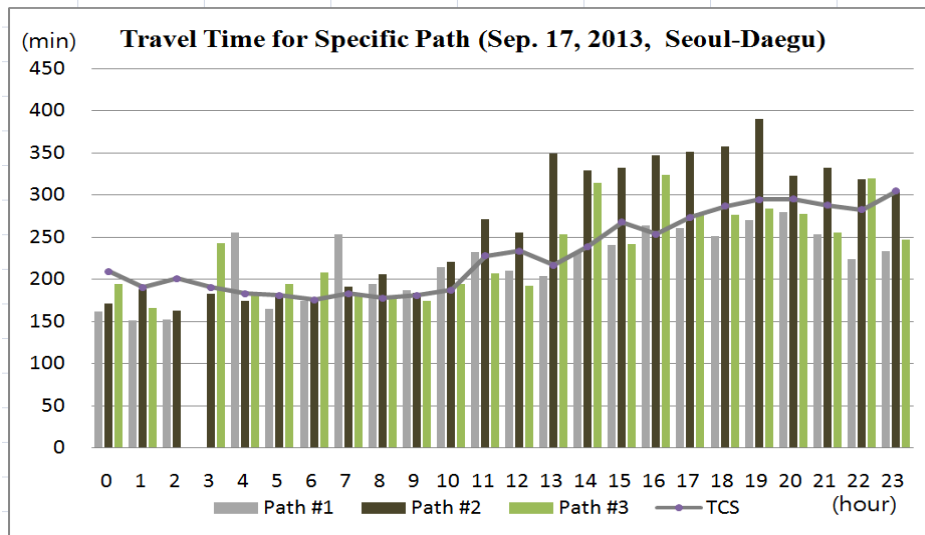


Figure 5. An example of travel time analysis for Seoul–Daegu for Sep. 17, 2013

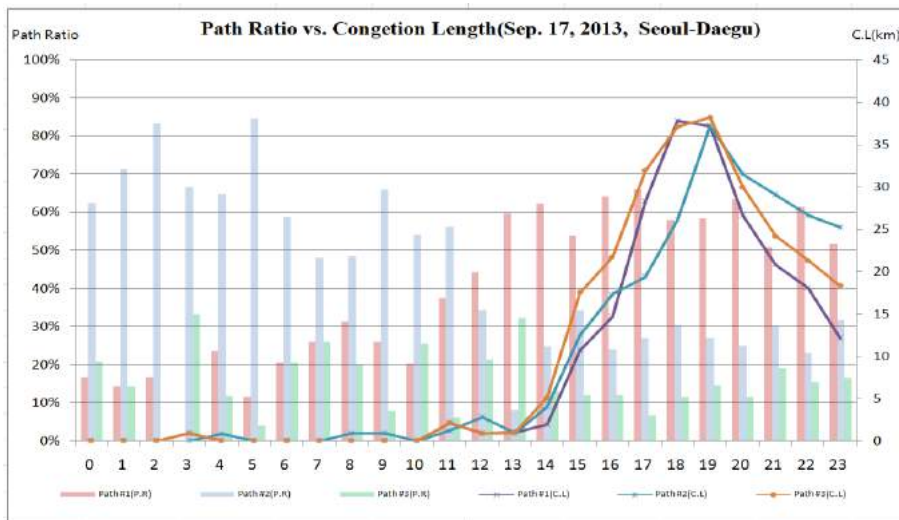


Figure 6. Relationship between distribution ratio and congestion length (Sep. 17, 2013)

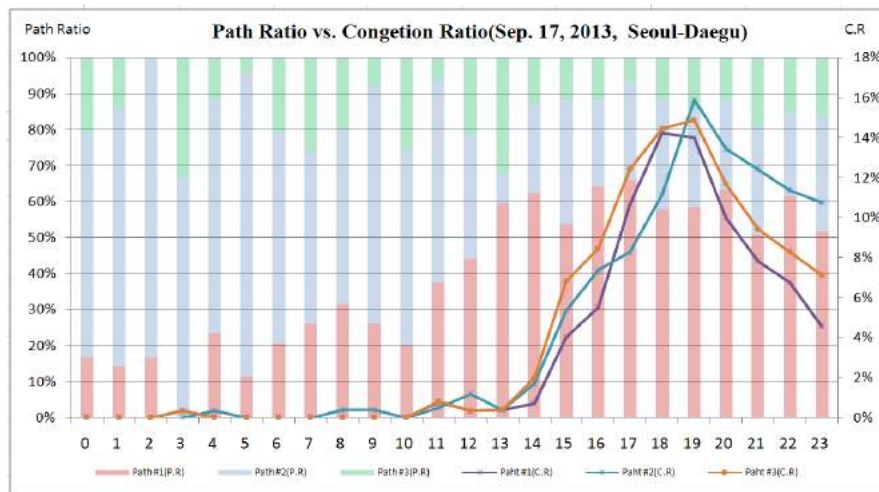


Figure 7. The relationship between distribution ratio and congestion ratio (Sep. 17, 2013)

According to abovementioned results for the Seoul–Daegu trip, the main path, which has the highest distribution ratio within the non-congestion condition, may be path #2. The main path's distribution ratio declines, and that of path #1 relatively escalates as the congestion increases. Meanwhile, the distribution ratio of path #3 is steadily low and is not affected by congestion. Under the assumption that users are aware of the traffic condition in their route, the congestion length is considered to influence the route choice from among the activated paths.

A correlation analysis was performed to determine the factors—from among congestion length, congestion ratio, and VDS-based path travel time⁶—that influence route decision. Table 5 lists the results of correlation analysis. The correlation coefficients of paths #1 and 2 are higher than 0.5; this result implies that a positive and negative correlation exist between traffic information on a route and route decision.

Table 5. Correlation analysis results

Distribution ratio of path #1	Congestion length 1	Congestion ratio 1	VDS path travel time 1
	0.502	0.501	0.557
Distribution ratio of path #2	Congestion length 2	Congestion ratio 2	VDS path travel time 2
	-0.624	-0.624	-0.683
Distribution ratio	Congestion length 3	Congestion ratio 3	VDS path travel time 3

⁶ VDS-based travel time was used in this study to satisfy the sample size requirements for correlation analysis.

of path #3	0.163	0.163	0.109
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6 ANALYSIS RESULTS OF SERVICE AREA USAGE

The KEC also operates RSEs in service areas to avoid overestimating the travel time when considering mainline travel time. If an OBU-equipped vehicle stops in a service area, this vehicle communicates with the service area RSE; the vehicle is then filtered as an outlier. Using this system configuration, the service area usage behavior was analyzed based on travel trajectory. The criterion for screening of the parked vehicle in a service area is a time difference of more than 5 min between the service area RSE and the next mainline RSE.

The completed travel trajectory of RF-type OBU vehicles is approximately 1.2 million per day; this value is 33% of the total day trips on the nationwide expressway.⁷⁸ Vehicle class 1, which includes passenger cars, vans (less than 16-seaters), and trucks (less than 2.5-ton maximum load capacity), represents the highest portion among total trips, but the service area visit ratio is not high in comparison with the occupancy of total trips. Vehicle class 2 includes buses (16–35 seaters) and trucks (maximum load capacity 2.5–5 tons) and has the lowest visit ratio. Vehicle class 3 comprises buses (more than 36 seaters) and trucks (maximum load capacity over 5 tons) and have a visit ratio approximately equal to that of vehicle class 1. Table 6 lists the fraction of the total number of vehicles visiting the service areas.

Table 6. Analysis of the service area visits of vehicles

Vehicle class	Total number of travels	Number of vehicles visiting a service area	Percentage of vehicles visiting a service area
1	1,162,245	60,388	5.2
2	6,309	220	3.5
3	41,864	2,152	5.1
Total	1,210,418	62,760	5.2

In this study, the service area usage pattern was analyzed depending on travel distance. According to the analysis results for the travel distance distribution of population, short distance travel of less than 50 km contributed most of the proportion of the total trips. Further, the trip frequency was found to decrease as travel distance increased. The results for service area use ratio showed behavior opposite to that of the results for travel frequency. That is, the service area use ratio increases as the travel distance increases. In the case of trips of more than 50 km for a class 1 vehicle, the use proportion of each interval especially increases by two times. Table 7 presents the analysis results for total trip and service area usage ratio depending on the travel distance.

Table 7. Analysis results for total trip and service area usage ratio depending on the travel distance

Distance	Classification	Vehicle Class			Total
		1	2	3	
<50 km	Total trips	764,352	4,316	19,341	788,009
	SA ⁹ visit frequency	9,607	54	117	9,778
	Ratio	1.3%	1.3%	0.6%	1.2%
50–100 km	Total trips	188,988	1,159	8,321	198,468
	SA visit frequency	14,806	70	214	15,090
	Ratio	7.8%	6.0%	2.6%	7.6%
100–200 km	Total trips	152,534	633	8,257	161,424
	SA visit frequency	21,093	45	606	21,744
	Ratio	13.8%	7.1%	7.3%	13.5%
> 200 km	Total trips	56,371	201	5,945	62,517
	SA visit frequency	14,882	51	1,215	16,148
	Ratio	26.4%	25.4%	20.4%	25.8%
Total	Total trips	1,162,245	6,309	41,864	1,210,418
	SA visit frequency	60,388	220	2,152	62,760

⁷ The source data for August 10, 2014, were used for this analysis. This analysis differs from previous trajectory analysis for several reasons.

⁸ The entire allowance for the truck Hi-pass over than 4.5 ton began on March 29, 2016. So, Trucks that weigh over 4.5 tons are not included in this population.

⁹ SA is an abbreviation of service area

	Ratio	5.2%	3.5%	5.1%	5.2%
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Analysis results of usage frequency depending on travel distance show that the usage frequency for trips of less than 50 km is 0.01 on an average. This observation can be interpreted as follows: drivers rarely use service areas on short distance trips. The usage frequency for class 1 vehicles for trips longer than 200 km is 0.29, which is the highest value among vehicles of all classes. The average for all the vehicles of the total distance was 0.13. Table 8 presents the analysis result of usage frequency for service area depending on trip distance.

Table 8. Analysis results for usage frequency depending on travel distance

Travel distance Vehicle class	<50 km	50–100 km	100–200 km	>200 km	Average
1	0.01	0.08	0.15	0.29	0.13
2	0.01	0.06	0.08	0.28	0.11
3	0.01	0.03	0.08	0.21	0.08
Total	0.01	0.08	0.15	0.28	0.13

The results for travel distance depending on departure time show that vehicles that departed during daytime cover longer travel distances than those that departed during nighttime with regard to average travel distance depending on departure time. The primary purpose of class 3 vehicles is transport of passengers or cargo, and these vehicles traveled longer than class 1 vehicles, which are mainly used for commuting or visits.

Table 9. Analysis results for travel distance depending on departure time

Departure time Vehicle class	20–06	06–09	09–17	17–20	Average
1	51 km	59 km	57 km	51 km	55 km
2	49 km	45 km	50 km	54 km	50 km
3	76 km	92 km	96 km	93 km	89 km
Total	51 km	61 km	58 km	53 km	56 km

Further, analysis of service area visit frequency depending on departure time showed that vehicles departing during daytime have much higher visit frequency than those that depart in the nighttime. However, vehicles departing in the morning peak hours of 6:00–9:00 AM have the lowest service area visit frequency. This behavior may be attributed to the characteristics of commuting trips. Table 10 presents an analysis summary for service area visit frequency with regard to departure time.

Table 10. Analysis results for service area visit frequency depending on departure time

Departure time Vehicle class	20–06	06–09	09–17	17–20	Average
1	9,787	6,704	37,882	9,823	64,196
2	79	44	75	33	231
3	209	481	1,083	476	2,249
Total	10,075	7,229	39,040	10,332	66,676

7 CONCLUSIONS

In this study, the current status of the ETC employed in expressways in Korea was introduced. Using the ETC data, an algorithm for estimating travel trajectory was developed in order to build a new traffic information service. After preprocessing the vehicle trajectory data obtained from the RSEs of the Hi-pass system, a tracking method was developed to estimate the path travel time between major cities. The developed algorithm allowed estimation of the actual path travel time and path distribution, which could not be obtained using the TCS data. Using the path travel time data, the congestion length, path travel time, and path distribution ratio were correlated. The path distribution ratio was found to be directly related to the traffic congestion. The distribution of the main path was found to be high under

non-congested conditions, and then, the route decision tended to shift to an alternative route with increase in congestion.

In addition, the behavior of vehicles parked in service areas was analyzed using ETC data. Traditionally, the behaviors of vehicles parked in service areas were analyzed via interviews and surveys, and were not subjected to data-driven analysis. Data-driven analysis can provide information on the number of vehicles parked in service areas and the frequency of parking by vehicle classes, travel distances, and departure times.

Diverse and innovative traffic information services such as estimation of detour rates on rural roads and incident detection can be considered in future study using the ETC data. This study may contribute to the development of the core algorithm and different applications for new services. The use of data from RF-type OBUs has shortcomings because the penetration rate is approximately 45% of the total OBUs. For using these results, an additional process to expand the data for the total number of trips is necessary. This problem can be solved by obtaining the population data through the SMART Tolling System¹⁰ until 2020.

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¹⁰ The SMART Tolling system is an advanced tolling system of KEC. This system allows to charge and pay on the express mainline without stopping. KEC is planning to operate nationwide until 2020.

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PAPER TITLE	Review of Estimation Method of Crack Depth in Asphalt Paving		
TRACK			
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KEYWORDS:

asphalt pavement, crack, crack depth estimation, crack characteristic value

ABSTRACT:

Among the pavement repair work conducted by NEXCO using high-functionality pavement (porous asphalt pavement), there is an increasing number of cases where the base layer is exposed to road surface water and even the roadbed needs to be replaced. When the repair depth has to be changed during work, it greatly affects the work period, and so it is desirable to precisely estimate the damage depth. General methods of determining damage depth include measuring cracks by core boring or with the falling weight deflectometer. However, it takes time and cost to achieve precision with these point-based survey methods, and traffic control is necessary. Considering these drawbacks as well as the focus on the importance of customer service, these methods are not recommended. This paper describes an evaluation of a crack depth estimation method that analyzes crack characteristics based on crack data obtained by a measurement technique that requires no traffic control..

Review of Estimation Method of Crack Depth in Asphalt Paving

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1 INTRODUCTION

Among the pavement repair work conducted by NEXCO using high-functionality pavement (porous asphalt pavement), there is an increasing number of cases where the base layer is exposed to road surface water and even the roadbed needs to be replaced. When the repair depth has to be changed during work, it greatly affects the work period, and so it is desirable to precisely estimate the damage depth. General methods of determining damage depth include measuring cracks by core boring or with the falling weight deflectometer. However, it takes time and cost to achieve precision with these point-based survey methods, and traffic control is necessary. Considering these drawbacks as well as the focus on the importance of customer service, these methods are not recommended. This paper describes an evaluation of a crack depth estimation method that analyzes crack characteristics based on crack data obtained by a measurement technique that requires no restriction control..

2. Crack Measurement System using a Laser

Since images taken by an ordinary road surface survey vehicle (line sensor camera) fail to clearly show cracks in a type I high-function pavement that is not continuously graded, a crack measurement system using a laser (LCMS: Laser Crack Measurement System) was used instead. This system, developed by Pavemetrics, Canada, is capable of obtaining coordinate positions and dimensions in a 5-meter long lane section as crack data and digitizing appearance characteristic values, based on which we can estimate the crack depth.

3. Selection of Crack Characteristic Values and Estimation Method

12 characteristic values were obtained from identification and analysis of the crack characteristics based on the obtained crack data (Table 1). The Random forest algorithm was selected from the proposed machine learning algorithms as the method to estimate the crack depth based on those characteristic values.

Table 1. List of crack characteristic values

No. of vertexes
Average crack width
Total crack length
Most angles
Average width of the longest cracks
Angle histogram
Average length of the cracks
Length of the longest crack
Angle of the longest crack
No. of cracks
Average measured depth
Average measured depth of the longest cracks

4. Learning with Random Forest and Procedure of Crack Depth Estimation

Random forest is a machine learning algorithm proposed in 2001. It has various features including the capability of running at high speeds even with a large amount of data and can evaluate the extent to which each characteristic value is effective for classification in the form of a contribution rate. A conceptual diagram of a random forest decision tree is shown in Fig. 1, while an estimation based on many decision trees is shown in Fig. 2.

The procedure of learning and crack depth estimation by the random forest algorithm is (1) preparing many decision trees with a branch structure (flow chart) based on a randomly adopted crack characteristic values and two or more judgment criteria in each characteristic value and (2) learning by giving right answers to the goals reached by applying the randomly selected crack data (sets of characteristic values) to each decision tree (actual crack depth, which is hereinafter “damage level”) then, (3) incorporating the crack data about which we want to estimate the damage level, to all decision trees, total the damage levels given to the goals of the reached decision trees, and estimate by majority vote.

5. Review of Estimation of Crack Depth

To review estimations of the crack depth, the crack data for each mesh are obtained, the actual damage level of each mesh (damage level A: crack having reached the upper base course; damage level B: crack having reached the binder course; and damage level C: crack remaining inside the surface course) is checked, and 723 datasets containing the crack characteristic values and actual damage levels are prepared.

However, since a large bias is determined to occur in the damage level, data from an area with a relatively good balance is doubled and data from areas with serious biases are randomly deleted so as to keep a balance. Eventually 617 sets were prepared. Of them, 447 sets (70%) were randomly extracted, and those sets were used (learned) to prepare 500 decision trees.

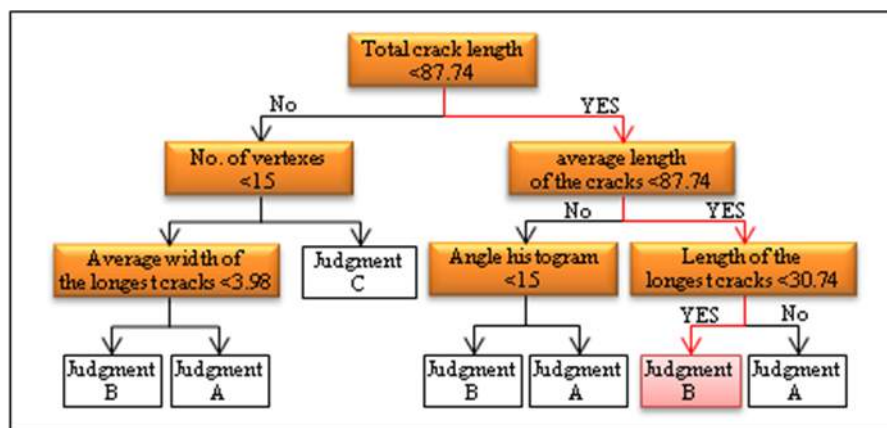


Figure 1. Conceptual diagram of decision trees in the random forest algorithm

6. Result of Crack Depth Estimation

The crack data of the remaining 170 sets (30%) not used in preparing the decision trees were given as tasks to experimentally estimate the damage level. The results of crack depth estimation are shown in Fig. 2. The overall match ratio is 88%, and the match ratio of each damage level is 93% for level A, 67% for B, and 85% for C, which are good results (Fig. 3) (Table 2). There are four characteristic values with higher contribution rate, namely “the number of crack peaks,” “the total length of cracks,” “the number of cracks,” and “the length of the longest crack.” It is thus revealed that the “depth” of a crack is proportional to its “length.”

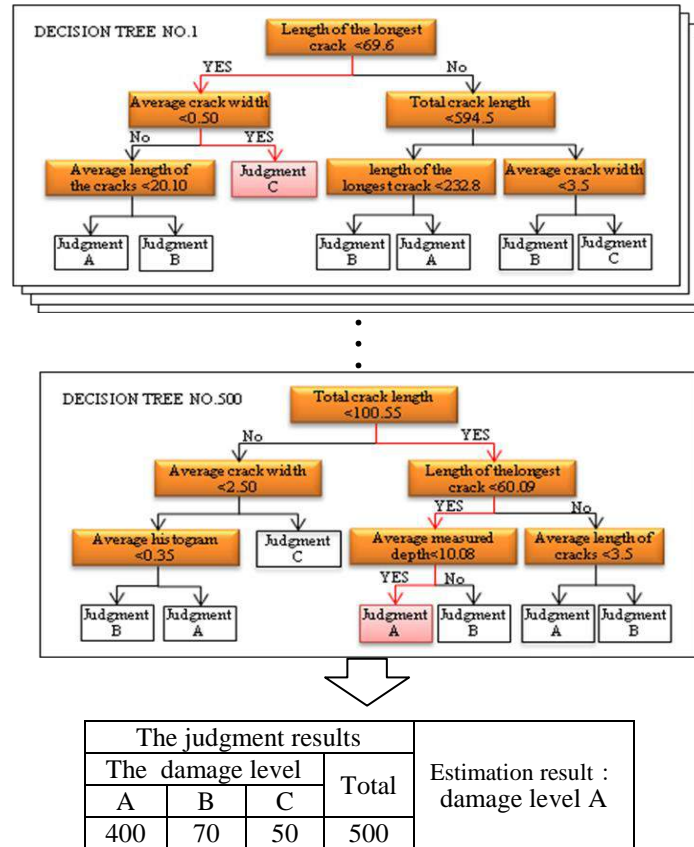


Figure 2. Conceptual diagram of estimation with many decision tree

7. Summary

The review made by the authors as reported in this paper confirmed that the damage level (crack depth) can be estimated by using the “crack characteristic values” (such as length) obtained from the laser-based measurement system (LCMS). Instead of the ordinary road surface survey vehicle using line sensor camera, “crack measuring” using said method can simultaneously “estimate crack depth.” This is considered to contribute to a reduced need of traffic restrictions for advance surveys or improve work efficiency by reducing plan changes. Tasks requiring further study include the non-uniform area coverage of the samples used in this review and the extremely small sample size of damage level B data. The authors intend to increase the amount of data (to reduce biases) and promote study toward establishment of this highly versatile crack depth estimation method.

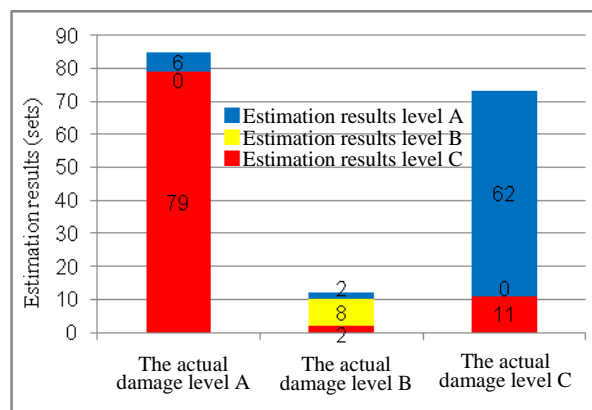


Figure . Result of crack depth estimation

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PAPER TITLE	A Basic Study on Comparison of Lane Width Measured with Sensors		
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KEYWORDS:

LiDAR, Camera, Lane width, Road Safety

ABSTRACT:

This study has selected lanes and widths of the lanes as the object for recognition among the road factors which should be recognized necessarily by the unmanned vehicle and compared the results of the accuracy measured under various conditions. To proceed the research, the National Highway which was relatively recently and thus of which the road condition is favorable was selected and as the equipment to measure and recognize the width of the lane through the actual measurement, the CCD camera and 3D LiDAR were used. According to the analysis result, the after the sunrise, the averages of the widths of the lane using the image or LiDAR were 3.34m and 3.21m and the RMSE was 0.19 and 0.34 and thus it was shown that the measurement of width of the lane using the image was more favourable in the phase of the accuracy. However, before the sunrise, the width of the lane was not detected in the image and in the case of the measurement of the width of the lane using the LiDAR, the average was 3.23m and the RMSE was 0.33 and thus the limitation of the measurement of the width of the lane using the image was proposed.

A Basic Study on Comparison of Lane Width Measured with Sensors

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1. INTRODUCTION

Various studies and technologies have been developed and proceeded in order to improve the safety of the road domestically and internationally and especially, the development of the technologies to decrease accidents due to human factors among traffic accident triggers is being proceeded. According to the actual analysis of the main causes of the Road Traffic Accidents, it is being reported that over 90% of the traffic accidents are occurring due to human factors. Lane Departure Warning System(LDWS), Smart Parking Assist System(SPAS) and Advanced Driver Assistance Systems(ADAS) as the various methods to reduce accidents due to human factors are being developed and used actually, which focus on supplementing the safe driving of the driver. Recently, being developed from the simple system which helps the safety driving of the driver, the vehicle in which the driver does not manipulate the steering wheel, but which drives mindlessly through the object recognition by the vehicle itself has passed the test driving and is driving actually on the road. For this unmanned vehicle to drive, the recognition of street structure technology for the actual section that the vehicle drives and the positioning technology are required necessarily, through which the street furniture should be recognized on-line which exists on the road. The median barriers, curbs and lanes and so forth which compose the basic elements of the road among the facilities existing in the road are the objects which should be recognized necessarily for safe driving of the unmanned vehicle. Especially, the recognition of the street furniture should be possible under various conditions which reflect the actual driving environment and an accuracy of over a certain level should be guaranteed as well. Especially, in case of the lanes, they are the element which should be recognized necessarily to secure the driving safety of the vehicle and to ensure safe driving by differentiating the vehicle groups and up to now, the lanes are detected and width of the lane is measured using the image recognition or LiDAR.

Thus, this study has selected lanes and widths of the lanes as the object for recognition among the road factors which should be recognized necessarily by the unmanned vehicle and compared the results of the accuracy measured under various conditions. To proceed the research, the National Highway which was relatively recently and thus of which the road condition is favorable was selected and as the equipment to measure and recognize the width of the lane through the actual measurement, the CCD camera and 3D LiDAR were used.

2. LITERATURE REVIEW

This chapter reviewed the existing studies which focus on recognizing the road information using various sensors.

Choi et al. (2012) have studied on the visual Odometry to presume the location of the vehicle driving in the road environment. For the position recognition, the stereo camera to acquire the 3 dimensional information in the front of the vehicle and the single camera to acquire images from the back of the camera were used. An experiment was performed about two long-distance routes acquired under the actual driving environment, and according to its result, over 97% of the matching success rate was showed, which proposed the conclusion that the methodology used in this study can presume correct position. Jang (2005) used the digital video camera and shot and analyzed the road paved in asphalt concrete and extracted the information on the road. He has compared the result with the outcome by way of the measurement of the base point and analyzed them and proposed the average errors of 0.0427m for the X axis, 0.0527m for the Y axis and 0.1539m for the Z axis. Also, the cracking rate, plastic deformation and longitudinal roughness, the road surface evaluation factors were able to be obtained through the processed image, through which the commonality index and maintenance index were calculated and the road evaluation was performed in the object road. Kim & Lee (2008) has performed the study of creating 3 dimensional geometric model of the road automatically by using the LiDAR data and numerical map. They have identified the points measured in the road surface through the division and grouping process of the Data Cloud inside the road section and have extracted the linear and surface information of the road. Yun et al. (2006) have proposed the way of measuring the cross slope of the road by using the vehicle for analysis of road safety. They have developed the algorithm which measures the cross slope during driving, using the vehicle which is equipped with the GPS-IMU and laser scanner and compared it with the actual data and verified statistically. Mun & Seo (2008) have developed the image analysis model which can measure swiftly using the line scan camera for the facilities of which the installation interval and heights are regularized according to the design(operation) speed and geometric structure among various road safety facilities. They have applied the developed model on the site and proposed the measurement results of the sizes and installation intervals of the facility. Jung et al. (2011) have performed the study of using the vehicle in which GPS-INS is installed and collecting the geometric structure

information. They have used the Genetic Algorithm and developed the algorithm to analyze the horizontal alignment and differentiated the direct line, curve and easement curve and so forth. The algorithm was evaluated using the freeway and national highway data and it was analyzed that each showed 90.48% and 88.24% of classification accuracy. Jang et al. (2015) have used the fact that the lanes are brighter than the road and extracted the characteristics of being strong to the illumination using the difference between the lanes and the surface of the road and the width of the lane and by using the extraction result of the characteristics, detected the straight lanes. By using the detected straight line and camera information, candidate curve groups were created in the 3 dimensional road and curve lanes among which the curve lanes were determined. Also, the speed and accuracy were improved using the detection information of the previous frame so that curve lanes can be traced swiftly in continuous frames. Kim et al. (2013) created a few putative factors for tracing through the combination of the characteristics of the Background Subtraction, LK-Optical Flow and community-based histogram, through which they proposed the tracing method which is relatively strong to the objects with change and noise. In other words, they have proposed the method of reliable tracing for external environment also through the verification method of the characteristic points traced by the optical flow which is strong to the change of the color and illumination, verification based on the regional histogram and the mixing method of these trackings. The laser distance sensor was used to propose the method of swift recognizing of the surrounding environment in urban environment made for an unmanned vehicle. Data was collected using the laser sensor and continuous points classification was performed, and detection and removal of the ground, object division and object classification tasks were performed. The conclusion was proposed that the realtime of the surrounding environment recognition algorithm of the unmanned vehicle was secured through the methods proposed in this study. Kim et al. (2013) have proposed the method of recognizing the obstacles real time in the actual road condition using the LiDAR. They have classified the scanned data into the data inside the road and data outside the road and the data inside the road was clustered using the Euclidean Distance. They also have obtained the location and size of the detected obstacles and calculated the speed and distance. The effectiveness of the algorithm has been verified using the LiDAR installed in the front bumper of the unmanned vehicle and the result of a swift 2ms of performance time has been proposed.

3. OVERVIEW OF ANALYSIS

The GPS-IMU to measure the location information and the image equipment or LiDAR to measure the objects are needed for the recognition of the lane and the measurement of the width of the lane. Information on each sensor used in this study is as follows.

3.1 3D LiDAR

The LiDAR sensor used in this study is the PUCK(16 channels) by Velodyne. The PUCK(16 channels) of Velodyne detects the scope of 360° horizontally and recognizes objects using 16 LiDARs per each 2° at 30° angle vertically. The points cloud can be measured up to 100m maximum and the table 1. has proposed the main specification of the Velodyne sensor used in this study.

Table 1. Specification of 3D LiDAR

Channel	16 channels
Accuracy	± 3 cm (typical)
Field of view (horizontal/azimuth)	360°
Rotation rates:	5 - 20 Hz
Wave Length	903 nm (min/max range 896/910nm)
Operating range	up to 100 m
Max altitude of operation	200 m
Angular resolution (vertical)	2°
Angular resolution(horizontal/azimuth)	0.1° - 0.4°

3.2 3D CCD Camera

This study has selected the Manta-G 125 in order to measure the width of the lane using the images. The CCD(Charge Coupled Device) camera was installed to collect the image of the section driven and the resolutions of the

camera installed were 1936 X 1456, 40 FPS(Frame Per Second) and CCD Progressive shape for the shooting. Table 2. has proposed the specification of the video equipment.

Table 2. Specification of CCD Camera

Resolution	1292 x 964
Max frame rate at full resolution	30 fps
Type	CCD Progressive
Interface	IEEE 802.3 1000baseT
A/D	14 bit
Output	8-12 bit
Sensor Size	Type 1/3
Sensor	Sony ICX445
Cell size	3.75 μm

3.3 Data Acquisition System

The investigation equipment for road inspection was used as the data acquisition system in this study. The vehicles in which each sensor was installed was used and the data collected through the sensors were synchronized based on the GPS-IMU location. The installation of sensors was showed Figure 1.



Figure 1. The installation of Sensors

3.4 Summary of the experiment

Result measured in the National Highway No.37 sector was proposed for the comparison of the widths of lanes. The detection was made for 1.6km road and the driving speed was 80km/h on the average. The object section used for the analysis of the width of the lane was a section which was constructed 15 years ago and is relatively good condition and the width of the lane of the object section was measured through actual measurement. As the representative time for the comparison between night and day, the measurement was made before and after the sunrise based on the sunrise time (5:12AM).

4. METHODOLOGY FOR ANALYSIS

This study has used the LiDAR and the image equipment in order to recognize the lane for an unmanned vehicle to drive and measure the width of the lane to drive along and this chapter proposed the process for the measurement of the width of the lane.

4.1 Detection of width of the lane using the LiDAR

The 3 dimensional LiDAR used in this study can measure the 3 dimensional coordinate values(x, y, z) from the sensor to the point and intensity reflected in the object. The lanes were recognized through the intensity measured by the LiDAR and reflected and for the detection of the width of the lane, the plane coordinates were extracted among the

3 dimensional coordinates and reflected into the X-Y axis coordinates and the threshold that the lanes are detected were deducted. The ROI was configured to improve the accuracy and detection of the lane and was set to be 5m in the rear and 5m in the right and left from the center of the LiDAR installed in the vehicle. Also, the 180° needed in the rear was set to be FOV(Field of View) and the data acquisition cycle was set to be 10Hz. The scope of intensity acquired in the Velodyne was 0 ~ 255, the average intensity of the road paved in asphalt was under 20 and the intensity about the lane was under 60. As the result of the analysis result through the pilot test performed in this study, the case that intensity was detected to be over 45 was used as the threshold about the detection of the width of the lane. The intensity of over the threshold was detected through the Velodyne and the lane was detected and the right and left side from the center axis of the vehicle was searched and the width of the lane was calculated. Figure2 proposed the lane detection screen using the LiDAR.

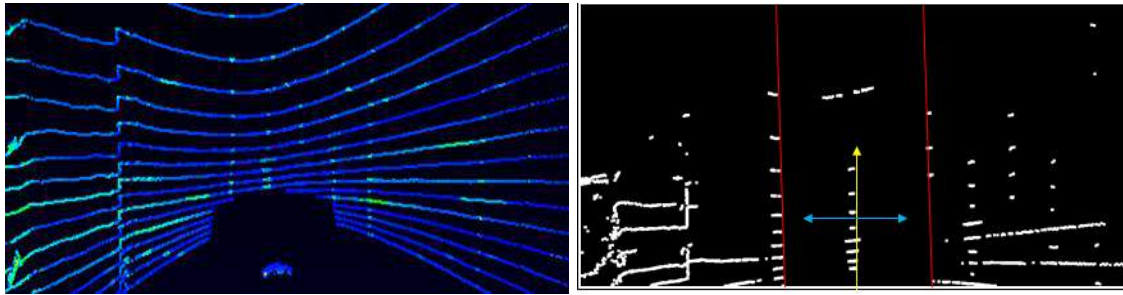


Figure 2. The process of lane detection

4.2 Detection of the width of the lane using the image

The camera installed in the front of the vehicle was used in the measurement of the width of the lane which uses the image. Calibration has been performed for correct measurement of the width of the lane and as in the figure 2, focus boards were installed in 2m intervals for the calibration and the image was acquired and the actually measured distance information was matched to the pixel coordinates of the image.

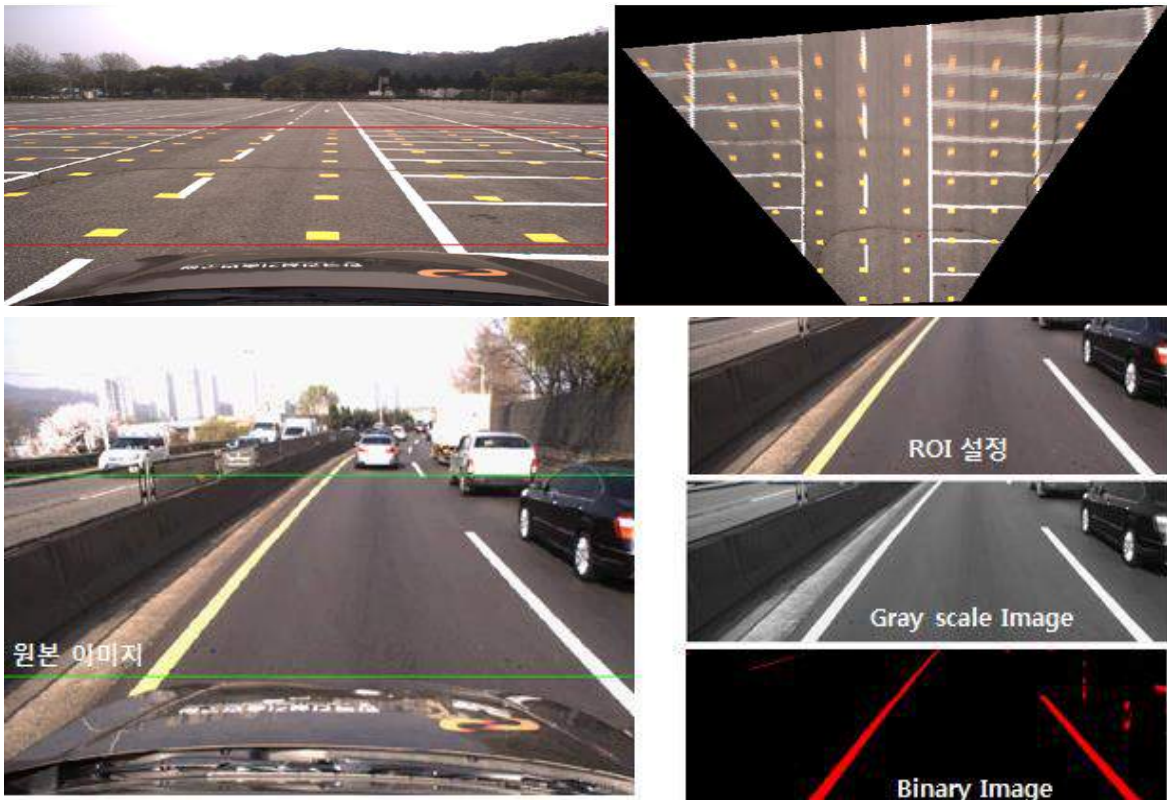


Figure 3. Image acquisition for the calibration

This study has used the IPM(Inverse Perspective Mapping) about the data acquired through the image equipment and detected the width of the lane. IPM conversion is the method of configuring the ROI(Region of Interest) of the object scope to be observed, removing the perspective effects, reflecting each pixel to other locations and creating new 2 dimensional pixel arrangement. 20m of the scope observed was set to be the ROI for the detection of the width of the lane and after binarizing the color image of the acquired data, the left and right from the center of the vehicle(center of the image) was searched and the pixel coordinates of the image with lanes are found. Convert the calibrated data of the pixel coordinates of the lanes acquired into the actual coordinates and the width of the lane is calculated using the actual coordinates of the lane.

5. ANALYSIS RESULT

Measurement result of the width of the lane through the measurement, image and LiDAR of the section of the National Highway No. 37 for the comparison of the width of the lane and the compared errors were proposed. In the result of the measurement before the sunrise, in case of having used the image, the width of the lane was not measured and in case of having used the LiDAR, the minimum width of the lane was 2.4m, maximum width of the lane was 3.79m and the average width of the lane was 3.23. After the sunrise, in case that the LiDAR was used, the minimum width of the lane was 2.61m, maximum width of the lane 3.79m and the average width of the lane was 3.34m. The summary result of the width of the lane using the measurement, image and LiDAR is proposed in the table 3.

Table 3. Comparison of descriptive statistics

	Measurement	Before the sunrise		After the sunrise	
		Image	LiDAR	Image	LiDAR
Minimum(m)	3.5	None	2.40	2.61	2.40
Maximum(m)	3.5	None	3.79	3.79	3.79
Average(m)	3.5	None	3.23	3.34	3.21

The result of analysis through the absolute values comparison of the results measured using the image and LiDAR with the measurement values is proposed in the table 4. The RMSE of the measurement result of the width of the lane using the LiDAR before sunrise was 0.33, RMSE of the measurement result of the width of the lane using the image after the sunrise was 0.19 and the RMSE of the measurement result of the width of the lane using the LiDAR was 0.34. According to the analysis result, a difference about image recognition result existed for during the day.

Table 4. Comparison of RMSE of analysis result

	Before the sunrise		After the sunrise	
	Image	LiDAR	Image	LiDAR
Minimum absolute error(m)	None	0.00	0.00	0.00
Maximum absolute error(m)	None	1.29	0.78	1.10
Average absolute error(m)	None	0.07	0.04	0.09
RMSE	None	0.33	0.19	0.34

6. CONCLUSION AND FUTHER STUDY

This study has selected lanes and widths of the lanes as the object for recognition among the road factors which should be recognized necessarily by the unmanned vehicle and compared the results of the accuracy measured under various conditions. According to the analysis result, the after the sunrise, the averages of the widths of the lane using the image or LiDAR were 3.34m and 3.21m and the RMSE was 0.19 and 0.34 and thus it was shown that the measurement of width of the lane using the image was more favourable in the phase of the accuracy. However, before the sunrise, the width of the lane was not detected in the image and in the case of the measurement of the width of the lane using the LiDAR, the average was 3.23m and the RMSE was 0.33 and thus the limitation of the measurement of the width of the lane using the image was proposed.

The following study is needed for the development of this study. First, collection of a large amount of data is needed for various sections. More objective results should be approached by expanding the analysis scope and object section. Second, Improvement of the comparison, verification and recognition method of various sensors is needed. Accuracy of measurement should be able to be enhanced through this. Lastly, a study is needed in which technology which recognizes other street furniture other than the width of the lane which was the object of this study can be developed and analyzed.

It is expected that the result of this study can become a basic study which can help the unmanned vehicle to recognize the street furniture and drive.

7. ACKNOWLEDGEMENTS

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PAPER TITLE	The Needs of Life Cycle Cost Application for Malaysia Green Highway Projects		
TRACK	Project Delivery, Road Financing		
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KEYWORDS:

Life Cycle Costing; decision making application; green highway; sustainability

ABSTRACT:

In the journey towards preserving highway infrastructures in a sustainable manner, Performance-Based planning and asset management are important to efficiently preserve and rejuvenate at its life cycle stages. There are many methods and efforts to integrate the current practices, however, there are many gaps to be considered between the ideal theory and real-time measures. The highway components asset life-cycle cost will have an impact in terms of larger cost investment due to the fact that systematic assessment is rarely applied. The total cost of different highway projects should be evaluated based on the life cycle cost (LCC), which includes all expenses and incomes during the lifetime of the construction until post-construction. This paper focuses on the importance of the LCC application to highway project and it also responds to the sustainability of highway infrastructure development. This paper also utilized and reviewed related literatures concerning life cycle cost (LCC) application for the green highway in order to spark the significance of this research. Then, this paper also highlights the expected findings, which lead to identifying important factors in developing real-time decision making application using LCC. The outcome of this paper will inspire the Malaysia's highway builders to respond to green highway development and make LCC as a tool of decision making.

The Needs of Life Cycle Cost Application for Malaysia Green Highway Projects

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1 INTRODUCTION

The construction industry has a significant impact on the environment, economy, and society. Surface transportation systems are one of the biggest contributors to greenhouse gas emissions, for which they are responsible for 38% of all CO₂ emissions (Tatari, O. et al. 2011). Increased awareness of the enormous ecological footprint of the infrastructure environment has substantially increased the importance and popularity of various green highway initiatives as a possible solution to remediate the damages incurred on the planet.

Many of these initiatives focus on enhancing biodiversity, improving air and water quality, reducing solid waste generation, and conserving natural resources of buildings. Therefore, being green, or sustainable, is the pressing issue coming from both internal and external drivers for construction and engineering companies. To assess how green or sustainable the highway is, several green rating systems, protocols, guidelines and standards have been developed in the past that respond to the need to evaluate and benchmark levels of building achievement in the green revolution (Yudelson, J. 2008a), Yudelson, J. 2010b).

The first green highway manual was established in 1998 by the US Green Building Council (USGBC). Leadership in Energy and Environmental Design (LEED) adopted a checklist approach which emphasizes on sustainability development. In 2008, New York State Department of Transportation established their Green Leadership in Transportation Environmental Sustainability (GreenLITES) (New York State Department of Transportation 2010). It is a transportation specific checklist with 4 levels of certification. A year later, University of Washington produced Green road assessment model (University of Washington 2011). In 2010, three green highway assessment tools have been established. Recycled Materials Resource Center and University of Wisconsin-Madison, Portland (Oregon) Bureau of Transportation; Santa Cruz County Regional Transportation Commission and Illinois Department of Transportation have formed Building Environmentally and Economically Sustainable Transportation Infrastructure Highways (BE2ST in Highways), Sustainable Transportation Access Rating System (STARS) and Liveable and Sustainable Transportation Rating System and Guide (I-LAST) respectively (Lee, J. et al. 2010, The North American Sustainable Transportation Council 2010, Illinois Department of Transportation 2010). On 2012, Federal Highway Administration produced FHWA Infrastructure Voluntary Evaluation Sustainability Tool (INVEST), and Institute for Sustainable Infrastructure Ranking System/Harvard Zofnass Program for Sustainable Infrastructure have established Envision (U. S. DOT, Federal Highway Administration 2012, Harvard University Graduate School of Design & the Institute for Sustainable Infrastructure. 2012).

Today's evaluation of cost mainly focuses on the investment costs with only little regard to future costs. For highway users, the expense of using this infrastructure is a result of the accumulated costs during the highway's lifetime. Initiatives that reduce the future costs (e.g. Energy savings, improved durability of highway components) often result in larger investment costs, (e.g. the addition of thermal insulation, more durable pavement materials etc.). If future costs are not included in the evaluation, these initiatives will not be implemented. Therefore, the total cost of different highway designs should be evaluated based on the life cycle cost (LCC), which includes all expenses and incomes during the lifetime of the construction. In order to obtain total cost, the measureable variable in the stage of life cycle costing should be identified and correlation between them should be established first.

This paper will review the related literatures which may bring the sense of the importance of life cycle cost (LCC) application for green highway. Besides, the steps to identify important factors in developing real-time decision making application using LCC will also be highlighted in this research.

2 LITERATURE REVIEW

A green highway can be defined as a roadway design based on a relatively new concept for roadway design that integrates transportation functionality and ecological sustainability (UTM-LLM 2014). An environmental approach is used throughout the planning, design, and construction stages. This green highway will give benefit towards transportation, the ecosystem, urban growth, public health, and surrounding communities. Awareness of the highway users and stakeholders is essential in order to ensure the equilibrium between development and sustainability. When built to the standards of the concept, green highways have invaluable benefits to the environment. A green highway needs more consideration of all parties from top to down. The government as a policy maker, will provide the rules and regulation to be practised by the concession company and highway users (UTM-LLM 2014).

The need for promoting sustainability and green highway construction requires a green highway assessment system. Therefore, Malaysia Green Highway Index manual was designed as a performance baseline standard in order to measure the level of greenness for current highway in Malaysia. This manual intends to come out with several fundamental elements of green highway development that is suitable to Malaysia's condition. Five (5) specific activities which are Sustainable Design And Construction Activities (SDCA), Energy Efficiency (EE), Environmental And Water Management (EWM), Material And Technology (MT) and Social And Safety (SS) form the key features in the Malaysia Green Highway Index Manual that suitable to Malaysia condition with Platinum level are the highest among four (4) certification levels. In the other hand, many parties in highway industry will get the benefit and courtesy by developing the sustainable green roads, pathways, and expressways by following developed Green Highway certification (UTM-LLM 2014).

Life-Cycle Cost Analysis (LCCA) was legislatively defined in Section 303, Quality Improvement, of the National Highway System NHS Designation Act of 1995. The definition as modified by TEA-21, is "[it is] a process for evaluating the total economic worth of a usable project segment by analysing initial costs and discounted future cost, such as maintenance, user, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment." A usable project segment is defined as a portion of a highway that, when completed, could be opened to traffic independent of some larger overall project (Walls III, J. et al. 1998).

Similarly, in simpler terms, LCCA is an analysis technique that supports more informed and, it is hoped, better investment decisions. LCCA is an analysis technique that builds on the well-founded principles of economic analysis to evaluate the over-all-long-term economic efficiency between competing alternative investment options. It does not address equity issues. It incorporates initial and discounted future agency, user, and other relevant costs over the life of alternative investments. It attempts to identify the best value (the lowest long-term cost that satisfies the performance objective being sought) for investment expenditures. Life Cycle Cost (LCC) analysis should be conducted as early as possible in the project development cycle (Walls III, J. et al. 1998).

Cost and schedule are two of the most important project components in the construction industry. Major research efforts have focused on the improvement of the effectiveness of cost and schedule control, cost and schedule integration and cost-schedule trade-off in construction. A survey indicates that about 33% of architecture/engineering projects do not meet cost and schedule targets (Bayraktar, M. E. 2006). A case study indicates that costs increased 9.2% and schedules increased 23.3% on average for four environmental and engineering projects. A report indicates that the median cost increase for design projects is about 10% (Bayraktar, M. E. 2006). Bayraktar (2006) also finds out that project costs were underestimated in 90% of the 258 infrastructure projects completed in the past 70 years.

Gao et al. (2002) find the project cost as the highest ranked factor that determines the success of the construction projects (Gao Z. L. et al. 2002). Hastak et al. (1996) present a prototype decision support system called COMPASS (Cost Management Planning Support System) for project cost control strategy and planning. The proposed methodology helps management evaluate potential degree of cost escalation for the

project under consideration and take proactive measures by identifying attributes that might be the reason for cost escalation including management errors, regulatory approval, and rework (Hastak, M., et al. 1996).

Dunston and Reed (2000) present the effectiveness of the Small Projects Team Initiative (SPTI) developed and used by the Seattle district of the U.S. Army Corps of Engineers. The SPTI's objective is to lower the design costs on construction projects where the design scope is simple. The study statistically confirms that SPTI leads to reduced design costs and reduced schedule growth (Dunston & Reed 2000). Barraza et al. (2000) present probabilistic monitoring of project performance using stochastic S-curves as a potential method to develop a better project control system (Barraza, G.A. et al. 2000). Similarly, Chang (2001) identifies cost/schedule performance indices and their values for design projects (Chang, A. S. 2001).

3 METHODOLOGY

In order to identify important factors in developing real-time decision making application using LCC, the methodology of the research is as follows:

1. At the beginning, literature review had been conducted in order to gather the information about the green highway & life cycle costing from journal, books, conference paper, internet and etc. From the literature review, the factor of green highway, which is relevant to the life cycle costing were identified. The factors in concern were the measurable variable which is cost item.

2. Then is the discussion on the criteria of cost items and confirming the cost item with the expert. After the measurable variables in green highway had been identified, the research continued with the literature review on the correlation between variable and Life cycle costing. Based on the literature and the focus group method, a preliminary table which illustrates the item cost and each stage of life cycle costing had been developed. There are 2 stages of life cycle costing involved, which are the initial cost and future cost. For initial cost, there are capital; construction cost (installation); and management cost. While for future cost, there are operation; maintenance/service; replacement; demolition; contingencies cost/risk and management cost.

3. Then is the survey questionnaire development with Decision Matrix (see Table 1). The purpose of the survey questionnaire is to identify the relevant cost item which best suit in each stage of the Life cycle costing & the correlation between them. The questionnaire sheets consist of two (2) sections as follows:

4. Section A: Respondents' Profile. This is to know the background of the respondents. . It is important to know the experiences and individual preferences of those who are involved with the highway construction.

5. Section B: Selection of the correlation between green highway variables and life cycle cost (LCC). There are five categories in the Section B where each category has its own Decision cross-matrix between cost item and life cycle cost (LCC). Respondents were request to indicate No relevance=1, least relevance=2, moderately relevance=3, strongly relevance=4, very strongly relevance=5 in the appropriate blank box provided alongside each statement.

6. The questionnaires will be distributed to the 10 targeted respondents, which consists of experts in highway construction industry, especially those who are involved in the development of green highway and life cycle costing. The survey will be conducted using face-to-face interview with the respondents in order to help them to fill the questionnaire survey form based on their expertise.

7. The findings from the questionnaire survey will be analyzed by using IBM SPSS Statistics 22. Spearman correlation coefficient will be obtained in order to show the strength of correlation between Energy efficiency cost items and life cycle costing.

8. By referring to Dr Weir, L. from University of the West of England, Spearman's correlation coefficient is a statistical measure of the strength of a monotonic relationship between paired data (Weir, L. 2015). In a sample, it is denoted by and is by r_s design constrained as $-1 < r_s < +1$ and its interpretation is similar to that of Pearsons, e.g. the closer r_s is to ± 1 the stronger the monotonic relationship.

Correlation is an effect size and so we can verbally describe the strength of the correlation using the following guide for the absolute value of rs:

- .00-.19 “very weak”
- .20-.39 “weak”
- .40-.59 “moderate”
- .60-.79 “strong”
- .80-1.0 “very strong”

9. Spearman’s correlation coefficient which will be fell under “strong” and “very strong” category are the important factors in developing real-time decision making application using LCC in green highway.

4 RESULTS AND DISCUSSION

After going through the introduction and literature review, we noticed that many of the assessment tools in green highway do not consider life cycle costing significantly. For our case study in Malaysia, the assessment tool in green highway is Malaysia Green Highway Index. This index also does not consider life cycle costing in their manual.

Many studies have shown that project cost is the highest ranked factor that determines the success of the construction projects. Thus, in order to apply green element in highway development successfully, it is important and urgent to undergo the LCC in the assessment.

Besides, Table 1 has been proposed in order to identify the important factors of green highway which will be used to develop real- time decision making application using LCC. This table will be used in the design of questionnaire survey form in the phase of data collection.

Table 1. Sample of questionnaire

ID	Criteria	Life Cycle Costing Cost Item	Initial Cost			Future Cost				
			Capital Construction cost (Installation)	Management Cost	Operation Maintenance / Service	Replacement	Demolition	Contingency Cost / Risks	Management Cost	

5. CONCLUSION

From extensive literature review, the importance of the LCC application to highway project has been addressed. A sample of table of measurable variables (cost item) of energy efficiency in green highway index was formulated, as presented in Table 1. The format of the table will be used to identify the important factors (cost items) of green highway which have significant contribution towards the establishment of real-time decision making application using LCC. Finally, the results of the questionnaire will be analyzed and will lead to the addition of LCCA in the Green Highway Design Manual as shown in research framework (see Figure 1).

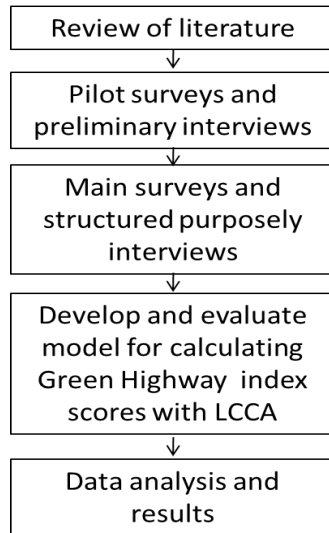


Figure 1. Research Framework

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PAPER TITLE (90 Characters Max)	APPLICATION OF MICRO-SURFACING SYSTEM – PLUS Experience along North South Expressway (NSE)		
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KEYWORDS: Micro-surfacing, Distresses, QA/QC

ABSTRACT:

Micro-surfacing is a polymer modified cold-applied paving system and it has been used as one of the preventive maintenance options to the pavement along the North South Expressway (NSE) since 1993. For the past 10 years, micro surfacing has been widely used by Highway Concessions in Malaysia and its use has increased significantly especially in southern part of the NSE.

Project selection in determining suitable candidate is critical as the main use of micro-surfacing materials is for pavement preservation before distresses appear. The main criteria for project selection are sound and well drained bases, surface and shoulder. Any localized defects must be repaired before application of micro-surfacing.

Findings from the micro-surfacing during construction indicated that the equipment, materials, application conditions, skill operators and safety as well as traffic control are the most important factors contributed to the success of the micro-surfacing work. A strict QA/QC is important to guide project personnel on the important aspects of micro-surfacing.

INTRODUCTION

Micro Surfacing is a mixture of asphalt emulsion, graded crushed mineral aggregate, mineral filler, water and other additives. The mixture is made and applied to existing pavement by specialized application machine that carry all necessary components, mixed them on site, and spread the mixture onto the road surface through an agitated spreader box. The typical mixing order is aggregate followed by cement, water, additive and the emulsion.

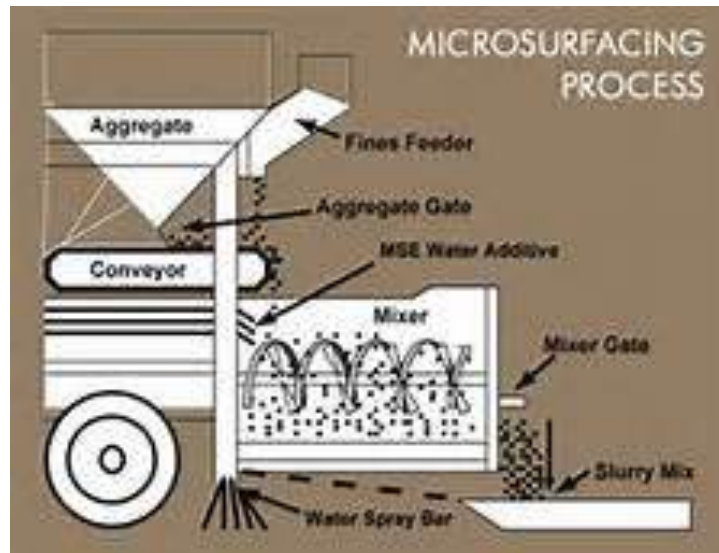


Figure 1. Schematic of a Micro-Surfacing machine

Micro surfacing is a thin surfacing and does not add any structural capacity to an existing pavement. Primary use of micro surfacing is to prevent future road surface distress in good pavement and as a corrective action in older and ageing pavement. Typically, it offers alternative option to prevent and/or reduce ageing/oxidation of asphalt, infiltration of surface runoff besides addressing surface raveling, weathering and loss of friction properties as well as provide better aesthetic view of overall completed section.

The completed micro surfacing works though proper quality control and correct selection of the materials will give the following results:-

- Increased stability at high temperature;
- Maximum, long lasting skid resistance; and
- Resistance against deformation, bleeding and raveling.

History

The use of a micro surfacing along the North South Expressway (NSE) was first introduced in year 1993 between KM23.00 and KM21.00 of the Senai Johor Bahru Highway. The two lanes dual carriageway was overlaid using micro surfacing system, operated by the local contractor i.e ACP-DMT Sdn Bhd, based on proven German technology.

Since completion of the first location, micro surfacing has been widely considered in the road maintenance option as it offers a cost-effective maintenance technique in addressing a broad range of common pavement surfacing problems. In addition, availability of well- trained operator equipped with special machine allow better implementation with minimal amount of inconvenience to the Engineer and road users.

Table 1 indicated the yearly consumption and coverage of micro surfacing system along the NSE for the past ten years. Increasing of the usage were recorded with the highest consumption was in year 2008 where more than six hundred thousand square meter areas were applied along the NSE.

Table 1. Yearly Consumption of a Micro-Surfacing from 2006 to 2015

Year	Quantity (sq.m)
2006	32,660.00
2007	58,730.00
2008	628,489.00
2009	599,876.50
2010	255,095.00
2011	Nil
2012	253,024.00
2013	238,318.00
2014	478,397.00
2015	442,893.00

Project Selection

Selection of project location is critical as the primary use of micro surfacing is for pavement preservation. Generally, pre-treatment such as pothole patching and localized mill and pave are required to address localized surface distress before application of micro-surfacing. Micro surfacing has been used along the mainline of the NSE on the slow lane, fast lane and emergency lane but not on the slip road and toll plaza areas.

Distress mode that can be addressed using micro surfacing include raveling, hairline cracks, polished asphalt/concrete surface, as well as low skid resistance. However, micro-surfacing may not be suitable to address rutting, cracking or any kind of pavement base failures.

Construction Process

The construction process shall include the following components:-

- Traffic Management and Safety
- Equipment
- Stockpile
- Surface preparation
- Application
- Quality issues
- Post construction

Traffic Management and Safety

Traffic management is necessary for both safety of road users and the workers performing the work. Traffic management control comprising construction signs, construction cones, and flagman should be in place before entry into work zone. Traffic management requirements shall comply with the Client's standard requirement.

Traffic management control is also required to ensure adequate time of micro surfacing to cure prior reopening to traffic. The curing time for micro surfacing will vary depending on the weather condition and pavement surface conditions at the time of application.

Equipment

All equipment, tools and machine used shall be maintained in satisfactory condition at all times to ensure a high quality product. The machine must have sufficient storage capacity for aggregate, emulsion, mineral filler, additive and water to maintain adequacy supply to the proportioning controls.

The mixture will be spread uniformly by means of a conventional augured surfacing spreader box attached to the mixer.

Stockpile

Stockpile should be as close as possible to the construction site to ensure operation run as smoothly as possible and provide good traffic flow through the construction area. In normal cases, area at the nearest toll plaza or layby will be selected as a stockpile area.

Surface preparation

The area to be covered with micro surfacing shall be thoroughly cleaned from loose material, vegetation, silt spot and other objectionable materials. Area with base failure must be rectified accordingly. Thermoplastic road markings must also be removed prior to placing a micro surfacing.

Application

Micro surfacing is produced and placed by a purpose-design machine, which carries all the material components. The machine will proportion these components on a continuous basis and applies it onto the road surface through an agitated spreader box. A sufficient amount of material shall be carried in all parts of the spreader box at all times so that a complete coverage is obtained.

Quality issues

Quality control is very important during construction to achieve a uniform surface finish and ensure successful of the work. The main areas of concerned are:-

1. Longitudinal Joints
2. Transverse Joints
3. Excessive binder
4. Dragging line
5. Damaged due to early opening to traffic and raining

The longitudinal joint should be straight or curve with the traffic lane and overlaps should not be in wheel paths. Construction of longitudinal joints along the acceleration and deceleration lanes approaching interchanges should be done carefully and must be butt joints. Overlaps should be kept to a minimum to control free flow of surface water. Figure 2a and 2b indicates good quality and poor quality of joints.



Figure 2a. Good quality longitudinal joint.



Figure 2b. Longitudinal joint on wheel path and excessive binder on joint.

Transverse joints are unavoidable as every time a truck is emptied a transverse joint is required. Transitions at these joint is critical and must be smooth to avoid creating bump in the surface. The main difficulty in obtaining a smooth joint occurs as the micro surfacing that is difficult to work by hand and breaks quickly. In some cases, excessive binder occurs at the joint as the contractor tend to over wet the mix. The operator shall be well trained and experienced in handling and getting used to the application machine. Figure 3a and 3b indicates good quality and poor quality transverse joints.



Figure 3a. Good quality transverse joint



Figure 3b. Poor quality transverse joint

Excessive binder may occur due to poorly designed micro surfacing mixture or mix with too high water content. This leads to a black and flush looking of completed surface with poor texture. Figure 4a and 4b indicates good and poor surface texture.



Figure 4a. Good surface texture



Figure 4b. Poor surface texture

Dragging line on the micro surfacing occurs due to presence of oversize aggregate or could be due to improper cleaning of spreader box. Therefore, spreader box must be cleaned properly before starting of works and all new delivery of aggregate must be tested to comply with the requirements. Figure 5 indicates dragging line on completed micro surfacing.



Figure 5. Dragging line on micro surfacing surface.

Heavy rain coupled with heavy traffic will likely lead to surface damaged especially on climbing location. However, light rain 2 or 3 hours after placing of micro surfacing seems acceptable. In some cases, sudden increase of traffic volume caused traffic congestion, thus require the lanes to reopen early and will also damage the newly laid micro surfacing. Figure 6 indicates raveling caused by rain and early opening to traffics.



Figure 6. Ravel off micro surfacing surface

Conclusion

Micro surfacing offers a cost effective technique in pavement maintenance options. It suitable to prevent and/or reduce ageing/oxidation of asphalt, infiltration of surface runoff besides addressing surface raveling, weathering and loss of friction properties.

Findings from the micro-surfacing during construction indicated that the equipment, materials, application conditions, skill operators and safety as well as traffic control are the most important factors contributed to the success of the micro-surfacing work.

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PAPER TITLE	Engineering Properties of PAC Thin Surfacing for Pavement Rehabilitation		
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KEYWORDS:

Porous asphalt concrete; Thin asphalt overlay

ABSTRACT:

Thin asphalt overlays have been used in mill-fill operations to surface badly cracked pavements simply because funding may not be available to add additional structure. With respect to the limitation of lift thickness and minimizing closures to traffic, the porous asphalt concrete (PAC) thin layer could be more durable and economical than dense-graded layers in terms of the engineering properties and functionality. However, concerns are raised regarding the effect of aggregate size on mechanical properties and performance of the PAC materials for thin overlays. The objective of this investigation is to have a better understanding of the use of PAC thin overlays for asphalt pavement rehabilitation. This will be accomplished by conducting a series of experimental works. Test results showed that the aged PAC mixtures were found to have the better performance in tensile strength and rutting resistance as compared to the unaged PACs, but the aging had the detrimental influence on the raveling resistance and moisture susceptibility. The long-term aging procedure led to a considerable reduction in tensile strength ratio for the PAC mixtures irrespective of binder type and gradation. In addition, the values of resilient modulus and tensile strength for the fine-graded PACs appeared to be higher than those for the coarser graded PAC mixture. Based on the results obtained from the wheel track testing, the use of premium asphalt enhanced the cohesive strength and adhesion, thereby reducing the proportional rut depth of PAC. In terms of the overall performance evaluation, the fine-graded PAC with the highly modified asphalt binder can be used without compromising engineering properties.

Engineering Properties of PAC Thin Surfacing for Pavement Rehabilitation

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1 INTRODUCTION

Asphalt thin layer has been utilized in mill-fill operations to resurface badly distressed pavements simply because funding may not be available to add additional structure. With respect to the limitation of lift thickness and minimizing closures to traffic, the porous asphalt concrete (PAC) thin layer could be a viable option for the high-speed surface layer rehabilitation. The use of PAC with fine gradation is desirable owing to the potential to be placed thinner and smoother layers. Although the PAC with a nominal maximum aggregate size (NMAS) of 19 mm has gained acceptance after a couple of research study and filed validation in Taiwan [Coleri et al., 2013; Moriyoshi et al., 2014; Putman and Kline, 2012], the engineering properties of finer gradation PAC are not know in great detail. In addition, the large portion of void contents would result in an increased potential for accelerating aging. Rapid oxidation of the asphalt binder films has an adverse impact on the cohesion and adhesion within the PAC mixtures [Mo et al., 2009; Mo et al., 2010; Molenaar et al., 2010].

In comparison with conventional dense graded asphalt concrete, the PAC differs from contains insufficient fines to fill all of the voids between the coarse aggregate particles. The air void content, ranging between 15 and 25 percent of the volume of an asphalt mixture, are designed to be interconnected, allowing the surface water drain vertically down to an impermeable layer. The water then travels laterally to the exposed edges of the PAC surface course. PAC also provides a highly skid resistance, reduces headlight glare and absorbs tire noise because of having a macrotexture on the pavement surface. The void content would affect the stone-on-stone contact structure and mechanical properties of PACs [Alvarez et al., 2010; Alvarez et al., 2011a; Alvarez et al., 2011b]. It was known that using device such as Corelock® to conduct the bulk density analysis. The dimensional analysis method was also recommended to compute the densities of the compacted mixtures and corresponding total air void content to obtain representative void content [Alvarez et al., 2009].

Due to the relatively lager amount of air voids, the PAC is designed to possess the stone-on-stone contact in order to offer a stable surface course structure. To quantify the stone-on-stone contact achieved in the mixtures, the maximum value of the voids in coarse aggregate (VCA) ratio of 0.9 was recommended to ensure adequate mixture durability. The VCA ratio compares the VCA of compacted mix (VCA_{mix}) to the VCA of the coarse aggregate fraction in the dry-rodded condition (VCA_{DRC}) to evaluate the existence of stone-on-stone contact [Alvarez et al., 2010b]. In addition to the issue regarding aggregate skeleton of porous mixtures, the Cantabro abrasion test was suggested to be useful for selecting material combinations and determining the optimum asphalt content [Alvarez et al., 2010c]. The proper asphalt grade and asphalt content are also selected to have desired sufficient film thickness of asphalt binder in order to provide good durability.

A number of studies related to PAC materials have been made over the past few years. Generally, finer mixtures had higher indirect tensile strength and better moisture resistance because the aggregate particles packed more closely together, while the coarser gradation had lower rut depth [Xiao et al., 2015; Mansour and Putman 2013]. The aggregate gradation also had significant effect on the porosity of the aggregate structure, which in turn controlled the macro texture and permeability of the porous asphalt mixtures [Martin et al., 2014]. Additionally, there had a few studies regarding the re-use of reclaimed materials added in virgin porous asphalt mixtures. Basically, these results showed that the mechanical properties and performance of the recycled porous asphalt mixtures were similar or better compared with the virgin open-graded asphalt mixtures in some cases [Chen and Wong 2015; Frigio et al., 2013; Frigio et al., 2015; Goh and You 2012].

Most research into porous asphalt mixtures has focused attention on mix design and mechanical properties. Little research is conducted on characterizing the aging on the engineering properties of porous asphalt concrete materials over a period of time. The primary objective of this investigation was to:

- characterize the stiffness of PAC,
- determine the resistance to ravelling of PAC,
- assess the resistance to rutting of PACs, and
- evaluate the moisture susceptibility of PAC.

2 MATERIALS AND METHODS

2.1 Materials

The experimental design included the use of three types of asphalt binders as shown in Table 1. The unmodified binder (UA) had a penetration value of 48. The two polymer modified binders were prepared using the proper content of SBS to achieve the desired viscosity requirements in accordance with the Public Construction Commission (2010). The SBS was slowly added into the heated base asphalt at 170°C and blended with a high-shear asphalt mixer at 3000 rpm for 60 minutes. It is noted that the modified asphalt (MA) contained 5% SBS polymer of 60/70 penetration grade asphalt (base asphalt), while the highly modified asphalt (HA) contained 12% SBS polymer of the same base asphalt. The viscosity values of each asphalt binder at unaged and aged conditions were measured using a Brookfield viscometer. The aging procedure was conducted on the asphalt binders by means of the thin film oven test (TFOT) over the aging periods.

One source of limestone aggregate and ordinary Portland cement were used for the aggregate and mineral filler, respectively. The basic properties of the limestone aggregate are tabulated in Table 2.

Table 1. Properties of asphalt binders

Properties	Asphalt Binders		
	UA	MA	HA
Penetration @ 25°C (d _{mm})	48	55	50
Viscosity @ 60°C (fresh) (Pa·s)	368	1,410	48,000
Viscosity @ 60°C (aged for 4 Hrs) (Pa·s)	568	2,055	59,500
Viscosity @ 60°C (aged for 48 Hrs) (Pa·s)	917	2,855	83,500
Viscosity @ 60°C (aged for 96 Hrs) (Pa·s)	1,034	3,320	100,000
Modification	Unmodified	Modified	Highly modified

Table 2. Properties of aggregate

Property	Coarse aggregate	Fine aggregate
LA abrasion value (%)	19.7	-
Flat and elongated 1:3 (%)	6.4	-
Flat and elongated 1:5 (%)	2.1	-
Absorption (%)	1.0	1.5
Soundness (%)	5.7	3.1
One broken surface (%)	100	-
Two broken surfaces (%)	97.3	-
Sand equivalent value (%)	-	81

2.2 Mix Design

The mixture design was performed in accordance with the Public Construction Commission (2010) for the two distinct gradations. The coarse aggregate gradation had a NMAS of 19 mm, while the NMAS of 12.5 mm was selected for the fine aggregate gradation. Both coarse and fine aggregate gradations had the same break-point (BP) sieve of 2.36 mm. A target air void content of 20% was selected for manufacturing the PAC specimens. The two types of gradations are shown in Table 3.

Table 3. Design gradation and specification

Sieve Size (mm)	19 mm Gradation (%)	19 mm Spec. (%)	12.5 mm Gradation (%)	12.5 mm Spec. (%)
25	100	100	100	100
19	95	95-100	100	100
12.5	64	64-84	95	90-100
9.5	38	-	59	-
4.75	18	10-31	23	11-35
2.36	10	10-20	14	10-20
1.18	8	-	13	-
0.600	7	-	11	-
0.300	6	-	8	-
0.150	5	-	5	-
0.075	4	3-7	3	3-7

The job mix formula was determined by conducting traditional Marshall mixture design method. The allowable asphalt content range was determined according to the results of the Cantabro abrasion and asphalt draindown tests. The various properties of mixtures at the optimum asphalt contents complied with the specification. The PAC specimens were then fabricated at optimum asphalt content for further mechanical testing as required. Table 4 shows the designations and various properties of the PAC mixtures.

Table 4. Results of PAC mixture design

Mix Designation	19/UA	12.5/UA	12.5/MA	12.5/HA
NMAS (%)	19	12.5	12.5	12.5
Stability (kgf)	508	658	651	741
Flow (mm)	2.5	3.5	3.3	3.8
Cantabro weight loss (%)	19.1	16.1	11.9	8.1
Draindown (%)	0.008	0.037	0.020	0.016
Optimum asphalt content (%)	5.1	5.0	5.0	5.0
Void content	20	18	18.5	19

2.3 Preparation of PAC Mixtures

For the preparation of the PAC, the aggregate mix was heated and then the asphalt binder was added to the mix to provide workability. To ensure that the aggregate was properly coated, the determination of mixing temperature was dependent on the viscosity of the asphalt binders. The mixing temperatures were selected to be 152°C, 158°C and 166°C for the PAC with UA, MA and HA binders, respectively. The mixing time was 60 seconds.

In order to simulate the mixing process in the plant and field construction effects on the PAC, the 4-hour aging procedure was performed on loose mixture in accordance with AASHTO R30. The loose mixture was placed to an even thickness ranging between 25 and 50 mm in a pan, followed by being conditioned in a forced-draft oven for 4 hours at 135°C. The loose mixture after the short-term condition was then removed from the oven for compaction.

A compaction effort of 50 blows was applied on each side at compaction temperatures for the PACs. The compaction temperatures, depending on the viscosity of the asphalt binders, were selected to be 148°C, 154°C and 163°C for the PAC with UA, MA and HA binders, respectively.

After the compaction effort, the specimens were allowed to cool at room for 24 hours prior to being extruded from molds. After extruding the specimens from the compaction molds, the compacted specimens were allowed to cool at room temperature for 16 hours before being subjected to the long-term aging process. The long-term aging procedure was then applied to compacted PAC that had been subjected to the short-term conditioning according to AASHTO R30. A compacted mixture of aggregate and asphalt binder was conditioned at 85°C in a forced-draft oven for 48 or 96 hours in order for simulating the aging that occurred over the service life of a PAC pavement.

2.4 Test Methods

2.4.1 Resilient Modulus Test

The resilient modulus (M_R) test was conducted at 25°C to evaluate the stiffness modulus of the PAC mixtures in accordance with ASTM D7369. The load was applied for 0.1 seconds along with a rest period of 0.9 seconds. The Poisson's ratio was assumed to be 0.35. The equation derived to calculate the M_R is listed as:

$$M_R = \frac{P}{Ht} (0.27 + \mu) \quad (1)$$

where M_R is the resilient modulus (MPa), P is the repeated load (N), H is the horizontal deformation (mm), t is the specimen thickness (mm) and μ is the Poisson's ratio.

2.4.2 Cantabro Abrasion Test

The Cantabro abrasion test was performed at 25°C to evaluate the resistance to raveling of the PAC mixtures. The compacted PAC specimen was placed in a Los Angeles machine without the charge of steel balls. The Los Angeles machine was operated for 300 revolutions at 30 rpm. The percentage of weight loss during this abrasion process was used to represent the raveling resistance of the PACs. The percentage of weight loss is shown as:

$$\text{mass loss (\%)} = \frac{\text{initial specimen weight (g)} - \text{final specimen weight (g)}}{\text{initial specimen weight (g)}} \times 100 \quad (2)$$

2.4.3 Wheel Tracking Test

The wheel tracking test was conducted at 60°C in a temperature controlled chamber to evaluate the resistance to permanent deformation of the PACs. The compacted PAC slab, housed in a 300 by 300 by 50-mm deep steel mold, was trafficked by a solid steel wheel at a frequency of 42 passes per minute under a loading pressure of 700 kPa for 1 hour. The vertical deformation induced at the middle of the specimen was recorded. In accordance with BS EN 12697-22, the deformation parameter of proportional rut depth was used to characterize the rutting resistance of the PACs, as shown in the following equation:

$$\text{Proportional Rut Depth (\%)} = \frac{\text{Top surface settlement (mm)}}{\text{Total thickness asphalt mixture slab (mm)}} \times 100 \quad (3)$$

2.4.4 Indirect Tensile Test

The indirect tensile (IDT) test was used to evaluate the strength of the PACs in accordance with ASTM D6931 [30]. The test was conducted on the PAC specimen at a deformation rate of 50.8 mm/minute and a temperature of 25°C. The equation for the indirect tensile strength at failure is provided as:

$$S_t = \frac{2000P}{\pi t D} \quad (4)$$

where S_t is tensile strength (kPa), p is maximum load (N), t is thickness of the specimen (mm) and D is diameter of the specimen (mm).

To evaluate moisture susceptibility of PACs, the tensile strength was measured before and after water conditioning of specimens. The tensile strength ratio was defined as the retained tensile strength as a percent of the original tensile strength.

3 RESULTS AND DISCUSSION

3.1 Stiffness of PAC Mixtures

The test results of the resilient modulus testing for the PAC mixtures are given in Figure 1. The M_R values showed an increase as the aging period increases, reflecting that given repeated loads resulted in lower strain for the aged PACs. For the fine PAC mixtures (NMAS 12.5 mm), the deviations are a result of different asphalt binders. The 12.5/HA had the higher M_R . The 12.5/HA can carry and transfer loads efficiently compared to the other PACs. The 12.5/MA and 12.5/UA showed a little difference in stiffness modulus although the viscosity of the modified asphalt was higher than that of the unmodified asphalt. The fact can be interpreted that the highly modified asphalt binder can increase the M_R of the PAC. With respect to the aggregate size on the stiffness modulus of PACs, the M_R value for the

12.5/UA appeared to be slightly higher than that for the 19/UA. The finer aggregate gradation was more cohesive due to slightly lower air void, as compared with the coarser aggregate gradation. M_R is generally a function of the asphalt binder properties in an asphalt mixture and this value can be increase with a stiffer binder. A higher resilient modulus indicates that a given applied load results in lower horizontal strain in the mix.

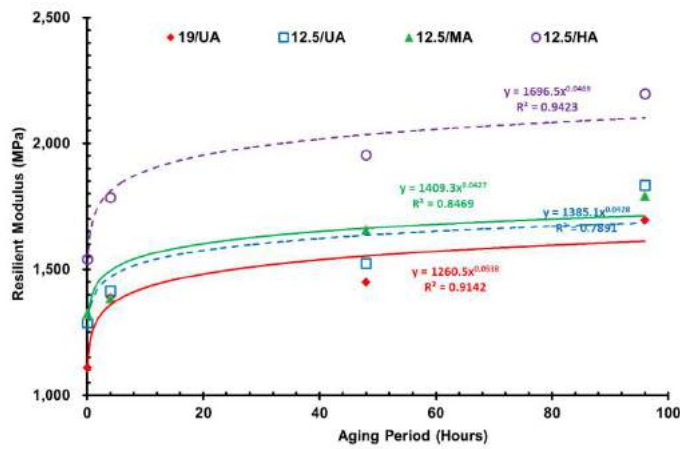


Figure 1. Resilient modulus for PAC mixtures with increasing aging periods.

3.2 Resistance to Reveling of PAC Mixtures

The test results of Cantabro abrasion testing for the PAC mixtures are shown in Figure 2. The results indicate that abrasion resistance was influenced by the gradation. The 19/UA had higher abrasion loss compared to the other fine-graded mixes, reflecting the coarser gradation had the susceptibility to raveling. An increase in abrasion loss in the coarser gradation when compared to the finer gradation was caused by the reduction in cohesion and adhesion within the coarser mixture. In addition, the Cantabro loss results showed that the percentage of weight loss increased with increasing the aging period. The aged PACs showed worse performance in resisting disintegration, indicating the weaker bonding between aggregate particles. The server disintegration for the long-term aged PAC specimens was caused by the oxidation of the asphalt binder. The movements of aggregate particles, caused by the abrasive action, developed sufficient tensile stress and strain which exceeded the cohesive strength of the mixes. The 12.5/HA had the least weight loss, followed by the 12.5/MA and 12.5/UA. The use of highly modified asphalt binder resulted in a reduction in percentage of the loss weight. It can be inferred that the asphalt binder with higher viscosity appears to contribute to the cohesion and enhance the resistance to raveling for the PACs.

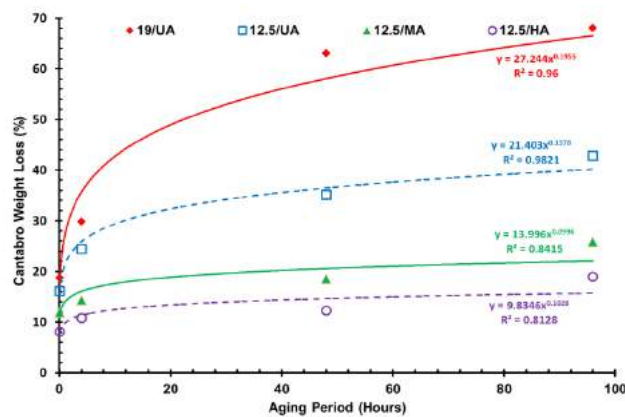


Figure 2. Cantabro abrasion resistance for PAC mixtures with increasing aging periods.

3.3 Resistance to Rutting of PAC Mixtures

The results of rut depth obtained from wheel track testing for each PAC mix are exhibited in Figure 3. The 19/UA had the smaller proportional rut depth as compared with the 12.5/UA. It is interesting to note that the 2-day and 4-day aging PAC mixtures for the 19/UA had the better performance in rutting resistance as compared to those for the 12.5/MA. It can be inferred that the use of coarse aggregate can improve the deformation resistance for the PAC if the asphalt viscosity was not high. When the highly modified asphalt binder was used, the proportional rut depth for the finer graded PAC was smaller than that for the coarser graded PAC. It can be interpreted that the finer graded PAC with highly modified asphalt has the potential of better resistance to rutting as compared to the coarser graded PAC with the

unmodified asphalt. In addition, the proportional rut depth for the PAC mixtures reduced with increasing aging period. The best performance in rutting was observed for the 4-day aging PAC mixes. For the finer gradation, the 12.5/HA was resistant to the permanent deformation, followed by the 12.5/MA and 12.5/UA. It reflected the asphalt binder with higher viscosity can improve the rutting resistance. Although the aggregate interlocking for the PAC is designed to have capacity to resist rutting, the binder viscosity can contribute to the cohesion, thereby enhancing resistance to deformation.

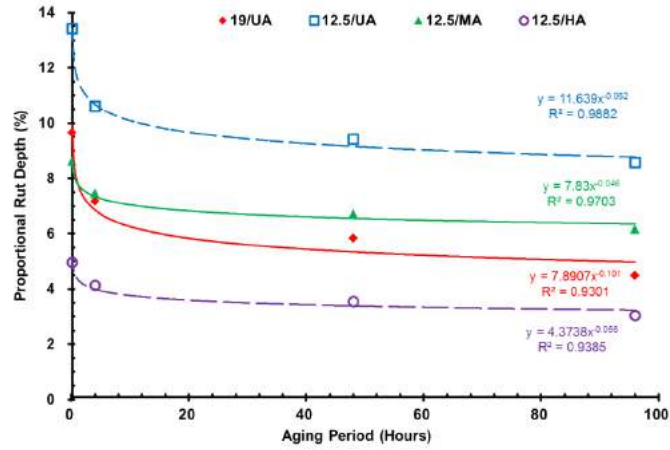


Figure 3. Rutting resistance for PAC mixtures with increasing aging periods.

3.4 Resistance to Moisture Susceptibility of PAC Mixtures

The test results of the indirect tensile strength for each PAC mix are presented in Figure 4. The 12.5/UA exhibited the better performance in strength as compared to the 19/UA. It demonstrated the cohesion for the finer graded PAC is higher than that for the coarser graded PAC. It also can be interpreted that the fine-graded PAC mix had higher tensile strength because its particles packed more closely together than the coarse-graded PAC mix. The coarser gradation was more susceptible to breaking. In addition, there was an increase in indirect tensile strength with increasing the aging period. It indicated that the indirect tensile strength was influenced by aging of the mix. It also inferred that the rapid oxidation of the asphalt binder film significantly increased the indirect tensile strength of the compacted PAC mixtures. The long-term aging results in an increase in the tensile strength for the PACs. Among the NMAS 12.5 mm PAC mixes, the 12.5/HA was found to have the best performance in the indirect tensile strength test. It is noted that due to the percentage of air void in PAC being much higher than in dense graded asphalt concrete, the PAC generally has less support from surrounding particles and thus has relatively lower strength compared to the dense graded asphalt mixture. Using the highly modified binder appears to enhance the adhesion between aggregate particles within the PAC mixes, thereby improving the indirect tensile strength values.

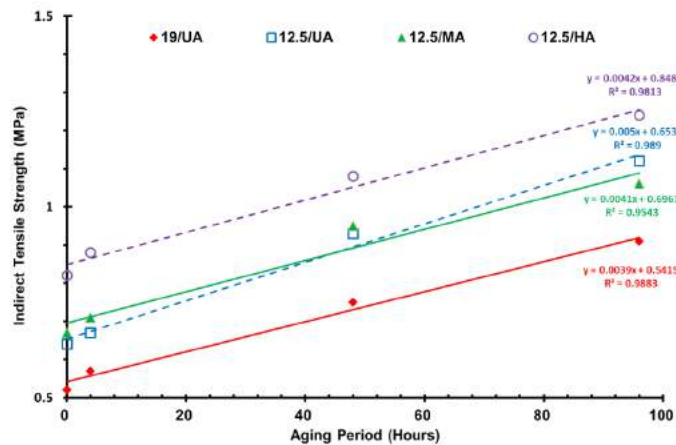


Figure 4. Indirect tensile strength for PAC mixtures with increasing aging periods.

The moisture susceptibility for the PAC mixtures in terms of the tensile strength ratio (TSR) is depicted in Figure 5. In regard to the aggregate size effect on the moisture-induced damage, it is found that a little difference in TSR between 19/UA and 12.5/UA. The finer aggregate gradation was shown to improve moisture susceptibility of PAC mixture. In addition, it can be seen that the TSR value decreased with increasing the aging period, reflecting the aged PAC mixture was susceptible to moisture-induced damage. The 12.5/HA had the highest TSR value among the

mixtures. This could be attributed to the use of highly modified asphalt binder, improving the resistance to moisture sensitivity. The long-term aged PAC mixtures were found to have a reduction in TSR value. This is due to the open nature of PAC mix which caused the rapid oxidation of asphalt binder together with the water retention in the voids.

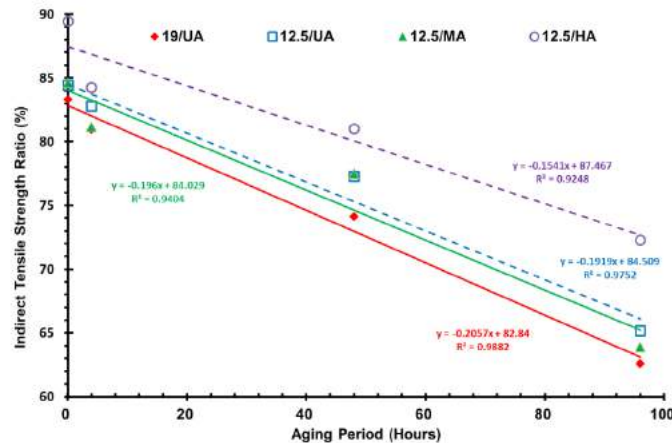


Figure 5. Tensile strength ratio for PAC mixtures with increasing aging periods.

4 CONCLUSIONS

This paper characterizes the influence of aging on the stiffness modulus, resistance to raveling, rutting resistance and moisture susceptibility of the PACs. It should be noted that the results of the laboratory tests described in this paper are confined to the material combinations, test procedures and equipment used in this study. The following conclusions based on test results can be drawn:

- The values of resilient modulus for the finer graded PAC appeared to be higher than that for the coarse graded PAC. The PAC with highly modified asphalt binder can increase the M_R of the PAC, thereby carrying and transferring loads efficiently.
- The coarse-graded PAC mixtures reflected the susceptibility to raveling. The use of highly modified asphalt binder appeared to contribute the cohesion and adhesion, thereby enhancing the raveling resistance for the PACs.
- The long-term aged PAC mixtures were found to have a considerably reduction in TSR value for the four types of the mixes. The fine-graded PACs was found to have better resistance to moisture-induced damage.
- The wheel tracking test results indicated that the finer gradation with the highly modified asphalt binder can improve the resistance to rutting for the PACs.
- In terms of the overall laboratory evaluation, the finer gradation PAC with the highly modified asphalt is a viable option without compromising engineering properties

5 ACKNOWLEDGEMENTS

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October 16-20, 2016 – Kuala Lumpur, Malaysia

PAPER TITLE	STUDY ON CHANGES OF RISK AWARENESS, SAFTY BEHAVIOR AND POSITION IN THE WALK WAY, THROUGH THE ROAD SAFETY EDUCATION FOR CHILDREN OF PRIMARY SCHOOL AND JUNIOR HIGH SCHOOL		
TRACK	ROAD SAFETY		
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KEYWORDS:

school zone
road safety education
walkway
bicycle user
questionnaire survey

ABSTRACT:

The purpose of this paper was to provide a road safety education to primary school and junior high school students. Through questionnaire surveys implemented before and after lectures as well as actual measurement, the study aimed to grasp the changes in students' risk awareness, behavioral intention and passing position, and encourage them to observe proper traffic behavior. The research area was a regional city in Japan, a primary school district in the suburban residential area of Shunan City. The primary school was Katsuma Primary School and the number of students was. Most of these students subsequently attend one junior high school, Kumage Junior High School.

From the results of research, following were clarified:

The higher grade students had higher risk awareness of the route to and from school. However, it is often observed that when they proceeded to junior high school, their risk awareness fell.

The lecture on the seven road safety rules and the five rules for safe bicycle use provided to students of primary school and junior high school could make their traffic behavior safer.

Their risk awareness maintained the same level into the next grade year.

To conclude, it was confirmed that in order to encourage compliance with the rules of the road, the implementation of road safety education to both primary school and junior high school is beneficial.

Study on Changes in Risk Awareness, Safety Behavior and Position in the Walkway, through the Road Safety Education for Children of Primary School and Junior High School in Shunan City, JAPAN

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1 INTRODUCTION

In April 2012, a traffic accident occurred on a prefectural road of Kameoka City, Kyoto where a light car crashed into rows of students going to primary school together with parents leading them. Ten of them were hit by the car; three died and seven sustained minor and serious injuries. Since this accident, accidents on the routes to and from school have become recognized as a social problem. Therefore security measures on the route to and from school have been advanced on a national level.

As the first step, in August 2012, safety inspections of the routes to and from school were implemented nationwide under the coordination of those concerned with schools, traffic police and road administrators. In July 2013, the “School Route Safety Measure Advisory” system was institutionalized as a project of the Ministry of Education, Culture, Sports, Science and Technology, and began its activities by deploying traffic specialists nationwide.

Since July 2013, the author has been appointed a school route safety measure advisor by the Department of Education of Yamaguchi Prefecture. Since then, the author engaged in instructing proper traffic rules to the students of primary schools and junior high school in accordance with road safety education as well as improving traffic space in cooperation with road administrators. It is common in Japan that primary school students go to school within walking distance. Therefore, as a result, they have to share the same traffic space on the road with other pedestrians, bicycles and cars. A greater percentage of junior high school students tend to commute by bicycle because of a greater commuting distance from home.

In this research, their changes of risk awareness and behavioral intention on the traffic space for primary school students on foot and junior high school students on bicycles while commuting to and from school were discussed. In particular, this research focused on accidental contracts by pedestrians and bicycles on the routes to and from schools.

The purpose of this paper was to provide a road safety education to primary school and junior high school students. Through questionnaire surveys implemented before and after lectures as well as actual measurement, the study aimed to grasp the changes in students’ risk awareness, behavioral intention and passing position, and encourage them to observe proper traffic behavior. The research area was a regional city in Japan, a primary school district in the suburban residential area of Shunan City. The primary school was Katsuma Primary School and the number of students was. Most of these students subsequently attend one junior high school, Kumage Junior High School (Figure 3).



Figure 1. Location of Yamaguchi Prefecture

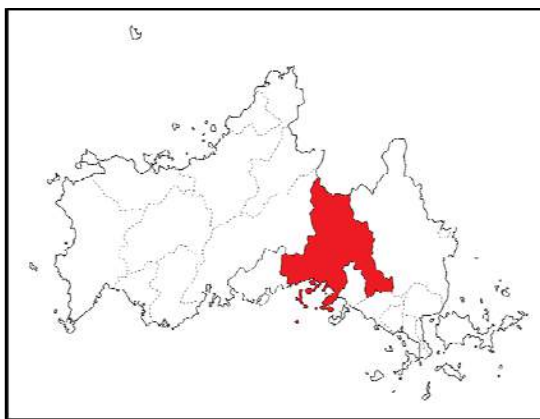


Figure 2. Location of Sunan City in Yamaguchi Pref.

The purpose of this section was to provide a road safety education for both Katsuma Primary School and Kumage Junior High School sharing the same route to and from school, grasp the changes in students' awareness and behavior, and obtain the knowledge necessary to encourage them to observe proper traffic behavior. (Figure 3).

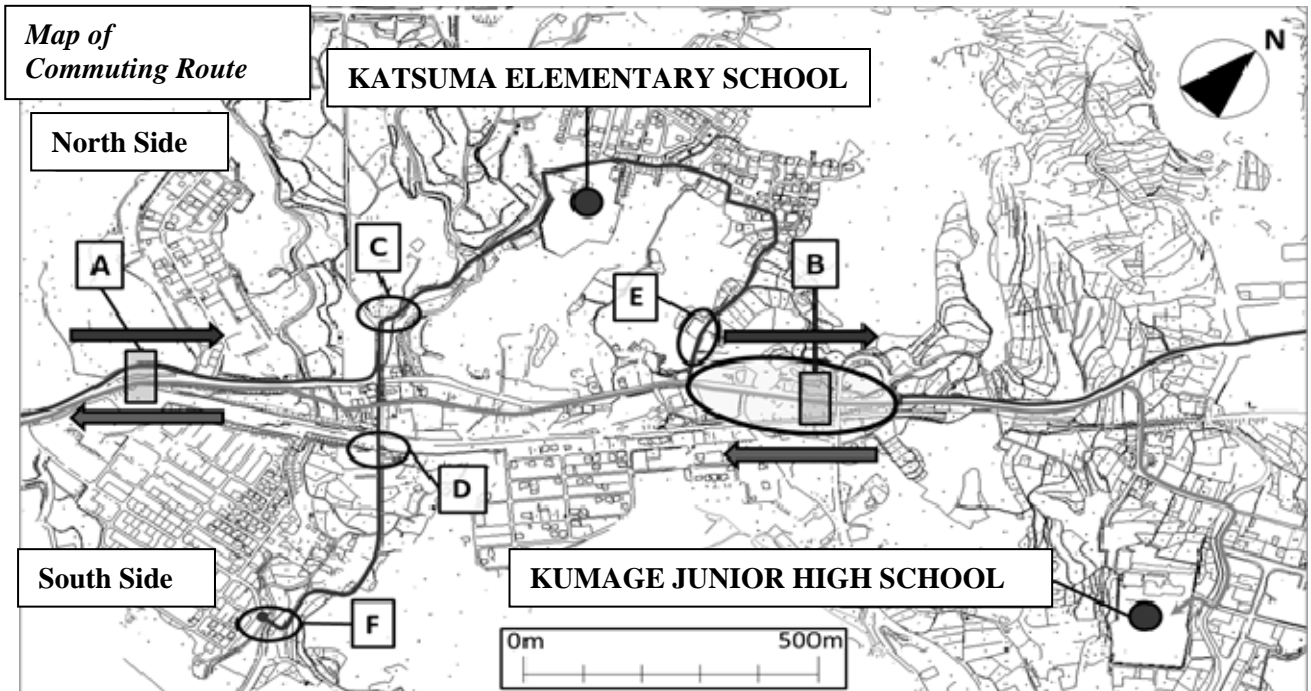


Figure 3. Location of Katsuma Primary School and Kumage Junior High School .

2 PURPOSE OF LECTURE FOR ROAD SAFETY AND MEASURING METHOD OF EFFECTS OF LECTURE

The purpose of this research was to encourage primary school and junior high school students to comply with traffic regulations and good manners while commuting to and from school through road safety education, and improve their traffic behavior.

By grasping the changes of awareness and behaviors concerning road safety through questionnaire surveys, and by observing changes in children's passing position through observation survey, the effects of the road safety education were discussed.

As for the method of research, road safety education was provided to both a primary school and junior high school in the Katsuma area of Shunan City. The road safety education focused on the improvement of traffic spaces shared by both the primary school and junior high school. The structure of the survey is shown in Figure 4.

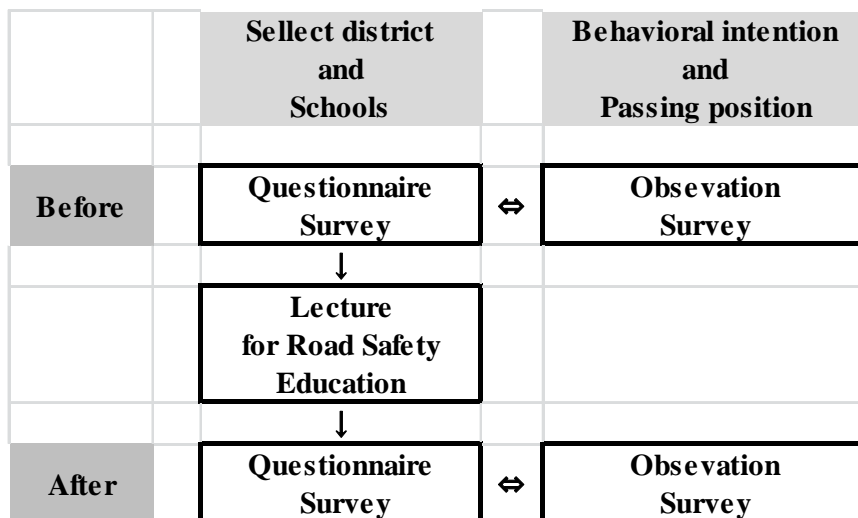


Figure 4. Structure of the Survey .

3 ABBREVIATIONS AND UNITS

3.1 OBSERVATION SURVEY

The number of pedestrians and bicycles passing through an observation site installed during the commuting hours for students of primary school and junior high school were counted by a counter for each direction.

At the same time, in order to confirm the passing positions of pedestrians and bicycles, a video shoot of the walkway was conducted. The changes in passing positions before and after the lecture were examined, and the impact of the road safety education on student behavior was discussed.

- i Outline of observation at Katsuma Primary School district (implementation date: May 22)
An observation was conducted at two points from 07:00 to 08:00: spot A (Photograph 1) on a walkway of National Route 2 and spot B (Photograph 2) on a side strip of a city road (Figure 5).
- ii Outline of observation at Kumage Junior High School district (implementation date: June 24)
An observation was conducted at spot C (Photograph 3) from 07:00 to 08:00, as shown Figure 5

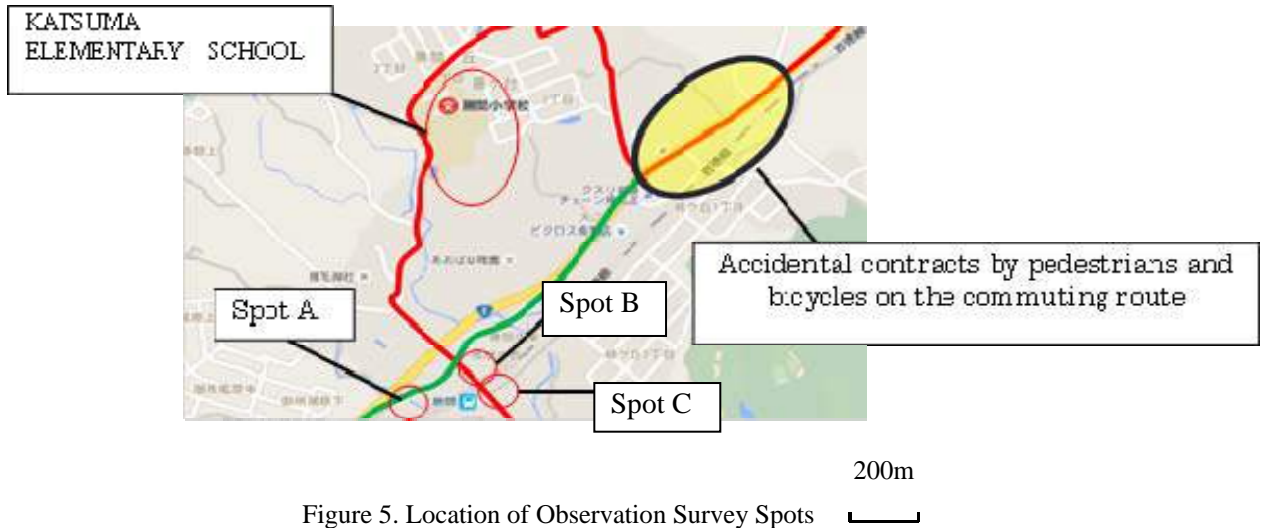


Figure 5. Location of Observation Survey Spots



Photograph 1. Snap Photo of Spot A



Photograph 2. Snap Photo of Spot B



Photograph 3. Snap Photo of Spot C

3.2 QUESTIONNAIRE SURVEY

The pre-questionnaire was conducted two weeks before the lecture was given and the results were aggregated, organized and reflected on the contents of the lecture.

There is the correspondence relationship in the question items between pre- and post-questionnaire.

As the question items were the same as the last two years, the changes in students' awareness in accordance with school year progress were confirmed.

3.3 DATE OF LECTURE ON ROAD SAFETY

- i June 2, 2015, Shunan City, Katsuma Primary School, 281 students of the third, fourth, fifth and sixth grades
- ii June 26, 2014, Shunan City, Kumage Junior High School, all 382 students

4 CHANGES OF AWARENESS AND BEHAVIOR OBSERVED IN THE STUDENTS OF KATSUMA PRIMARY SCHOOL

4.1 CHANGES OF AWARENESS AND BEHAVIOR (from 2013 to 2015)

That reduces the recognize who the safety is, that is, it means that the person who was aware of the danger has increased. The percentage of the fifth grade students in 2013 who "didn't think" the route to and from school was safe (or were aware of danger) rose to 33% after the lecture.

The percentage of the sixth grade students who were aware of danger in 2014 had remained high at 36% before the lecture and it rose to 46% after the lecture. However, that of the students of Kumage Junior High School in 2015 from Katsuma Primary School lowered to 17% before the lecture (please see Table 2).

It is considered to be due in part to the students' awareness being reset when their commuting means changed from foot to bicycle.

4.2 CHANGE IN ACCORDANCE TO SCHOOL YEAR PROGRESS (from 2013 to 2015)

The fifth grade students in 2013 were good at maintaining risk awareness for three years. However, the sixth grade students' risk awareness lowered after the lecture compared to before the lecture. The first grade junior high school students retained low risk awareness at 13% before the lecture and 20% after the lecture (please see Table 3).

However, the second grade junior high school students kept 19% before the lecture and it can be confirmed that their risk awareness would be maintained and promoted by repeating the lecture.

Table 2. Question: Do you think about the route to and from school was safe.

	5th grade KATSUMA P.S. (2013)		6th grade KATSUMA P.S. (2014)		1st grade KUMAGE J.H.S (2015)	
	before	after	before	after	before	after
think	65%	46%	43%	38%	48%	36%
N/A	18%	21%	21%	16%	35%	27%
didn't think	17%	33%	36%	46%	17%	35%

	6th grade KATSUMA P.S. (2013)		1st grade KUMAGE J.H.S (2014)		2nd grade KUMAGE J.H.S (2015)	
	before	after	before	after	before	after
think	64%	60%	60%	46%	47%	37%
N/A	19%	28%	27%	33%	34%	40%
didn't think	16%	12%	13%	20%	19%	23%

4.3 COMPLIANCE OF SEVEN ROAD SAFETY RULES

The three items of "Keep to the left when walking a narrow road," "Do not cross a broad road where there is no pedestrian crossing" and "Walk on the opposite side of the walkway from the road" were only observed at a rate of around 50-60%. However, after the lecture, it increased up to just below 80% (please see Figure 6).

Some primary school students were not conscious of passing position and it was found that their awareness of it improved after the lecture.

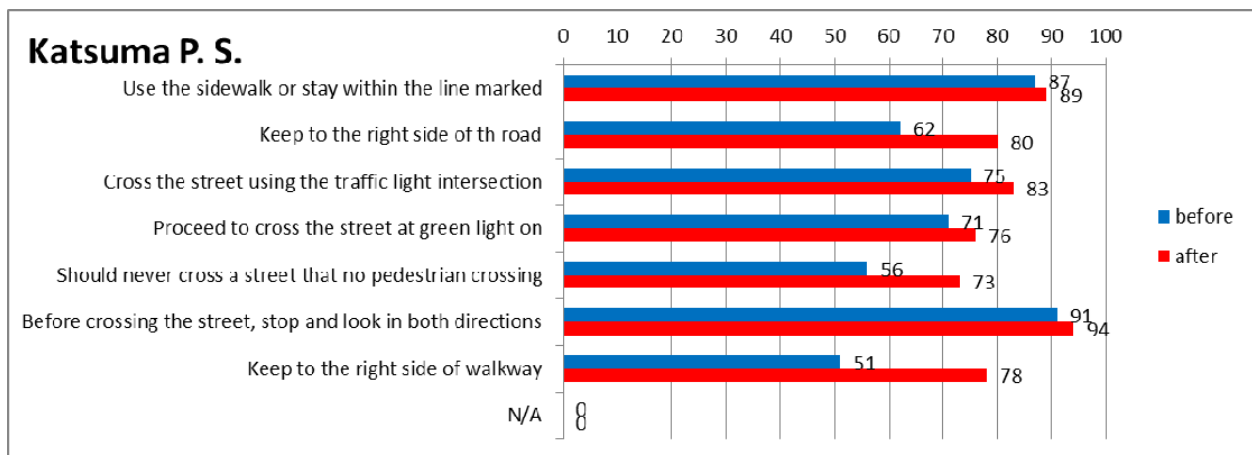


Figure 6. Compliance of Seven Road Safety Rules, Katsuma Primary School (Katsuma P.S.)

4.4 BEHAVIORAL CHANGE AT SPOT A

A behavioral change regarding “Walk on the opposite side of the walkway from the road” of the seven rules was recognized. In accordance with the observations before and after the lecture, those who walked on the opposite side of the walkway from the road rather than its center increased. However, this has not been quantified yet.

4.5 BEHAVIORAL CHANGE AT SPOT B

The behavioral change regarding “walk on the inner side of the side strip relative to the roadway in roads without walkways” was recognized. The percentage of primary school students who walked on the side strip’s outer side decreased from 20% before the lecture to 14% after the lecture.

Because the students commute to and from Katsuma Primary School in groups, the scenes where upper grade students guide the lower grades not to stray outside side strips are often observed.

5 CHANGE IN AWARENESS SEEN IN KUMAGE JUNIOR HIGH SCHOOL STUDENTS

5.1 RISK AWARENESS WHEN COMMUTING TO SCHOOL

No remarkable change was observed in risk awareness of the first grade junior high school students in 2014, shown as 15% before the lecture and 19% after the lecture. When they proceeded to the second grade, it increased a little from 18% before the lecture to 21% after the lecture; however, the change was not remarkable (Table 3).

5.2 CHANGE IN ACCORDANCE TO SCHOOL YEAR PROGRESS (from 2014 to 2015)

A great change was observed in risk awareness of the 2nd grade students in 2014 from 19% before the lecture to 33% after the lecture.

After they proceeded to the 3rd grade, the percentage before the lecture decreased to 26% and its percentage after the lecture was remained at 32%, the level of the previous year. The percentage of risk awareness was highest in the third grade students among all three grades (Table 3).

Table 3. Question: Do you think about the route to and from school was safe.

	1st grade KUMAGE J.H.S (2014)		2nd grade KUMAGE J.H.S (2015)	
	before	after	before	after
answer				
think	60%	52%	48%	38%
N/A	25%	28%	34%	40%
did`nt think	15%	19%	18%	21%

	2nd grade KUMAGE J.H.S (2014)		3rd grade KUMAGE J.H.S (2015)	
	before	after	before	after
answer				
think	49%	45%	41%	39%
N/A	31%	22%	33%	29%
did`nt think	19%	33%	26%	32%

5.3 STATUS OF COMPLIANCE WITH SEVEN ROAD-SAFETY RULES

Before the lecture, the three items of “Keep to the left when walking a narrow road,” “Do not cross a broad road where there is no pedestrian crossing” and “Walk on the opposite side of the walkway from the road” were observed only 50% of the time, compared with other items; however, after the lecture it increased to 70%. From the above, it was clarified that students’ awareness was also promoted by the lecture in Kumage Junior High School. (Figure 7).

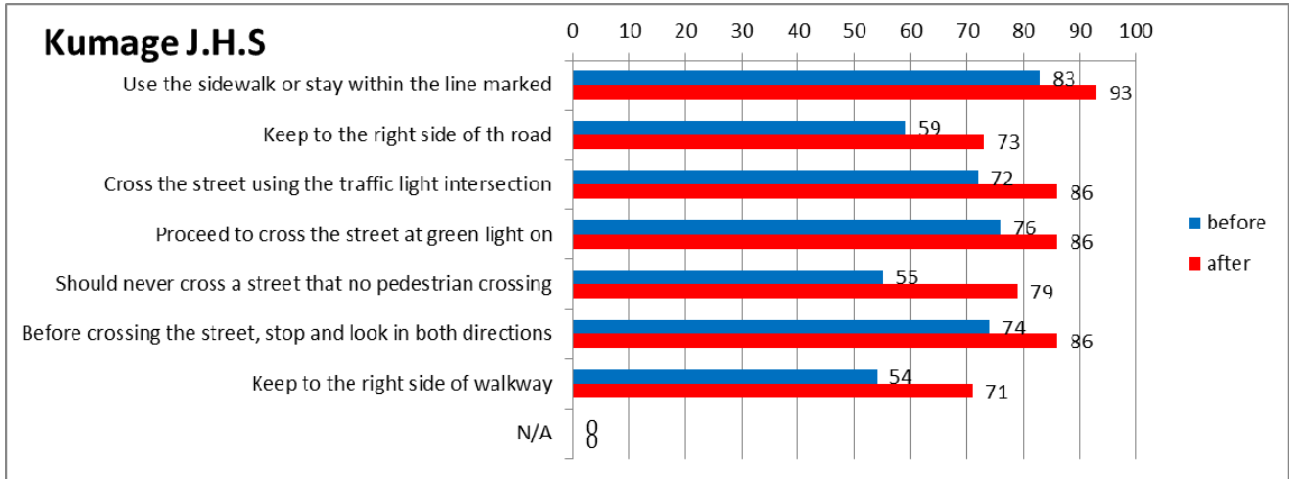


Figure 7. Compliance of Seven Road Safety Rules, Kumage Junior High School (Kumage J.H.S)

5.4 COMPLIANCE INTENTION OF FIVE RULES FOR SAFE USE OF BICYCLE

Before the lecture, the awareness of “Can use walkway only when it is specially approved” and “Go slowly on the left side of roadway” were low in particular. It is considered that the awareness of the students commuting to school by bicycle became low because their commuting means changed(Figure 8).

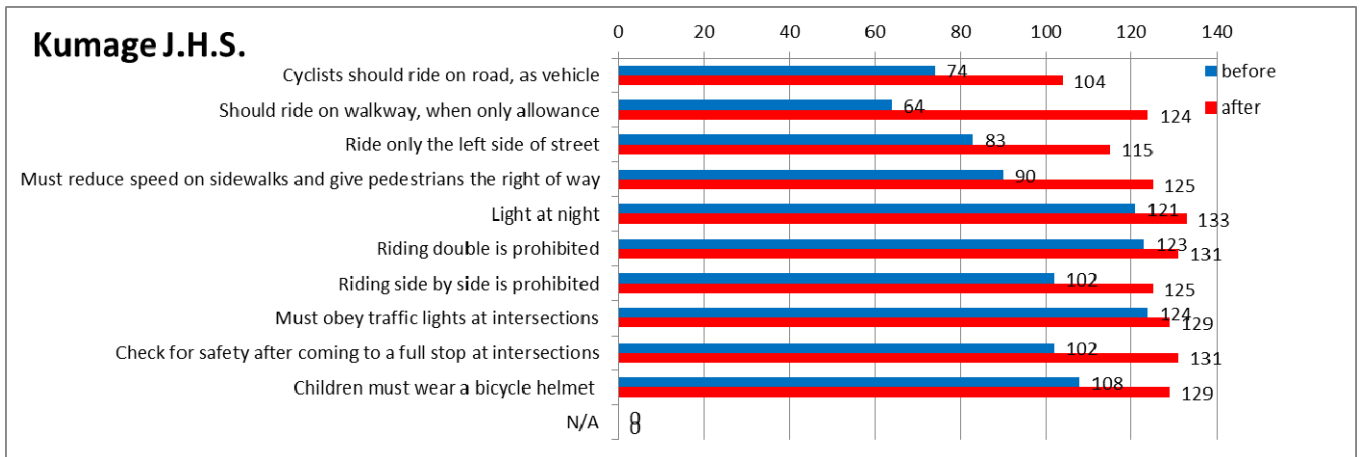


Figure 7. Compliance Intention of Five Rules for Safe Use of Bicycle, Kumage Junior High School (Kumage J.H.S)

6. CONCLUSIONS

From the results of research, following were clarified:

The higher grade students had higher risk awareness of the route to and from school. However, it is often observed that when they proceeded to junior high school, their risk awareness fell.

The lecture on the seven road safety rules and the five rules for safe bicycle use provided to students of primary school and junior high school could make their traffic behavior safer.

Their risk awareness maintained the same level into the next grade year.

To conclude, it was confirmed that in order to encourage compliance with the rules of the road, the implementation of road safety education to both primary school and junior high school is beneficial.

The author would like to deploy the same method to other areas of Shunan City, Yamaguchi Prefecture and continuously conduct research.

7 ACKNOWLEDGEMENTS

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This is part of the achievement obtained through the instruction by our laboratory activities to graduate research students of National Institute of Technology, Tokuyama College (Kozo Harada, Keita Fukuda and Mao Harada) in the previous year.

It is also part of the achievement of the research made possible by the Sasagawa Scientific Research Grant from The Japan Science Society. I would like to thank all those who have been involved in this research.

8 CITATIONS AND REFERENCES

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Sustainability, Pavement, Asphalt, EME2, Perpetual

ABSTRACT:

With a drive toward sustainability and pavement longevity, there is an ever growing evolution in pavement materials and design concepts. In this paper, Pavement Management Services has proposed four mechanistic-empirical design options for a failing road in Moreton Bay Regional Council (MBRC) in Queensland, Australia in 2016. Of the four pavement designs, two consisted of conventional asphalt and granular pavements, whilst the others consisted of foamed bitumen and a perpetual pavement design.

An extensive pavement investigation was carried out including Falling Weight Deflectometer, boreholes, and geotechnical testing; which was used in assessing the in-situ conditions of the pavement and material properties. The investigation allowed formulating of the foamed bitumen and a perpetual pavement design alternatives. The foamed bitumen option considered the utilisation of existing materials to incorporate into the rehabilitation, whilst the perpetual pavement considered a three layered asphalt pavement for an extended lifecycle. The three layered asphalt coarse (applied to the perpetual pavement), utilised the characteristics of three different mixes to limit strain at critical locations within the pavement, to control the mode of failure. By doing so, the pavement was set to last in excess of 100 years without structural improvement and minimising pavement life cycle cost.

Furthermore, a financial analysis has been conducted incorporating a preliminary cost comparison between convention asphalt design (deep lift asphalt) and perpetual pavement design. A 100 year analysis period was considered for a better comparison with the perpetual long life pavement.

This paper outlines the investigation and pavement design options and methodologies used in the design process. The pavement designs have been proposed and accepted by council, and are awaiting a final decision in the design option for tendering process.

Sustainable Pavement Design - Delivering Sustainable Options Considering Existing Materials and Pavement Lifecycle.

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1 INTRODUCTION

Large road networks, aging infrastructure and material availability are just a few terms to describe a typical councils continuing battle to maintain their road assets to an acceptable level of service and customer satisfaction. Also adding population growth, asset acquisition, budget constraints and time constraints, makes a good argument to strategise towards sustainable and innovative techniques and materials, utilising existing infrastructure and extending asset design lives.

With no exception the above mentioned stressors and constraints, Moreton Bay Regional Council (MBRC) engaged Pavement Management Services (PMS) in 2015 for a pavement design on an 820m deteriorated road in their network - Victoria Avenue, Margate, QLD. The arterial road is one of the major routes in a grid network seeing some 7.1×10^3 vehicles per day - the majority of which are light vehicles serving the surrounding residential properties and schools with 4.0% heavy vehicles.

The pavement investigation encompassed a visual inspection to understand the mode of failure and possible design constraints; boreholes for pavement profile and material characterisation; laboratory testing for material properties and Falling Weight Deflectometer (FWD) testing for the in-situ structural condition. Utilising this information, the remaining life of the pavement was determined following the linear elastic layered system and mechanistic-empirical methodology. The four proposed design options - also following the mechanistic-empirical design method, consisted of a mill and re-sheet, deep lift asphalt, foamed bitumen stabilised and a long life perpetual pavement using a three layered asphalt coarse - incorporating the high modulus asphalt Enrobés à Module Elevé Class 2 (EME2).

The aim of this paper is a case study into the methodology of the pavement investigation. The paper will focus on how the results were utilised, the specifications and guidelines used and the design options considered – focusing on the foamed bitumen treatment and long life perpetual pavement design using EME2.

A preliminary cost analysis comparison between deep lift asphalt and perpetual pavement designs has also been included in this paper.

2 VISUAL INSPECTION

The start of any investigation is to ascertain what the current state of the pavement is. This may be in the form of destructive, invasive and non-destructive testing, however, a simple ‘on foot’ survey with a trained eye can reveal the mode failure, construction constraints, traffic movements and directions to take when considering testing and design.

For Victoria Avenue, the on foot inspection revealed a large asphalt paved surface consisting of a two lane - bidirectional road, with road side parking varying from 13m to 18m total width. The condition was poor, with regular occurrences of fatigue cracking, longitudinal and transversal cracking and remedial works in the form of crack sealing, pothole repairs and heavy patching. There was also evidence of pumping with light colored material appearing where cracks were present. Rutting and deformation of the pavement was not a prevailing pavement distress, indicating a reasonable base and subgrade condition. The table below shows photos of the pavements general condition.

Table 1. General Pavement Condition



2 PAVEMENT INVESTIGATION METHODOLOGY & RESULTS

2.1 BOREHOLES AND LABORATORY TESTING

A total of ten (10) boreholes were drilled at regular intervals along the Victoria Avenue. The thicknesses of the pavement layers were recorded, with material characterisation, which were interpreted for an adopted pavement profile. Dynamic Cone Penetration (DCP) tests were also carried out on the subgrade at each borehole as an indication of insitu subgrade strength. The results of the borehole logs revealed variations in the pavement profile, not only in the asphalt coarse, but in the unbound granular base coarse. The asphalt thickness was typically 60mm thick with two boreholes seeing 100mm. The base material varied between 170mm to 445mm with the thinner pavement trending toward the end of the road.

The subgrade material was quite consistent as a Sandy Clay and a Clayey Sand to a lesser extent. The DCP results of the subgrade were sporadic, ranging from a correlated CBR of 9% to 24%, however, lab testing and back calculation will provide more insight to the subgrade strength and insitu stiffness.

The base and subgrade samples taken from the boreholes were sent to the laboratory for further testing. Laboratory testing consisted of moisture content, particle size distribution (PSD), Atterberg limits and California Bearing Capacity (CBR). The PSD was used to classify the granular material in terms of particle size distribution and subsequent material grading in accordance with Main Roads specification MRTS05 (DTMR 2015b). The results found that most samples fell in the lower grade material envelope 'E' with one exception as a grading envelope 'C'. The ratio of the 0.075 and 0.425 sieve also characterised this material as a high quality Type 2.1, where the grading envelope 'E' materials have no material type characterization.

For a foamed bitumen treatment, it is recommended a grading envelope 'C' should be used, however, Ramanujam J. et al. indicates a broader grading envelope reproduced in the table below, in which seven out of the nine samples adhere to (Ramanujam. J et al. 2009). The percent passing the 0.075mm sieve was also well above 5% which is a critical property recommended by Austroads to get a good mortar like bitumen mastic between larger aggregate particles (Austroads 2011). On this basis, foamed bitumen was considered a viable design option.

Table 2. Grading Limits for Foamed Bitumen Treatment

Sieve Size (mm)	Percentage Passing By Mass	
	Minimum	Maximum
26.5	80	100
2.36	25	85
0.075	5	20
Plasticity Index*	Maximum 10%	

*Plasticity Index for the base material was not tested

In addition to PSD of the granular base material, Atterberg limits were taken for all subgrade samples, along with eight CBR's from selected boreholes. The Subgrade proved to be variable across Victoria Avenue, however, showed sound CBR values ranging between 6% and 12%. The moisture content

was close to -and generally below- its Optimum Moisture Content (OMC) and the swell was very low, with exception of the subgrade material in borehole 4. Following the Austroads Guide Part 2, the lab results indicate a prominent subgrade material of a low to moderate expansive nature, however, indicate weak areas of very high expansive nature.

2.2 FALLING WEIGHT DEFLECTOMETER (FWD) TESTING & CUSUM ANALYSIS

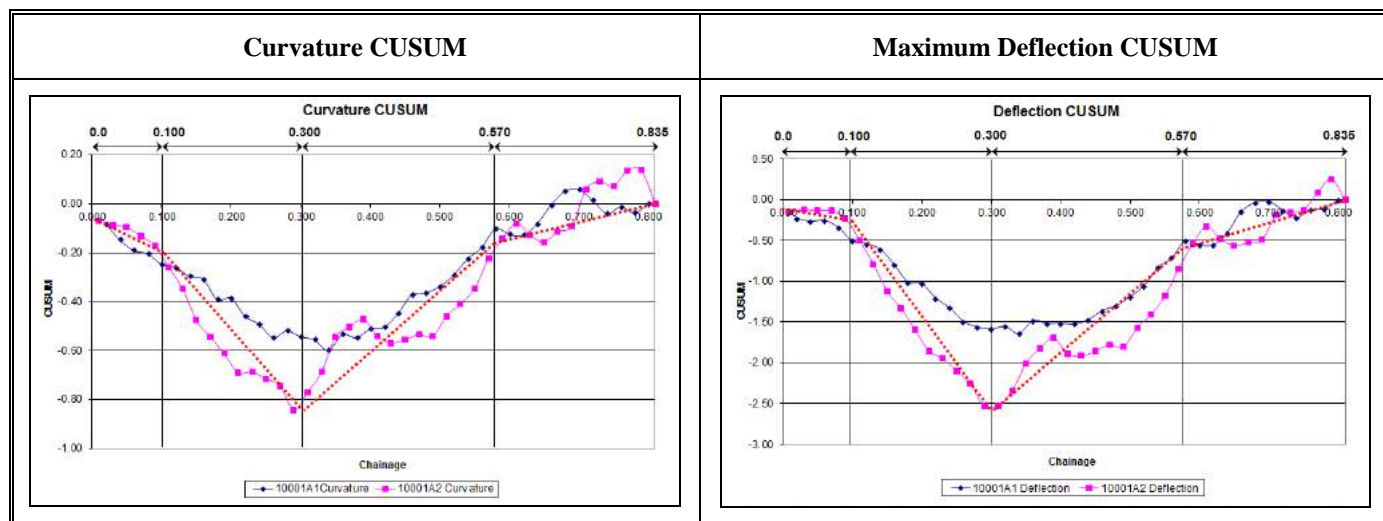
The FWD testing was conducted on both the trafficable and parking lanes of Victoria Avenue using a Dynatest Heavy Weight Deflectometer (HWD). Due to the standard principles of the testing, the HWD will be referred to as FWD for this paper. For the trafficable lanes, the testing was performed at 20m intervals, staggered between the two lanes (10m stagger) and at 40m intervals on the outer wheel path for the parking lanes. The loading of the FWD was conducted at 40kN. At each test point the peak applied load and peak deflections were recorded from 9 geophones, with spacing's ranging from under the center of the load to a distance of 1.5m from the load. The deflections were used to define changes in the pavement structure using statistical methods and for the back calculation of pavement layer moduli. For the purposes of this paper, the results will focus on the trafficable lanes of Victoria Avenue as this is where the deep rehabilitation was proposed.

Defining the existing pavement profile is a key element to successful pavement modeling and design. As noted in the borehole investigation, there were variations in the existing pavement profile, however, boreholes do not define where the changes are. In this case, FWD deflections were used to identify areas of similar response and hence similar construction and changes. To identify these changes in the pavement profile, a Statistical Cumulative Sum (CUSUM) analysis was carried out.

The CUSUM is a sequential analysis typically used for monitoring change detections. It is a powerful tool in defining zones of similar response within a project dataset. Pavement areas with consistent response appear as fairly straight lines on the CUSUM plot, while points where the slope changes dramatically correspond to section boundaries.

The table below shows the CUSUM for curvature and deflection which identified homogeneous sections for both trafficable lanes.

Table 3. Characteristic Deflection Plot and CUSUM



The CUSUM plot revealed four homogenous sections of similar response. Utilising this method and interoperating information from the boreholes, four pavement profiles were adopted and used in the subsequent pavement modeling and design.

2.4 BACK CALCULATION FOR PAVEMENT LAYER MODULI

Based on the deflection results of the FWD testing and profiles determined from boreholes and CUSUM analysis, the existing pavement layer elastic moduli values were back-calculated using the ELMOD6 computer program and the deflection basin fit method. The following table is a summary of the modulus results and adopted pavement profiles.

Table 5. Pavement Layer Moduli

Chainage (m)	Asphalt (MPa)		Base (MPa)*		Subgrade (MPa)	
	Thickness (mm)	Modulus (MPa)	Thickness (mm)	Modulus (MPa)	Thickness (mm)	Modulus (MPa)
0 - 100	110	2294	790	154	Infinite	92
100 - 300	63	3732	576	396	Infinite	102
300 - 570	83	1326	373	170	Infinite	61
570 - 835	59	2095	210	400	Infinite	61

The subgrade modulus correlated well with the lab CBR results ranging between 59MPa to 120MPa whilst the granular base ranged between a low 127MPa to 480MPa. The asphalt modulus also ranged between a low 1,241MPa to 2,376MPa. These characteristic moduli values were used in the design and were based on the lowest tenth percentile level of confidence.

2.6 MECHANISTIC EMPIRICAL PAVEMENT DESIGN

The pavement designs were completed in accordance with the General Mechanistic-Empirical Procedure published in the Austroads Guide to Pavement Technology Part 2, as well as the Department of Transport and Main Roads (DTMR) Pavement Rehabilitation Manual and design supplement to Austroads Part 2 (Austroads 2012) (DTMR 2012)(DTMR 2013).

The Mechanistic-Empirical procedure, like the name suggests, encompasses a mechanistic modeling approach to pavement design, utilising the theory of elasticity and properties of the pavement materials to predict stresses and strains at critical locations within the pavement. The empirical side, or sometimes referred to by researchers as a calibration model, is used to define the limiting criteria at which the pavement will be considered to fail and hence, derived for empirical observation. The amalgamation of these two concepts yields an allowable number of axle repetitions that the pavement can withstand before exceeding the limiting criterion.

These failure mechanisms are evaluated based on traffic loading from a standard axle configuration consisting of a dual-wheeled single axle loaded with 80kN. The critical locations within the pavement –for these particular designs- consist of tensile strain at the bottom of the asphalt, and the compressive strain at the top of the subgrade.

2.6.1 FATIGUE OF ASPHALT MATERIALS

The pavement design to prevent fatigue of the asphalt is designed to limited tensile strain at the bottom of the asphalt layer to a tolerable level throughout the life of the pavement. The asphalt fatigue relationship used in the Austroads Guide is the laboratory fatigue relationship published by Shell (1978) adjusted to predict fatigue life in the pavement using a reliability factor according to the desired project reliability (Austroads 2012).

For conventional bituminous binders used in asphalt placed on moderate-to-heavily trafficked pavements, the general relationship between the maximum tensile strain in asphalt produced by a specific load and the allowable number of repetitions of that load is:

$$N = RL \left\{ \frac{6918 \times (0.856 \times Vb + 1.08)}{S_{mix}^{0.36} \times \mu\epsilon} \right\}^5 \quad (1)$$

Where:

- N: Allowable number of Standard Axle Repetitions (SAR).
- $\mu\epsilon$: Tensile strain produced by the load (microstrains).
- Vb: Percentage by volume of bitumen in the asphalt (%).
- Smix: Mix stiffness or modulus (MPa).
- RL: Reliability factor for asphalt fatigue (Table 6).

Table 6. Project Reliability

Desired Project Reliability				
80%	85%	90%	95%	97.5%
2.5	2.0	1.5	1.0	0.67

For the foamed bitumen design this equation was used to evaluate the tensile strain at the bottom of the foamed bitumen layer, and also used for the bottom of the deep lift asphalt design. For the perpetual pavement design, this equation was not used as the limiting factor for this design was the stain itself, not asphalt fatigue.

2.6.2 SUBGRADE PERMANENT DEFORMATION

Subgrade permanent deformation is considered in the mechanistic-empirical design by limiting the vertical compressive strain at the top of the subgrade to a tolerable level throughout the life of the pavement. The strain induced is not fully recoverable, and after repeated loading, the permanent deformation accumulates at the subgrade level, and overlaying pavement layers. These permanent deformations are typically manifested as rutting in the wheel paths.

The number of cycles to failure until an unacceptable level of permanent deformation of the subgrade is computed by the following equation, noting that Reliability Factors have not been incorporated as the relationship is expected to provide suitable reliability for SARs up to 1×10^8 .

$$N = \left(\frac{9,300}{\mu\epsilon} \right)^7 \quad (2)$$

Where:

- N: Allowable number of Standard Axle Repetitions before failure criterion is met.
- $\mu\epsilon$: Vertical strain at the top of the subgrade (microstrains).

As the unbound granular materials are considered stress dependent, it is important to consider that the modulus of unbound materials is strongly influenced by the level of stress, which at the same time will limit the allowable number of repetitions. In other words, considering the stress dependency of the unbound materials, like a natural subgrade, it will carry a higher number of Standard Axle Repetitions if it has a high resilient modulus, which means high stiffness in the presence of a traffic load.

2.7 PAVEMENT DESIGN OPTIONS

In accordance with the design proceedings the following assumptions and limitations were used in the design process.

- Each design considers six seasons throughout the year incorporating the minimum and maximum air temperature and mean monthly rainfall at the project site, determined from Bureau of Meteorology
- Pavement surface temperature throughout the year is based on BOM data and the US Asphalt Institute relationship published in the Superpave Series No. 1 (SP-1), considering the latitude location of the project site.
- Predicted asphalt pavement temperature at different depths was based on the BELLS equations.
- Modulus of new asphalt materials are based on asphalt bitumen and mix volumetric properties using the SHELL nomographs, temperatures and loading frequency for each of the six seasons throughout the year.

- Sub-layering of granular materials was in accordance with the Austroads method.
- Fatigue Endurance Limit (FEL) was targeted at 100 micro strains and 200 micro strains for the subgrade.
- Foamed Bitumen and EME2 specification and design parameters followed DTMR and Austroads Spec.
- All treatments were structurally equivalent and have been designed to meet a structural design life of 20 years with exception to the Perpetual Pavement Design (100+ years).

In consultation with MBRC representatives four (4) pavement designs were proposed including a mill and re-sheet for the parking lanes and a deep lift asphalt, foamed bitumen stabilised and a EME2 long life perpetual pavement for the trafficable lanes. The following section details the three pavement designs considered for the trafficable lanes – a simple mill and re-sheet option was proposed for the parking lanes as a cost effective option.

2.7.1 FOAMED BITUMEN DESIGN

This treatment considered pre-milling the existing asphalt material into the base material to a depth less than the final foamed bitumen depth. The material is then to be graded to height in which material will be removed to allow for a new asphalt wearing coarse and to keep the existing levels. The material will then be treated with hydrated lime, then foamed bitumen using a in situ stabilisation machine, compacted, and capped with a wearing coarse.

The foamed bitumen layer was modeled as an asphalt layer and the modulus determined from the mix properties. The foamed bitumen mix consisted of 3.5% Class 170 bituminous binder and 2% lime as a secondary stabilising agent following Austroads (2015), although may be altered depending on material susceptibility from sampling and testing (Ramanujam. J & Jones. J, 2000). The following is a summary of the foamed bitumen design.

Table 7. Foamed Bitumen Design Summary

Direction	Chainage (m)	New Dense Grade 14mm (mm)	Foamed Bitumen (mm)	New Granular Base Type 2.1 (mm)	Existing Granular Base (mm)
Prescribed/Counter	0 - 100	50	160	-	690
Prescribed/Counter	100 - 300	50	100	-	486
Prescribed/Counter	300 - 570	50	195	-	210
Prescribed	570 - 835	50	100	-	119
Counter	570 - 835	50	150	230	-

The design was carried out utilising the existing pavement structure to be mixed in with the foamed bitumen, and also as a granular base. With a 50mm asphalt wearing coarse, the foamed bitumen layer was increased in thickness to satisfy the failure criterion. The existing granular material was also modeled in the design with moduli derived from the FWD testing and back calculation. In all but one case, the existing granular material remained as a granular base, however, was too thin to be considered a structural layer at the end of Victoria Avenue in the counter direction – in which the subgrade needed to be excavated and new material laid.

2.7.2 DEEP LIFT ASPHALT DESIGN

This treatment considered milling the existing asphalt surface and all/part of the base material, and replacing with new asphalt materials consisting of an asphalt base and wearing coarse. The following is a summary of the deep lift asphalt design.

Table 8. Deep Lift Asphalt Design Summary

Direction	Chainage (m)	New Dense Grade 14mm (mm)	New Dense Grade 20mm (mm)	Existing Granular Base (mm)
Prescribed/Counter	0 - 100	50	120	730
Prescribed/Counter	100 - 300	50	80	507
Prescribed/Counter	300 - 570	50	150	255
Prescribed*	570 - 835	50	70	149
Counter*	570 - 835	50	125	94

Like the foamed bitumen design, the deep lift design consisted of a 50mm wearing coarse, and the asphalt base thickness was increased to a thickness that satisfied the failure criterion.

2.7.3 PERPETUAL PAVEMENT DESIGN USING EME2

This design considers a relatively new concept in pavement design where the life of the pavement is set to exceed 40-50 years without undergoing any structural rehabilitation as opposed to a 20 year design (D. Timm, R. Willis,). The notion involves designing a pavement to limit the flexural strain at the base of the asphalt layer (Fatigue Endurance Limit -FEL), and the vertical compressive strain at the top of the subgrade.

Conceptually, if one was to think of a rubber band that is subject to a cyclic loading (traffic) and is stretched to a limit (strain defined by the pavement thickness and modulus) - it would fail at a set number of cycles (20 year design) - this would represent a traditional pavement design. However, if a rubber band was subject to the cyclic loading only by a small amount (little strain) then the rubber band would recover and last indefinitely (conceptually). This strain limiting criterion is the principal behind this paradigm shift in pavement design and works towards a perpetual pavement lifecycle and getting the most out of your asset with minimal lifecycle maintenance and cost. The FEL is not a new concept in pavement design and has been hypothesised by Monismith from as early as 1972, indicating a FEL of 70 micro strains, however, Witczak indicates that a range between 70 and 250 micro strains may exist depending on the asphalt mix (Monismith C.L & McLean D.M 1972)(Witczak 2014). For the perpetual design of Victoria Avenue, the FEL was targeted at 100 micro strains as 70 is considered as too conservative (Sullivan B et al.). The strain at the top of the subgrade was limited to 200 micro strains (Newcomb D. et al, 2001)(Liedy J. 2014).

The design consists of a multi layered asphalt pavement comprised of an asphalt wearing coarse on top of a high modulus, rut resistant asphalt base, and a fatigue resistant asphalt material at the bottom. The top and bottom asphalt layers for this design consisted of conventional dense grade asphalt mixes with nominal 14mm and 20mm aggregate size respectively. The mid layer consisted of the high modulus EME2 mix. The EME2 mix is a hot mix -performance based asphalt- that was designed in the mid-seventies in France. The mix consists of a high viscosity bituminous binder with its stiffness attributed to the bitumen properties and hard penetration grade (Austroads 2014). The product is now being implemented into Australia by road authorities and other road agencies. This design followed the pilot specification published by the DTMR for the EME2 mix properties and modulus determined using the same methodology as previously discussed (using shell nomographs and temperature adjusted to six seasons) (DTMR 2015). The table below shows a summary of the perpetual pavement design.

Table 9. Perpetual Pavement Design Summary

Direction	Section	Location	New DG14 (mm)	EME2 (mm)	New DG20 (mm)	Existing Granular Base (mm)	New Granular Base Type 2.1 (mm)
Prescribed/Counter	1	Ch 0.000 to Ch 0.100	50	120	110	630	-
Prescribed/Counter	2	Ch 0.100 to Ch 0.300	50	100	90	397	-
Prescribed/Counter	3	Ch 0.300 to Ch 0.570	50	140	140	125	-
Prescribed/Counter	4	Ch 0.570 to Ch 0.835	50	130	140	-	200

As can be seen, to limit the strain at the critical locations, a thick asphalt layer is to be placed, consisting of a 50mm DG14 wearing coarse, a 100 to 140mm rut resistant EME2 mix and a 90 to 140mm fatigue resistant DG20 mix. Although the overall asphalt thickness will incur high initial costs, consideration must be given to the designs structural longevity and minimal maintenance requirement throughout its life cycle.

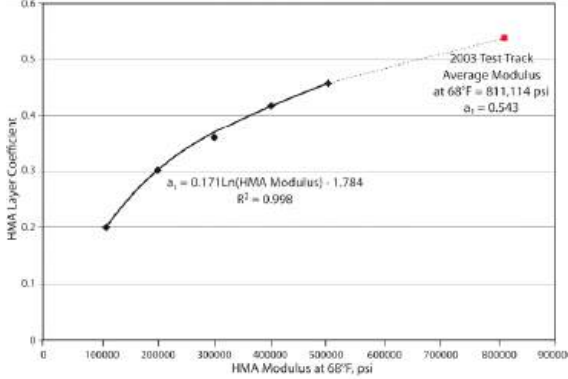
2.8 LIFE CYCLE COST ANALYSIS

A preliminary cost analysis has been conducted using the AASHTO 1993 Pavement Design Guide for modelling the pavement performance for a period of analysis of 100 years. The cost comparison was performed using the deep lift asphalt and perpetual pavement design options. On this basis several

rehabilitation options were considered along the period of analysis (100 years) using deep lift asphalt rehabilitation strategy.

for cost comparison purposes, the modeling was conducted on section No. 3 (counter direction) of Victoria Avenue between Ch 0.300 to Ch 0.570. The following table summarises the AASHTO-93 modelling variables.

Table 10. Structural Analysis. AASHTO-93 Variables

Material	Type	Stability/Module	a_i AASHTO-93 Method	Reference
Asphalt Mix	DG14 / DG20 Nominal Size aggregate = 18 mm. % asphalt cement = 4-6 (by Mass) Total Voids = 3-7%	7.5 KPa. (1,686 lb.) Flow=2-4 mm.	0.40	<u>Stability:</u> Brisbane City. Urban Management Division Reference Specifications for Civil Engineering Work. <i>S310 Supply of Dense Graded Asphalt. December 2001</i>
Asphalt Mix	EME2	25°C~68°F/80 KPH: 4,200 MPa (609,159 psi.)	0.55	 <p><u>Module:</u> Department of Transport and Main Roads. Queensland Government. Australia. <i>Technical Note 142. High Modulus Asphalt (EME2). Pavement Design. March 2015</i></p> <p><u>Structural Coefficient (a_i):</u> NCAT (U.S.A) Research.</p>
Granular Subbase	TYPE 2	127 MPa (18,420 psi)	0.09	Drainage Coefficient (m) = 1.0
Subgrade	BH5: Sandy Clay, Med/High Plasticity. BH6: Clayey Sand, High Plasticity Clay Fines	59 MPa (8,557 psi) Equivalent Design CBR Value = 6%	N/A	

1 KN = 224.8 pound 1 MPa. = 145.05 psi. 1 Inch = 25.4 mm.

Using the AASHTO-93 modelling, traffic information, and the variables shown above, the following Standard Axle Repetition to failure (SAR's) and pavement design life (n) were obtained.

Table 11. Traffic Information

Growth Traffic Rate (%)	1,8
Design Period (Years)	20
Growing Factor (F)	24,03

Table 12. Deep Lift Asphalt

Reliability R (%)		96
SN	e_i (mm.)	4,05
	a_i	
	50	0,40

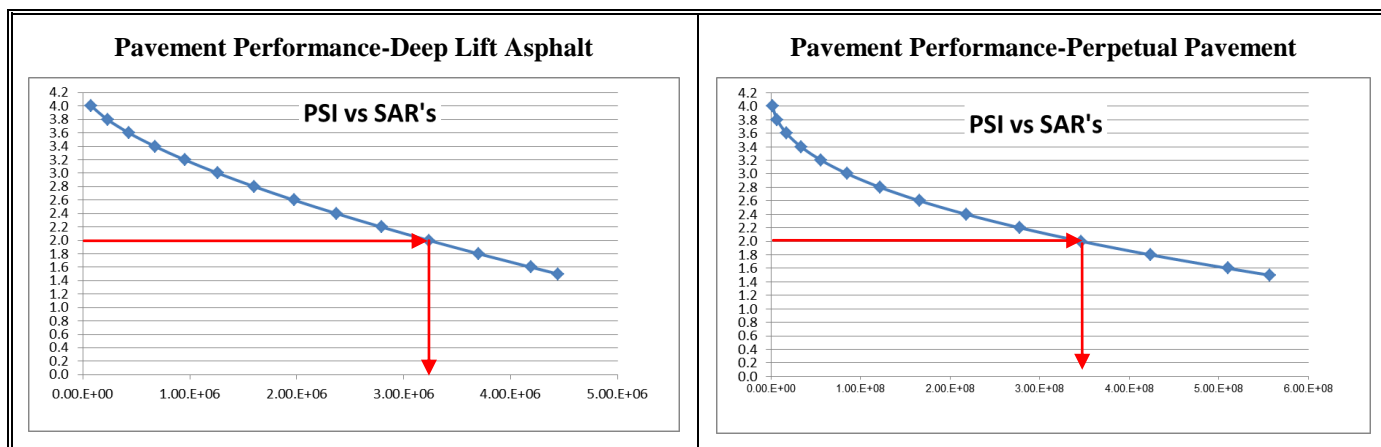
	150	0,40	
	255	0,09	
Po			4,2
Pf			1,5
WT18			4,4447 x 10 ⁶
<i>Design Standard Axle Repetition for Fatigue of Asphalt</i>			4,38 x 10 ⁶
Equivalent Axles (Initial year), EEO			182.249,00
n (years)			20

Table 13. Perpetual (long Life) Pavement

Reliability R (%)		73	
SN	e _i (mm.)	a _i	
	50	0,40	
	140	0,55	
	140	0,40	
125	0,09		
Po			4,2
Pf			1,5
WT18			556,571,328
<i>Design Standard Axle Repetition for Perpetual Pavement</i>			550.000.000
Equivalent Axles (Initial year), EEO			182.249,00
n (years)			227,47

As can be seen on the table above, following AASHTO-93 pavement design methodology the SAR's to failure and pavement design life expectancy (n) are very consistent compare to the Austroads M-E pavement design guide. On this basis a pavement performance behavior and structural analysis was conducted for a further cost life analysis. The following table shows a summary of the pavement performance and structural analysis using an increment of ΔPSI of 0.2 of both rehabilitation strategies.

Table 14. Pavement Performance-Structural Analysis



Using a final PSI (Pt) equal 2.0 as an intervention level, the expected SAR's_(f) were 3.24E⁶ and 3.47E⁸ for the deep lift asphalt and perpetual pavement designs, respectively. These SAR's_(f) at Pt equal 2.0 are reached at 15 years for the deep lift asphalt and 199 years for the perpetual pavement designs. It is equivalent to a damage (d) based on the relationship of the design SAR's_(ini) and expected SAR's_(f) ($d = \text{SAR}'_{s(f)} / \text{SAR}'_{s(ini)}$) of 0.73 and 0.54 for the deep lift asphalt and perpetual pavement designs, respectively. Using the condition factor (CF), damage value (d) and remaining life (RL) concept included in the AASHTO-93 guide, the effective structural number (S_{neff}) of both rehabilitation strategies was calculated. The S_{neff} of the deep lift asphalt is expected to be equal to 3.27 and it resulted in a structural rehabilitation number (SN_{reh}) of 0.78 (SN_(ini) – S_{neff}). The SN_{reh} of 0.78 is transformed into AC overlay using AC structural coefficient (a_{CA}) of 0.4 (similar to the DG14 and DG20 used for design). On this basis the AC overlay at 15 years (Pt=2.0) is 50mm. Regarding the perpetual pavement strategy, conceptually, the rehabilitation treatment is considered to be milling and replace the 50mm AC (DG14) on top of the surface.

This should allow the pavement structure to be perpetual considering that cracking is occurring from the surface to the bottom and the high modulus mix (EME2) should avoid the cracking progression to the lower layers. It is expected (based on AASHTO-93 modelling) that the perpetual pavement rehabilitation treatment to be conducted at the 199 year (Pt=2.0) of the initial design period.

For life cost analysis purposes life cycle period of 100 years of was evaluated. On this basis, after the initial construction cost of the deep lift asphalt strategy, six major rehabilitation treatments have been considered along the analysis period (100 years). The rehabilitation treatments are scheduled every 15 years. Along that period, 12 years of routine maintenance consisting of crack sealing, superficial and heavy patching are also scheduled. The major rehabilitation treatments consist of milling 120mm and replace 140mm of AC for the first two initial periods (15 year and 30 year). It will allow maximum overlay of 40mm in 30 years and having SN_{reh} of 0.78 as expected. The remaining major rehabilitation periods (45, 60, 75 and 90 years) are expected to be milling 200mm and replace 200mm in order to keep the surface level and same SN_{reh} of 0.78.

Regarding the perpetual pavement, after the initial construction cost, three surface treatments consisting on milling off 50mm and replace by 50mm were scheduled at 30, 60 and 90 years of the analysis period (100 years). It should be noted that based on the modelling, no surface treatment should be conducted on the perpetual pavement strategy up to 199 years of the design period. In addition, 10 years of routine maintenance (crack sealing) has been scheduled.

The following table shows the major rehabilitation treatments cost and routine maintenance cost scheduled to the two rehabilitation strategies.

Table 15. Deep Lift Asphalt Treatment Cost

Year	Treatment	Cost (\$/sqm)
0	Construction	569.60
15 and 30	Major: Mill 120mm and Replace 140mm	398.70
45, 60 75 and 90	Major: Mill 200mm and Replace 200mm	569.60
12 years during design period	Routine: Crack Sealing (initial 3 years)	10.00
	Routine: Surface Patching (next 4 years)	30.00
	Routine: Heavy Patching (final 5 years)	50.00

Table 16. Perpetual Pavement Treatment Cost

Year	Treatment	Cost (\$/sqm)
0	Construction	1075.50
30, 60 and 90	Surface Treatment: Mill 50mm and Replace 50mm	138.30
10 years during design period	Routine: Crack Sealing	10.00

Using the initial construction costs and routine maintenance shown in tables above the following Net Present Values (NPV) were calculated to the Deep Lift Asphalt and Perpetual Pavement rehabilitation strategies using a rate of 5%.

Table 17. Net Present Value

NPV (\$/sqm)	
Deep Lift Asphalt	Perpetual Pavement
-1,255.61	-1,156.39

The negative values of the NPV show the cost of the investment. Benefits that are in general positive were not included on this analysis.

2.9 CONCLUSION

After a detailed pavement investigation and testing, four pavement designs were proposed to rehabilitate Victoria Avenue in Moreton Bay Regional Council. Three of which were for the main carriage way and a cheaper option for the road parking lanes. The design process followed the mechanistic empirical design methodology and utilised properties of the mix designs to determine the asphalt material modulus and was adjusted for a six seasonal design. All designs incorporated the existing pavement structure where

possible, particularly the foamed bitumen treatment and utilised high performance materials for a perpetual pavement design.

The life cycle cost analysis showed that the NPV of the perpetual pavement strategy including routine maintenance cost is lower compared to the deep lift asphalt option. It makes the perpetual pavement strategy more attractive and cost convenience. A full benefit/cost (B/C) of the alternatives was not conducted, but it is anticipated that the perpetual pavement should have lower user costs (so higher benefits) compared to the deep lift asphalt, mainly because the condition of the pavement is higher for a longer period of time on the perpetual pavement option. It will make a better B/C ratio on this alternative. Other financial variables like inflation or variable rates were not considered in the cost analysis that was conducted.

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PAPER TITLE	INTEGRATED ASSET MANAGEMENT SYSTEM FOR THE SECOND PENANG BRIDGE (JAMBATAN SULTAN ABDUL HALIM MU'ADZAM SHAH)		
TRACK			
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KEYWORDS:

- Jambatan Kedua Sdn. Bhd. (JKSB) has embarked into an ambitious project of developing an Integrated Asset Management System (IAMS) for JSAHMS in January 2015.
- The project was tendered out and awarded to Gammerlite Sdn. Bhd. for a seven years contract which includes a one year on the system development stage and the remaining six years for the operation and implementation stage.
- Based on the construction cost for JSAHMS of RM4.5 billion and the numerous quantities of assets involved, the system is regarded as the first and the biggest integrated asset management system in Malaysia.
- The system has been successfully developed and currently into the early stage of operation and implementation period.

ABSTRACT:

Jambatan Kedua Sdn. Bhd. (JKSB) is the concessionaire responsible for the construction, operation and maintenance of the RM4.5 billion Jambatan Sultan Abdul Halim Mu'adzam Shah (JSAHMS). JKSB Asset Management System has been designed to comply with the internationally recognized ISO 55001:2014. JKSB is implementing the Integrated Asset Management System (IAMS) at a cost of RM134 million that includes a Structural Health & Monitoring System to provide a more realistic maintenance and repair program for critical assets with more than 500 numbers of sensors to be installed. The development of the Integrated Asset Management System (IAMS) for JSAHMS as a whole comprise of the following modules:

- Bridge Management System;
- Highway Management System;
- Building Management System;
- Mechanical, Electrical & Electronic Management System;
- Landscape Management System; and
- Accident Information Management System.

JKSB aims to acquire the maximum benefit and value-of-money and obtain innovative solutions in maintaining its JSAHMS assets by introducing the Performance Based Contracts maintenance contractors in the near future.

Implementation of IAMS with incorporation of ISO 55000:2014 system at JSAHMS shall deliver the highest quality of asset management towards meeting its organizational objectives of balancing financial, environmental and social costs, risk, quality of service and performance which are all related to the condition of the assets.

Integrated Asset Management System for The Second Penang Bridge (Jambatan Sultan Abdul Halim Mu'adzam Shah)

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1. INTRODUCTION



Penang has witnessed a rapid growth over the period of 15 years, being a major transportation hub. Despite the existing 1st Penang bridge connecting Perai on the mainland and Gelugor on the island, the traffic flow to the island has reached 65,000 vehicles per day and number is expected to rise further. The construction of **Sultan Abdul Halim Mu'adzam Shah Bridge (JSAHMS)** also known as the Second Penang bridge was aimed at alleviating the traffic woes at the existing bridge.

The second bridge is also expected to transform Penang into the key logistics and transportation hub for the northern region of Malaysia under the Northern Corridor Economic Region (NCER) programme and Growth Triangle comprising of Indonesia, Malaysia and Thailand (IMTGT). The construction of the iconic bridge was completed on 1st March 2014 and is the longest in South-East Asia with a total length of 24km, with 16.9km spanning across the straits of Malacca.

Jambatan Kedua Sdn.Bhd. (JKSB) is a company formed by the Malaysian Ministry of Finance and was appointed as the concessionaire to construct, manage, operate and maintain the bridge. Upon completion of the bridge construction, JKSB had tendered out a project to develop and implement an overarching systems solution to manage all its infrastructural assets; a system which is also envisioned to be instrumental for establishing an asset maintenance strategy and annual works programme for all assets. The project was called **Integrated Asset Management System (IAMS)** for Jambatan Sultan Abdul Halim Muad'zam Shah.

2. OBJECTIVE OF IAMS

There are two widely accepted definitions of asset management:

“the combination of management, financial, economic, engineering and other practices applied to physical assets with the objective of providing the required level of service in the most cost effective manner” (International Infrastructure Management Manual).

“a comprehensive and structured approach to the long-term management of assets as tools for the efficient and effective delivery of community benefits” (American Public Works Association).

The IAMS project is a holistic approach to infrastructural asset management as it endeavors to embed industry-wide accepted best practices in asset management in the IT systems; combined with the regular evaluation of asset condition during pre-determined site inspections to determine intervention or preventive measures to maintain the useful life of the assets and lower the Total Cost of Ownership.

There are two principles to the implementation of IAMS for JSAHMS; the capture and maintenance of the asset inventory with regular inspections and maintenance/rehabilitation of the assets throughout its useful life; real-time monitoring of the asset components by means of instrumentation which captures eigen-frequencies of the bridge and for the purpose of remotely managing streetlights to reduce energy consumption optimize the lifespan of the streetlights.

The asset condition data methodically gathered over a pre-determined frequency of periods form the basis for analyzing the performance and determining the deterioration model of the key asset components. Maintenance plan and intervention points shall be determined based on the system computed analytics and the expert opinion of the asset experts.

The high-level objective of IAMS project can be summarized as follows:

1. Processing and analysis of collected data and information to monitor performance against prescribed maintenance standards
2. Basis for understanding possible consequences of alternative decisions related to preservation of highway assets
3. Structured programme minimizing life cycle cost of asset ownership
4. Maintaining the required service level of JKSB as the concessionaire
5. Sustaining infrastructure
6. “Cradle to Grave” for maximum performance and usage of built asset

3. WHAT IS IAMS FOR JSAHMS

The Integrated Asset Management System (IAMS) comprise of specialized IT management systems for the assets that are managed under the IAMS implementation.

Assets are inventoried and organized by hierarchies to the lowest manageable level to facilitate realistic inspection programs and sustainable maintenance strategies.

The assets that fall under the bounds of IAMS are:

- The Highway Management System – where it can be further characterized as assets classed as pavement, road furniture and drainage
- The Bridge Management System – where the bridge components are structurally segmented by the approach spans, cable stayed section, the interchanges, culverts and the expansion joints
- The Building Asset Management System – which comprise of asset systems such as the infrastructure, rooms, internal fixtures and fittings, roof, internal and external fabric
- The Mechanical and Electrical Management System – which is further divided to M&E components in Building, Highway Lighting System, Traffic Control and Surveillance System and Toll Collection System

- The Landscape Management System – which are divided by the various zones and the landscape components which are trees, palms, shrubs and turf.

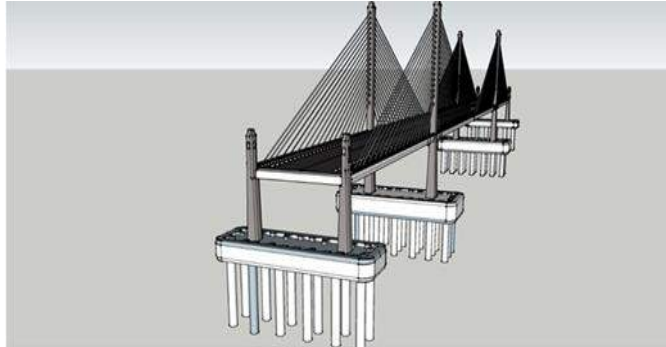


Figure 1: As Built Drawings Converted to 3D Drawings

Baseline data is obtained from the operational activities, historical asset data information from the construction-stage documents, in-depth understanding of the structures and its behavior to local climate and environment, and other engineering expert input combined, derive the deterioration model of the structures, trigger values for maintenance or rehabilitation works. IAMS is envisioned to assist the stakeholders in making decisions with regards to design of the structures and its performance.

As built drawings of the structures are digitalised and presented in 3D models and relevant documentation (construction drawings, design and test reports, etc.) are linked to each asset nameplate. The asset-linked document forms the knowledgebase from which operational and/or management decisions can be made. IAMS system is customised and configured to manage the various assets, their attributes, condition data, related documents, historical maintenance and rehabilitation information (work orders and Non Conformance Records (NCRs) and Supervision Observation Reports (SORs) issued etc.). The implementation of IAMS project for JSAHMS is generally split into the development phase (Year 1) and the implementation phase (Year 2 – 7).

Contract Period for Whole Works (7 years)		
Performance Bond (7 years)		
Development Stage (1 year)		Implementation Stage (6 years)
Development of Overall System Framework	System T&C	Full Implementation & Handling Over System
Development and Establishment of Computerized System and Data Structure		
Asset Registration and link to MHA		
Condition Data Gathering and Input		
		Defect Liability Period (6 years)
		Design Guarantee Period (6 years)
Maintenance and Support Period (10 years i.e. Contract Period (7 years) plus additional 3 years thereafter)		

Figure 2: Project Phase Framework

The first year of the project involves asset inventORIZATION (registration), asset condition data gathering which provides the baseline of asset condition, the establishment of the systems infrastructure framework and asset management systems. System requirements are acquired, analyzed and customized to meet the operational and management requirements.

Construction stage documents are digitalized by scanning the hard copies of the documents and storing them in the server. The documents are linked to the assets and organized by the asset records.



Figure 3: Survey Equipment for Asset Data Collection

The subsequent years of the project duration (implementation phase) involves the transition of operational activities using the system. System will evolve during the implementation phase to align with the operational processes and mature in terms of data and system features.

4. IAMS DELIVERABLES

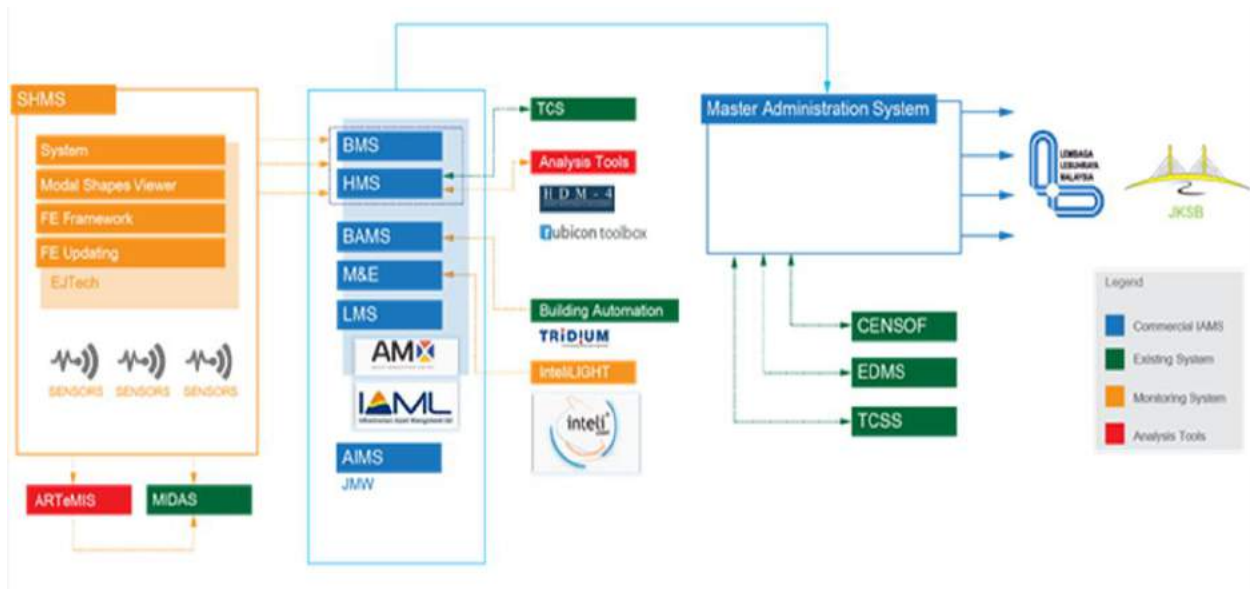


Figure 4: Development & Establishment of IAMS

The Asset Management infrastructure for Bridge, Highway (Pavement, Embankment, Drainage and Road Furniture), Building, M&E and Landscape will be managed by a single solution (AMX) software, implemented separately. AMX is capable of handling various sectors of infrastructure in a single interface. AMX will be clustered over separate servers for the individual asset sectors and the necessary configurations are performed to reflect the sector specific operations and functions.

The advanced analytical requirements of these individual core components will be supplemented by specialized systems respectively (e.g. Rubicon and HDM-4 for HMS). Existing systems are interfaced with IAMS to obtain supporting data to facilitate asset performance analysis.

HDM-4 or equivalent deterioration modelling/analyzing/ projection systems will be applied in AMX for pavement to project pavement condition and the cost associated with its maintenance and rehabilitation over a fixed timeframe. This will aid in planning, prioritizing and programming the maintenance works.



Figure 5: Asset Registration of M&E Components

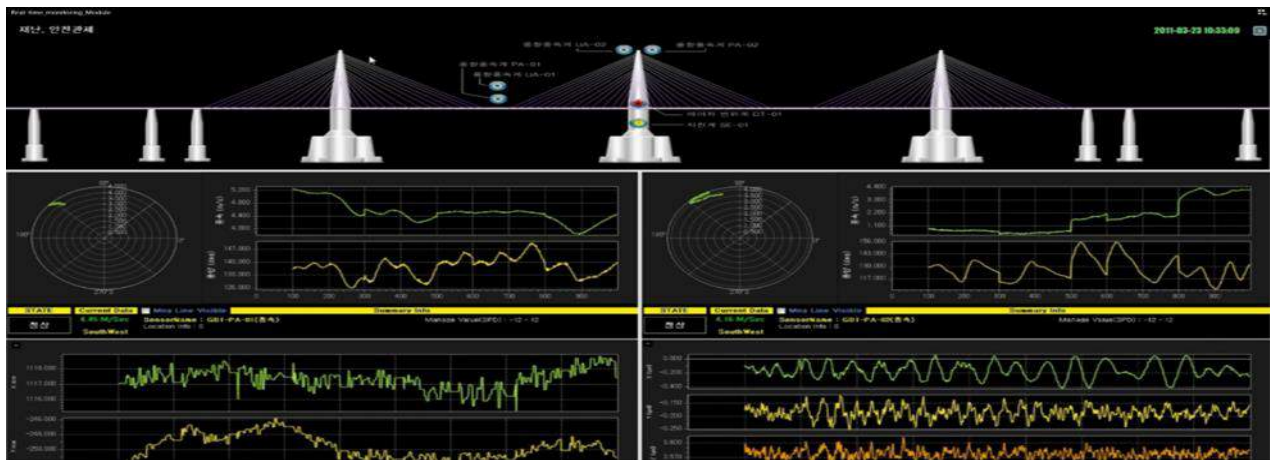


Figure 6: Structural Health Monitoring System

The building management system (BAMS) component of IAMS implementation will depend on the supplemented data provided by the existing Building Automation System (Tridium) that is already in place at JKSB's premises for information (from the various building environment controls) and inventory of all assets within the managed building(s). Readings from the various building control components will be imported into the BAMS to compute analysis data and present trending information on the Dashboard for management to make informed decisions and revise management or investment strategies.

The Structural Health Monitoring System (SHSM) will comprise of the sensory component (sensors placed on bridge and embankment), data acquisition units (PC-based data acquisition systems, which are a combination of permanent and portable), data processing (Ambient Vibration Monitoring), data storing (an SQL based database that stores the raw acceleration data) and data delivery system (built-in Modal frequency extractor) that provides graphical visualization (charts and graphs) of bridge responses in the wide screens of the SHMS monitoring centres.

The built-in Operational Modal Analysis (OMA) feature in SHMS will be used for automatic processing between the acceleration data reading component and the Modal Data Reader and viewer component within SHMS.

Flashnet's Intelilight street lighting control and management solution will be used with the M&E Asset Management System to manage and maintain the streetlights. Intelilight is an intelligent remote lighting management system that uses LONWorks (open protocol) to facilitate communication between each light pole in the street lighting grid and the system. The M&E management system and Intelilight streetlight management system combined, will be utilized to provide asset inventORIZATION and tracking, service life prediction analysis, maintenance management and reporting functionality.

The Accident Information Management System (AIMS) within IAMS provides accident plotting feature among other traffic collision information management capabilities which will be used by the Traffic and Accident Specialist and the Road Safety Auditor to plan to study and report on traffic accidents on a monthly basis. AIMS will be used as a Decision Support Tool to evaluate the safety and operational effects of geometric design decisions of the expressway.

The Landscape Management System (LMS) is configured to manage all components of the landscape as assets. The built-in analytics configured in LMS will assist in tree growth forecast, planning, budgeting, benefits calculation and more. The system also caters for root cause analysis for recurring tree defect observations.

5. STRUCTURAL HEALTH MONITORING SYSTEM (SHMS)

The SHMS is the dedicated real-time monitoring system which comprise of static and dynamic sensors connected via high speed Fiber Optic Backbone network, the high-end computer servers and network infrastructure and the software solution with analytic capabilities. A modular concept has been approached in the design of the SHMS with the intention to monitor structural condition and evaluate structural degradation as it occurs rather than detect structural failure. These fundamental modules are describe in the ensuing sections.

Sensory System

This module comprises of a variety of sensors and their respective interfacing units used to measure the physical parameters of the bridge structure and its surrounding environment. The measurements can be classified into four types which are environment, operation load, structural feature and structural response.

Data Acquisition Unit

This module will be implemented in with two categories of equipment which are the Data Acquisition Units (DAU) (controlled with proprietary software) and the network systems (tethered) which facilitate the acquisition of data, processing them, temporarily storing them and transmitting it to the monitoring centres.

Data Processing and Controls

The Data Processing component of SHMS encompasses automatic conversion of the raw data into useable information. It consists of data collection, transmission, pre-processing, analysis, post-processing, and storage and archival of the data in a format that is easy to access and present. The main goals of data interpretation in connection to the monitored structure are; structural identification during construction and operation, FE model updating, condition assessment, alarm configurations, service life prediction and maintenance planning. Data analysis may allow for damage location and quantification as well as for condition assessment.

Structural Health Monitoring Component

The SHMC is a routine and automated process of signal transmissions picked up by the relevant DAUs during continuous monitoring. In the event of detecting anomalous signals, a fault report will be generated to the system maintenance team to investigate the data further. Corrective procedures shall be initiated and the fault report will be kept in open status till normal operations are resumed.

When no anomalies are detected, the data processing and analysis module of SHMS will compare the measured data against the defined performance criteria for loads, environments, bridge characteristic and bridge responses. Performance criteria with definite threshold values will be defined and the signals transmitted will be compared against these threshold values for anomalies. If no anomalies are detected, SHMC will continue to monitor and update the database for the purpose of reviewing and refining the monitoring criteria and also to produce the necessary reports. These are carried out on routine. However, when any of these criteria is violated, the Structural Health Evaluation component will be triggered.

Seismic Activities Monitoring using SHMS

A seismometer is installed at the yard of JKSB site office close to the bridge. It gathers up to one hundred (100) uninterrupted samples per second of the three axes and the server has the capacity to store up to three (3) years of these data. The seismometer is configured to issue an alert seismic activities peaks beyond 10 gal.

During, the earthquake that took place at Southwest of Sumatera, Indonesia (approximately 1500km away from Penang) on 2nd March 2016, weak Peak Ground Acceleration readings were captured for all three axes. Though no apparent impact or damage was detected of the bridge, a weak signal of the earthquake is captured by the seismometer.

Similarly on 1st June 2016, an earthquake struck Kepulauan Mentawai Region, Indonesia which is approximately 820km away from Penang. Peak Ground Acceleration readings were captured for the three axes. While no apparent impact or damage was detected for JSAHMS, the highly sensitive seismometer detects the weak signals of the quake.



Figure 7: Earthquake at Kepulauan Mentawai Region, Indonesia



Figure 8: Peak Ground Acceleration Readings captured for the Earthquake at Kepulauan Mentawai Region, Indonesia.

6. BENEFITS OF IAMS DELIVERABLES FOR JSAHMS

The System Development stage for IAMS has been successfully completed and JKSB has embarked into the implementation stage for the next six years. We are confident that IAMS shall provide us with the following benefits:

- **Good Business Practice.** IAMS results in better decisions. Aligning management of infrastructure with strategic policies and direction will support the long-term success of the organization's mission, goals and objectives.
- **More Meaningful Financial Reporting.** The reporting of the cost of asset ownership can be derived when a cost element is included in the financial module of IAMS.
- **Improved Regulatory Compliance.** Standards and best practices embedded within IAMS framework will improve regulatory compliance for asset management. Part of asset management involves the implementation of better O&M practices, which can significantly improve compliance.
- **Improved Reliability.** More structured day-to-day attention to system assets and their condition means that unexpected failures are less likely, thus minimizing emergency repairs and customer relations problems. Assessing the risk implications of asset failure helps focus resources on critical priorities and reduces overall risk to the organization.
- **Long Term System Integrity.** The concept of "sustainable infrastructure" is gaining increased visibility, probably due to the problems in many sectors where sufficient reinvestment in infrastructure has not been made. By relating costs to asset condition and conducting long term planning for each asset, policy makers get the facts they need to help sustain the infrastructure.
- **Cost Savings.** There is evidence that asset management systems that maintain infrastructure in a sound and reliable condition and are based on minimizing life cycle costs, can significantly reduce operating and maintenance cost, as well as long-term capital expenses. A life cycle approach means that the utility always gets the most assets for its money.
- **Eligibility for Funding.** The apparent need for increased infrastructure spending coupled with concerns over the quality of infrastructure management prevalent in the industry have led to a range of provisions in proposed funding legislation that include requirements for "asset management plans." Although it is speculative at this time, it is likely that qualifying for funding assistance in the future will require a demonstration of some level of proper asset management.

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PAPER TITLE	DEVELOPMENT OF SUSTAINABLE AND CUSTOMIZED ROAD ASSET MANAGEMENT SYSTEM INCORPORATING THE LESSONS LEARNED FROM THE PAST INITIATIVES IN VIETNAM		
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ABSTRACT:

In recent years, Road Asset Management System (RAMS) is expanding in developing countries also with assistance from donor agencies. However, due to insufficient capacity of road administrator (human and financial) and lack of customization of the system into local context, RAMS is operable only in few countries. This paper attempts to analyze the reason of failure of past attempts of introduction of RAMS in Vietnam and introduce a new integrated RAMS which is being developed in Vietnam for national roads (currently 21,000km) with assistance from the World Bank and JICA. The new RAMS consists of sound road asset database with strict data quality control function, maintenance planning system for major road assets such as pavement, bridge and drainage. As for pavement, Pavement Management System is developed and it consists of manly three modules, namely pavement deterioration evaluation, strategic budget simulation (for long term plan) and repair work planning module (for short to medium term plan). Pavement deterioration evaluation and strategic budget simulation modules are customized by importation from Kyoto Model which was developed by Kyoto University, Japan. Pavement deterioration evaluation module evaluates the deterioration speed considering the most influential factors using Markov Transitional Probability theory which is incorporated in Kyoto Model. The strategic budget simulation with Monte-Carlo simulation is performed for three different scenarios (worst to best scenarios). Repair work planning module formulates annual and 5-year repair work plans with due consideration of pavement deterioration speed. Similarly, Vietnam Bridge Management System formulates maintenance plan based on bridge condition index which will be computed based on bridge inspection data. In addition to VPMS and VBMS, other management systems such as drainage management system are also developed. For RAMS's sustainability, other activities such as development of maintenance and management manuals, pilot repair works and trainings for central to field level staff are also carried-out.

DEVELOPMENT OF SUSTAINABLE AND CUSTOMIZED ROAD ASSET MANAGEMENT SYSTEM INCORPORATING THE LESSONS LEARNED FROM THE PAST INITIATIVES IN VIETNAM

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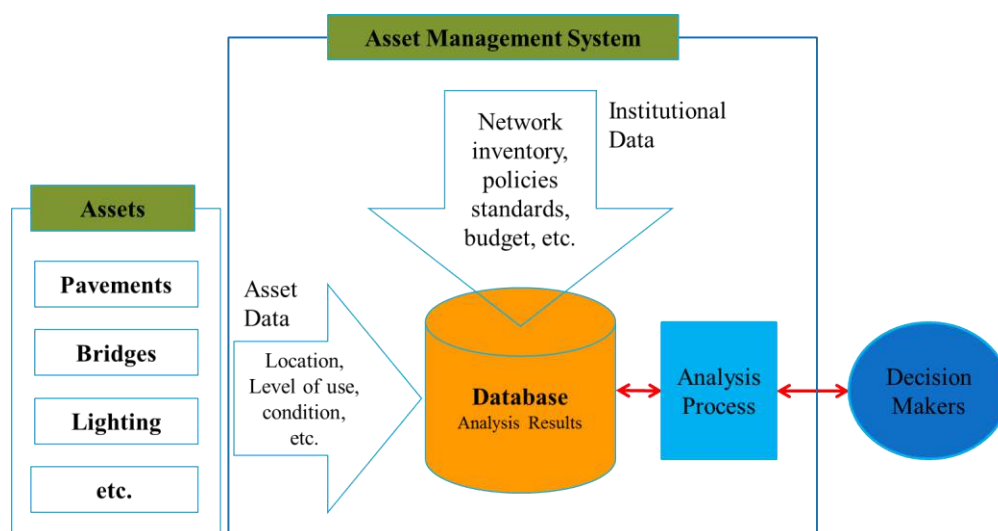
1. INTRODUCTION

Road transportation infrastructures are backbone of development for every country. Many countries have been making significant investments in road transport infrastructures. These infrastructures are used and exposed to natural forces, therefore their condition deteriorates over the time due to infrastructure aging, traffic loading and degradation due to natural exposure. It is challenging to all level of road agencies varying from central to local level to preserve the functionality of the existing transportation system while at the same time funding expansions of the transportation network to cope with the increasing demands. In developed countries, there is a gradual shifting of investment policy from new construction to maintenance management. However, in developing countries including countries with high economic growth rate, investment in new construction (i.e. expansion of road network) and maintenance of existing road transportation network are equally important. To cope with such kind of balancing investment approach, a systematic approach is needed to optimize the resources. In this regard, Road Asset Management System (RAMS) can play a crucial role by assisting decision maker to take appropriate decision. Road asset management is a systematic process of effectively maintaining, upgrading and operating road assets, combining engineering principles with sound business practice and economic rationale, and providing the tools to facilitate a more organized and flexible approach to making decisions necessary to achieve the public's expectations.

Road asset management offers a discipline for integrating asset data in a way that facilitates its use for decision-making. It provides a framework for managing road networks using a long-term perspective, rather than the relatively short-term view currently adopted by many road operators. These all contribute to improving the ability to deliver services in the long term, which is one of the catalysts for the implementation of modern asset management systems. According to the Organization for Economic Cooperation and Development (OECD), a sustainable road asset management system should include:

- Data collection methods that are affordable, appropriate and provide relevant information;
- Road information management system (or database management system) that is flexible and capable of producing both standard and ad hoc reports;
- A decision support system that can be used to investigate the consequences of various management decisions and strategies; and
- Adequate management information that are practical and pertinent to the needs of road organizations.

Figure 1 shows the key components of road asset management system. Asset management is a data-driven process that provides road operators with a very important analysis capability. Therefore, reliable and good quality data are indispensable and those data can make available only by collecting with standard method and stored in proper database management system.



Source: Asset Management for the Road Sector, OECD

Figure 1. Components of Road Asset Management System

2. PAST AND ONGOING INITIATIVES OF INTRODUCTION OF RAMS IN VIETNAM

Since 1998, several attempts have been made to implement RAMS in Vietnam under technical assistance from the World Bank (WB), Asian Development Bank (ADB), Japan International Cooperation Agency (JICA) and other donors. A number of decrees, decisions and circulars are promulgated and available in relation to road asset management in Vietnam. These regulatory provisions have been revised timely to cope with actual demands. Nevertheless, the actual implementations are not satisfactory due to various circumstances including mainly institutional issues, lack of trained human resources, and software issues. Table 1 shows the status of RAMS introduced in the past and their operational status at present in Vietnam.

Table 1. Existing RAMS software in Vietnam

Computer Software	Operability	National Roads			Provincial Roads	
		DRVN	RMBs	PDOTs	PDOTs	PPCs
Pavement Management System (PMS)						
VPMS	Operational (Upgrading)	√	√	√*		
HDM-4	Non-Operational	√				
RoSyBASE	Non-Operational	√	√	√		
RoSyPLAN	Non-Operational	√				
RoSyMAP	Non-Operational	√	√	√		
VPRoMMS	Operational (some provinces only)			√	√	
RoadNAM	Non-Operational (license issues)				√	√
Bridge Management System (BMS)						
VBMS	Operational (Upgrading)	√	√	√	√**	√**
Bridge CV (Hard-Copy)	Operational	√	√	√		
Asset Valuation System						
National Asset Database	Operational	√	√	√	√	√
Note: * Technically operational if same quality of data are collected and used for PMS						
** Currently under upgrading to make applicable in provincial roads						
DRVN: Directorate for Roads of Vietnam		RMB: Road Management Bureau,				
PDOT: Provincial Department of Transport		PPC: Provincial People's Committee				

In the recent year, priority has given in development of bridge and pavement management system including database, however in the very short time, there are other several assets which also needs to be managed and maintained. The Decree No.10/2013/NĐ-CP promulgated on January 11, 2013 (The management and use of road transport infrastructure assets) recognizes a total of twenty-eight (28) road facilities as road asset which have to be properly maintained and asset values have to be estimated and reported to Ministry of Finance (MOF) annually. Recently, Government of Vietnam has established the Road Maintenance Fund (RMF) in order to secure sustainable maintenance fund to provide needed maintenance budget to the road operators. Decree No. 18/2012/NĐ-CP promulgated on March 13, 2012 (Road Maintenance Fund) guides the utilization of RMF. According to Decree No. 18, a systematic maintenance plan shall be prepared and submitted to RMF council for approval.

In this section, attempt is made to review RAMS established in the past in Vietnam. DRVN RAMS introduced in the past are briefly reviewed as below.

(1) Database System

So far DRVN has no comprehensive road database system which become fully operational. In the past, it was decided to use RosyBASE as official road database system in DRVN. However, it could not reach to operational level due to various circumstances including system related issues and data quality. Road data collected and inputted in the system in 2007 resulted to the waste of resources due to incapability of the system to control the quality of data while inputting.

(2) Highway Development and Management-4 (HDM-4)

HDM-4 is a software package exclusively developed for pavement management system. HDM-4 serves as the primary tool for the analysis, planning, management and appraisal of road maintenance, improvements and investment decisions. HDM-4 has four main areas of application; strategic planning, roadwork programming, project analysis and research and policy studies. In Vietnam, HDM-4 was introduced in 1998 in the first time to the national road network, since then, six trials had made from 2001 up to 2006 with assistance from the World Bank and Asian Development Bank. Some international consultants had conducted the analysis to prepare three-year work programs under technical assistance projects. HDM-4 is still in use fat project level specially for economic analysis of the projects funded by the World Bank.

Since HDM-4 has no dedicated database system to feed data into the system, it has to be rely on external database/dataset. In the past, DRVN had practiced to prepare HDM-4 dataset in MS-Excel format as well as converting from database management system such as RosyBASE. Pavement condition data were collected in different times were populated in MS-Excel format and imported in the system for running HDM-4. In 2007, DRVN (then Vietnam Road Authority; VRA) had decided to use HDM-4 as official planning system and RosyBASE as official database system for managing national road network in Vietnam. In order to integrate HDM-4 and RosyBASE as planning system and database respectively, an interface had developed to convert data from RosyBASE to formulate HDM-4 dataset. In 2007, when RosyBASE was decided as an official database system, VRA has collected necessary data by its own fund first time to input data in RosyBASE. However, due to poor quality of data inputted in RosyBASE, the data conversion interface could not work properly thereby arising system error. As a result, integration of HDM-4 and RosyBASE could not be successful which eventually forced to DRVN to abandon both systems and opt for alternative. Thus, HDM-4 could not be a management system for DRVN to manage entire road network till now.

(3) Rosy System

The Rosy system consists of four (4) components; RosyBASE, RosyPLAN, RosyMAP and RosyADMIN. A full set of Rosy was introduced to DRVN headquarter and four (4) Regional Management Bureau (RMB) in 2005. In 2007, RosyBASE was endorsed as official database system of DRVN. Under Special Assistance for Project Implementation Phase II (SAPI II) in 2008, Rosy's trial runs were conducted and revealed that unidentified errors caused the failure of trial runs. Due to termination of contract with Rosy supplier, system trouble could not fix at that time. Analytical function of the RosyPLAN customized for DRVN was presumed as same as the program analysis module of HDM-4. None of the Rosy system except

RosyBASE had used as official system for management of road network in Vietnam. Therefore, RosyPLAN, RosyMAP and RosyADMIN had not reached to operational level in DRVN in the past.

(4) Vietnam Pavement Management System (VPMS)

Since 2011 Vietnam Pavement Management System has been developed under JICA Technical Cooperation Project; The Project for Capacity Enhancement in Road Maintenance in Vietnam. In JICA Phase I Project implemented in the period from 2011.09 to 2014.04, VPMS had been developed for national road under jurisdiction of RMB I region as pilot region. JICA Phase II Project which is being implemented from February 2015 to upgrade the outputs of the JICA Phase I Project to make them applicable to the nationwide national road network in Vietnam. In JICA Technical Cooperation, DRVN is also involving actively in its development and technology transfer activities which have been conducting through seminar, workshop, on-the-job training and working group meeting.

VPMS has been developed taking consideration of lessons learned from past attempts of introduction of RAMS and their operational status in DRVN. VPMS framework was designed taking consideration of importance of dedicated PMS database, data quality assurance, management need of DRVN, technology transfer, user friendly operation, etc. VPMS has its own database which will provide necessary data to run it. As pavement condition data are key for pavement maintenance planning, pavement condition data collection vehicle has also been provided to DRVN to ensure the quality and consistency of pavement condition data. Therefore, in the framework of VPMS, it does not only consider the computer software for maintenance planning but also the data collection, dataset preparation, database management, displaying pavement condition data on the GIS map (for monitoring), and pavement data analysis system in the same all-in-one software package.

VPMS consists of three main modules, namely pavement deterioration evaluation module, strategic budget planning module and repair work planning (medium-term and annual plan) module. Based on the result of pavement deterioration evaluation module (i.e. pavement deterioration prediction), VPMS can formulate strategic budget plan, medium-term (5 year) repair work plan, and annual repair work plan.

(5) Vietnam Bridge Management System (VBMS)

VBMS has been developed under JICA Transport Sector Loan Project as a standalone system. Development of VBMS and bridge data collection and inputting was conducted under transport sector loan project. The bridge database operates in the web-based system and data are coded in GIS format as well. Currently Second Transport Sector Loan Project (Component B) is improving and applying VBMS for formulating bridge maintenance plans.

VBMS consists of four (4) modules, namely inventory module, bridge inspection module, planning module and system administration module. Inventory module manages inventory information, providing inventory information for the annual report. Bridge inspection module manages periodic inspection information, conditions and evaluation results of technical state. Similarly, planning module enables to evaluate investment scenarios by applying bridge condition index of bridge.

(6) Asset Valuation System

Ministry of Finance (MoF) has developed asset management system in accordance with Decree No. 52/2013/NĐ-CP promulgated on June 03, 2009 (Management and Use of State Properties). DRVN is also using the same system for road assets. Department of Finance (DoF) of DRVN is in charge of managing asset database on national roads under management of RMBs and Provincial Department of Transport (PDoTs). The unit asset value is defined by asset type, class of asset, classified type of asset and terrain. The asset value is stored in the Asset Valuation System, however, those data are manually calculated and does not have any function to calculate asset value by connecting to asset database. The value of the asset constructed before 1st March 2013 was calculated by unit price per kilometer which depends on road class, terrain type, etc.

(7) Other Management System

There are no other management systems exist in DRVN to manage road assets other than pavement and bridge neither in the past nor at present.

3. LESSON LEARNED FROM THE PAST INITIATIVES

In the past, road network data were collected by different departments without integration and rigid quality control due to absence of systematic road network data collection and evaluation procedure. Past attempts to introduce RAMS in DRVN were not successful mainly due to institutional issues and the complexity of the proposed systems. Also it has been pointed out that past technical assistances from the donors have been focused on providing technology and equipment, rather than human capacity development and its process. Through the analysis of failure of the past attempts in data collection and introduction of road asset management system in DRVN the following failure reasons are identified:

- Absence of database in asset management system; database is a base for asset management system
- Lack of clear responsibilities assignment among DRVN departments
- Lack of trained human resources
- Lack of adequate budget for data collection and database management
- Lack of ownership (none of department took the ownership)

These all problems analysis and efforts for improvement were made under JICA SAPI Study (Phase I and Phase II). As a recommendation of SAPI Study, JICA and DRVN agreed to develop new VBMS and VPMS including database. As a result, with assistance from JICA, development of VBMS and bridge data collection and inputting have been carried out under transport sector loan project. Similarly, development of database framework, pavement management system and pavement condition data collection of RMB I have been carried out by JICA Phase I and Phase II Projects in collaboration with DRVN.

Taking the past lessons and ongoing development initiatives by DRVN into account, a new RAMS by integrating existing VBMS and VPMS is now under development under Vietnam Road Asset Management Project (VRAMP) with assistance from the World Bank. A new RAMS is being developed by paying attention varying from fundamentals of asset management system to its long lasting sustainability, as well as to capacity building / technology transfer in the overall cycle of road asset management.

4. FRAMEWORK FOR DEVELOPMENT OF A NEW CUSTOMIZED RAMS IN VIETNAM

As of December 2014, the total length of road network in Vietnam has reached to 309,832 km, of which national road is 21,109 km, provincial road is 26,218 km, district road is 53,299 km, commune road is 178,294 km, urban road is 20,076 km and exclusive road is 10,836 km. Currently, roads are managed by Directorate for Roads of Vietnam (delegated to RMBs), Provincial Department of Transport, Build Operate and Transfer (BOT) operators. With the increase in road length, the challenges in their maintenance and management have also increases due to budget constraint in road maintenance. Therefore, the systematic and scientific decision making tools is needed for road operators to allocate the maintenance budget efficiently in order to ensure safe and smooth traffic operation.

Thus, framework for development of a new customized RAMS is developed by reviewing of all concerned regulatory provisions, assessing needs of each concerned institution, capacity development of human resources and taking account of lesson learned from the past initiatives. The following framework are considered to develop customized RAMS in DRVN.

(1) Asset Database

Road asset database is a major components of DRVN RAMS. Therefore, database framework is prepared by thoroughly reviewing regulatory provisions (including data required as per Decree 10/2013/NĐ-CP and Decree 18/2012/NĐ-CP), data necessity of all concerned organizations and data requirements for management systems such as VPMS, VBMS, drainage management system and other asset management system. A total of 39 road assets are considered as road assets to include in the road asset database framework. In addition to these road asset specific data, maintenance history and condition data of major road assets, traffic volume data, road administration data, traffic accident data, etc. are also included. Since many organizations from central to field levels are the prospective user of road asset database, the number of

data items is comparatively high. Therefore, in order to optimize the resources effectively and efficiently, data collection priority has been defined.

Table 2. List of Assets included in Road Asset Database

Asset / Data Group	Asset Group	No.	Asset Name	Data Input Priority	Criteria for Data Collection Priority				
					Stipulated in Decree 10	Urgency for formulating maintenance plan	Directly Related to Road Traffic Operation (Facility in road surface)	Road Side Facilities which affect on road traffic operation and surrounding environment	
General Information	-	1	Road Administration Data	I	-	○	○	-	
	-	2	Road Main Details	I	-	○	○	-	
	-	3	Road Overlap Section	I	-	○	○	-	
	-	4	International (Asian/Asean) Highways	I	-	○	○	-	
Road Transport Infrastructure Assets (Fixed Assets)	I	Main Structures	5	Road (Pavement)	I	○	○	○	-
			6	Bridge	I	○	○	○	-
			7	Road Tunnel	II	○	-	○	-
			8	Ferry Terminals	II	○	-	○	-
			9	Underpass (Box culvert)	II	-	-	○	-
	II	Slope Works	10	Retaining Wall	II	○	○	-	○
			11	Road Slope (Protection Work)	III	○	-	-	○
	III	Drainage Work	12	Box Culvert	I	○	○	○	○
			13	Pipe Culvert	I	○	○	○	○
			14	Roadside Drain	I	○	○	-	○
			15	Manhole	I	-	○	○	○
			16	Drainage on the Road Slope	II	○	○	-	○
			17	Spillway / Causeway	I	○	○	○	-
	IV	Crossings	18	Road Intersection	II	○	○	○	-
			19	Railway Crossing	II	-	○	○	-
			20	Pedestrian Crossing Bridge	III	-	-	-	○
	V	ITS & Fee Collection	21	Intellegent Transport System (ITS)	I	-	-	○	-
			22	Weigh Stations	II	○	-	○	-
			23	Toll Booths	II	○	-	○	-
	VI	Road Side Work for Traffic Safety	24	Traffic Lights	II	○	○	○	-
			25	Road Lighting	III	-	-	○	-
			26	Road Signs	III	○	-	○	-
			27	Road Surface Marking	II	○	○	○	-
			28	Guardrails / Guide Posts	III	○	○	-	○
			29	Barrier	III	○	○	-	○
			30	Road Separators (Median Strip)	III	○	○	-	-
			31	Milestones (kilo post)	III	○	○	○	-
			32	Noise Barrier	IV	-	-	-	○
			33	Shade Fence	IV	-	-	-	-
	VII	Building and Service Area	34	Parking Lots / Rest Stops / Service Area	III	○	-	○	-
			35	Bus Stops	III	○	-	○	-
			36	Bus Terminals	III	○	-	○	-
			37	Road Disaster Response Facility	I	-	-	-	-
38			Road Administration Office	II	○	○	○	-	
VIII	Land and Utilities	39	Utilities within ROW	IV	-	-	-	○	
		40	Land belongs to Road Asset	IV	○	-	-	○	
O&M Vehicle and Equipment	IX	Movable Equipment	41	Highway Inspection Vehicles	I	-	-	-	
			42	Rescue Vehicle/Equipment	I	-	-	-	
			43	Ferries	I	○	-	-	
Traffic Volume	-	44	Traffic volume	I	-	○	○		
Asset Inspection (Condition) Data	-	45	Pavement, Bridge, Drainage, etc.	I	-	○	○		
Asset Maintenance/Repair History Data	-	46	Pavement, Bridge, Drainage, etc.	I	-	○	○		
Traffic Safety & Management	-	47	Traffic Accident Record Data	II	-	-	-		
		48	Black Spots	II	-	-	-		

Road asset database is designed for web-based and cloud computing technology. The proper attentions have been paid for web-security, load balancing, data backup system, etc. The database will be relational among related databases, connected with GIS maps and linked with state asset database. Figure 2 shows the conceptual image of road asset database structure.

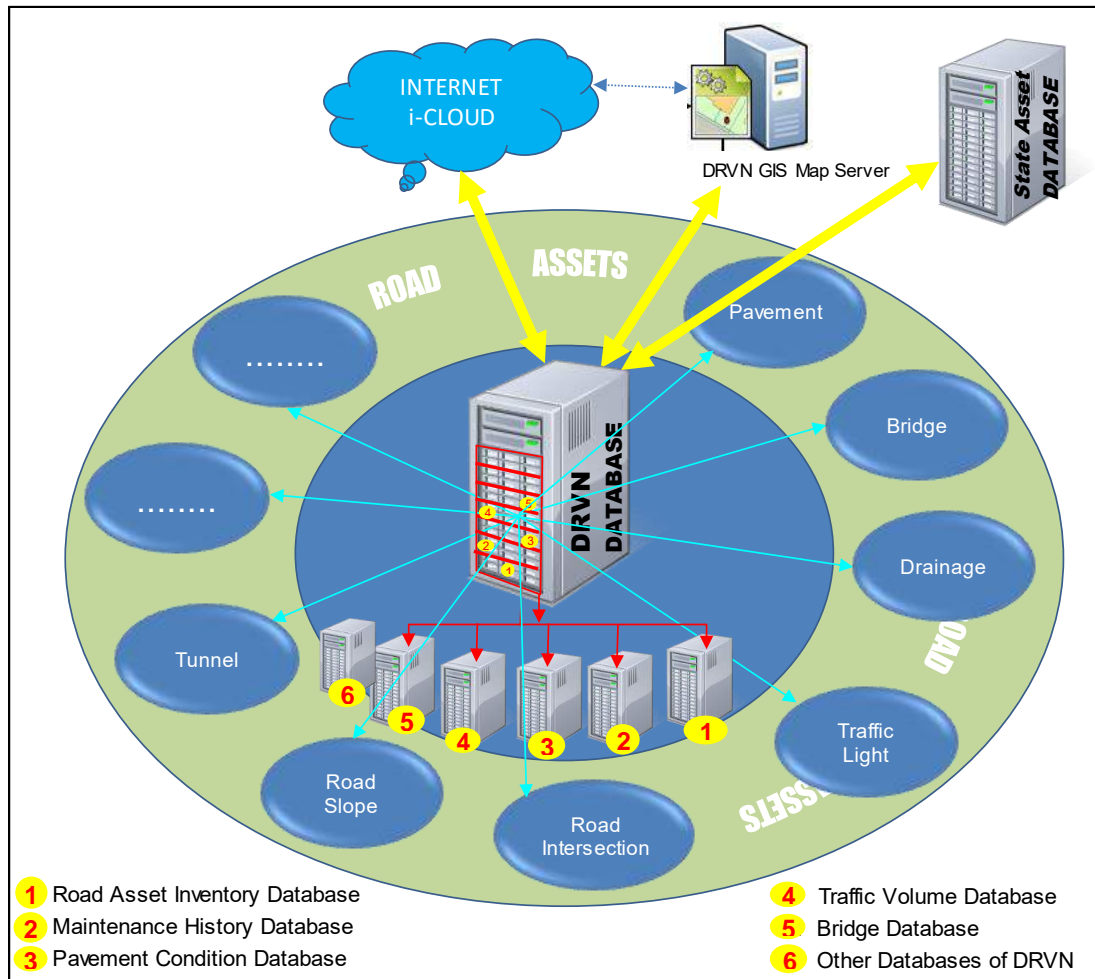


Figure 2. Conceptual Image of Road Asset Database Structure

(2) Road Asset Management System (RAMS)

Taking consideration of requirements of DRVN and lessons learned from the past attempts of introduction of RAMS and their current operational status in DRVN, the following key criteria are applied in selecting RAMS for DRVN.

Table 3. Key Criteria for Selection of RAMS for DRVN

1. Availability of In-built Database	6. Optimization of Currently Available Resources in DRVN
2. Availability of Management Functions Required for DRVN	7. Compatibility to Cloud Computing and Web-based Technology
3. Possibility of Integrating Different Management System	8. Availability of External Technical Support for System Maintenance and Management
4. Possibility of Customization into Vietnamese Context	9. User Friendly Operation
5. Possibility of Editing and Updating the System	

RAMS that had been attempted to introduce in the past and Commercial-Off-The-Shelf (COTS) were evaluated with the above key criteria and decision was made to develop a new customized RAMS by integrating VPMS and VBMS which are under development. RAMS is developed in a flexible platform using local IT consultant so that it can be updated whenever necessary. The components of DRVN RAMS are shown in Figure 3.

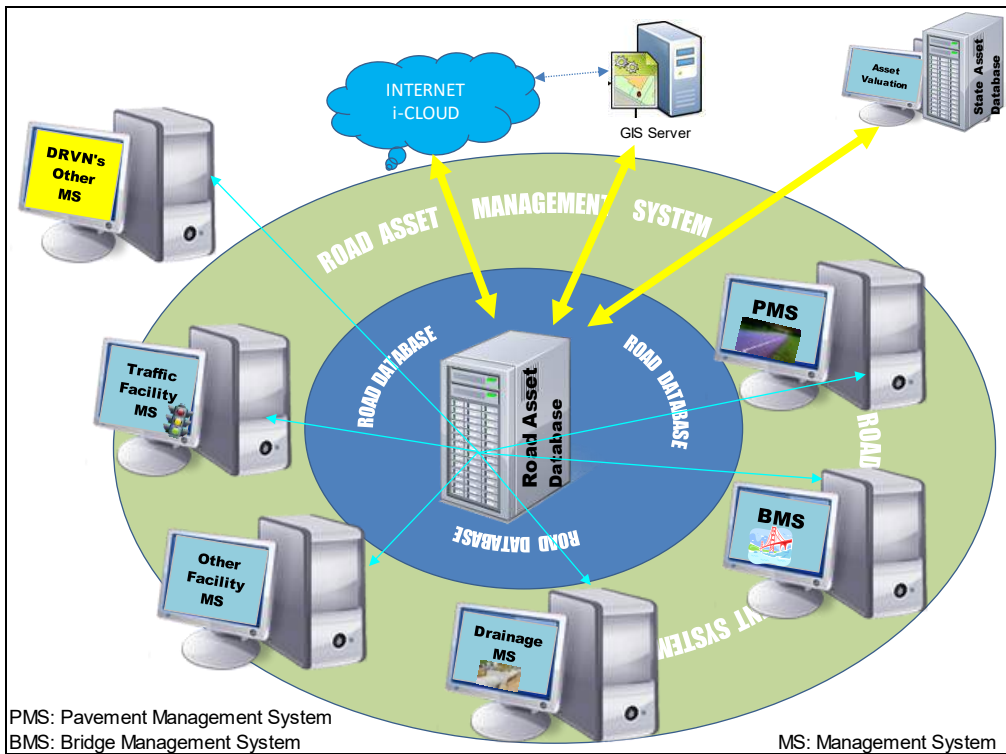


Figure 3. Components of Vietnam Road Asset Management System

(i) Bridge Management System

Vietnam Bridge Management System (VBMS) which has developed under Transport Sector Loan (TSL) Project and being upgraded is to be integrated into the DRVN RAMS. VBMS has mainly four modules, namely administration, inventory, inspection, and planning modules. VBMS is already in operation under DRVN and PDOTs. VBMS has also been integrated with GIS map. Bridge Condition Index (average and critical; BCI_{avg} and BCI_{crit}) is used as a key index for bridge maintenance planning. The maintenance planning flowchart is shown in Figure 4.

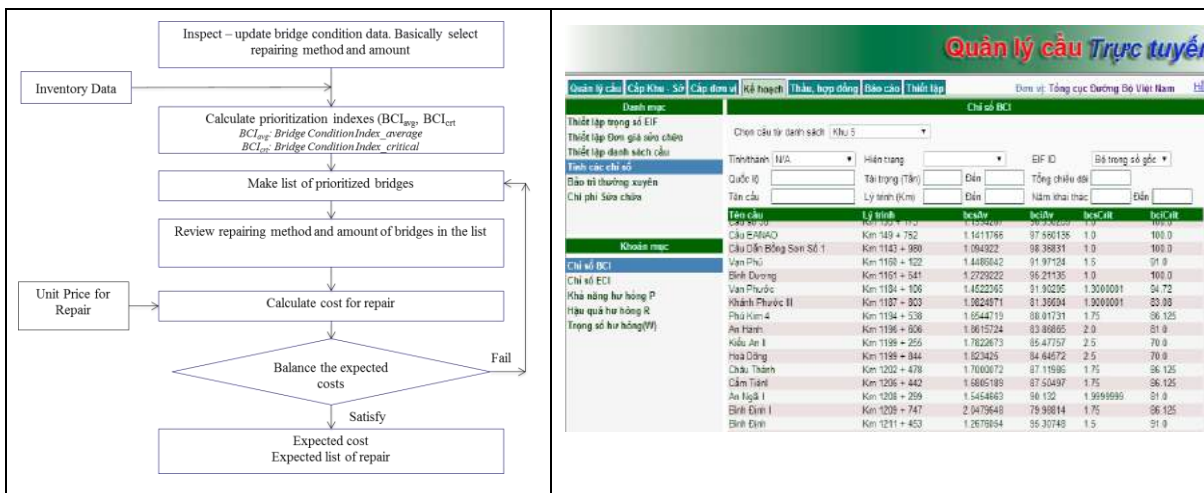


Figure 4. Maintenance Planning Flowchart and Sample Output (approx. 4,300 bridges)

(ii) Pavement Management System

Vietnam Pavement Management System (VPMS) which is being upgraded under JICA Phase II Project in collaboration with DRVN is to be integrated. VPMS is being developed with its own PMS database and the pavement condition data are to be collected by data collection vehicle provided by JICA for assuring data consistency and accuracy. VPMS can formulate a strategic budget plan (long term) and a medium to short term budget plan (annual and 5-year) based on pavement deterioration speed. Pavement deterioration evaluation and strategic budget simulation modules are developed by importation from Kyoto model; developed by Kyoto University, Japan. Pavement deterioration evaluation module is developed using markov transitional probability theory (hazard model). PMS has three main modules, namely pavement deterioration evaluation module, strategic budget simulation module and repair work planning module. Budget simulation can be performed by three scenarios; maintain current budget level (worst scenario), maintain current pavement condition level and maintain target management level (best scenario). Similarly, repair work planning module assist in formulating annual and medium term repair work planning by due consideration of deterioration speed. Some outputs of VPMS are shown in Figure 5.

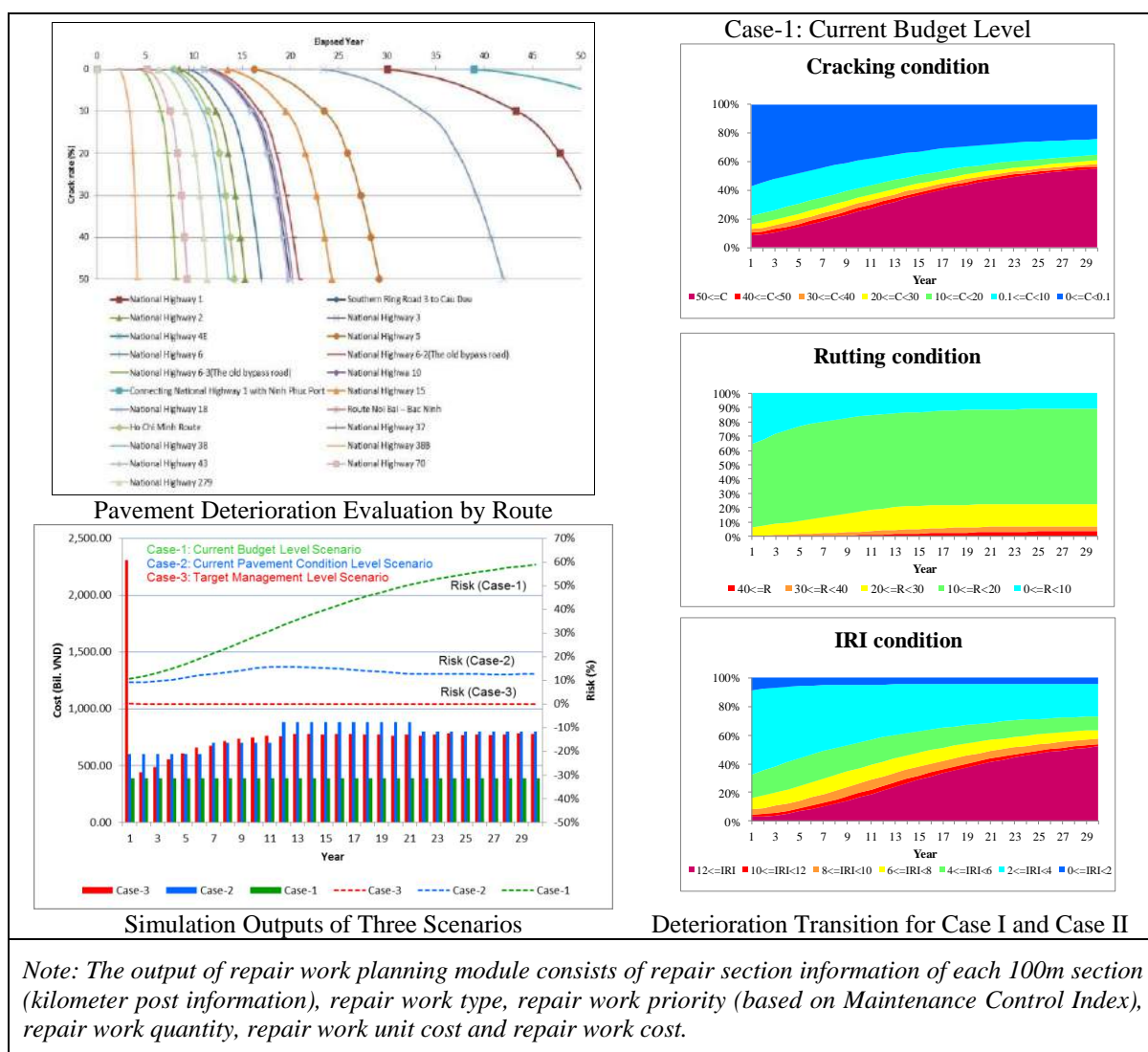


Figure 5. Output of VPMS run for National Road Network (approx. 4,700km) under RMB I Jurisdiction

(iii) Drainage Management System (DMS)

Road drainage system is one of the important factors in deteriorating pavement condition and surrounding environment. Though it does not directly relate to traffic operation, the improper management of drainage system may adversely effect on pavement layers including sub-grade, shoulder erosion, inundation

on road surface, etc. Unlike pavement deterioration forecasting, deterioration of the drainage system cannot be predicted. Therefore, drainage inventory and inspection data should be utilized for maintenance planning of drainage system. The following three main modules are considered for development of the drainage management system.

a. Inventory Module

The inventory module manages the inventory data related to the drainage system. The inventory data includes data of cross-sectional drainage system (box culvert, pipe culvert), longitudinal drainage system (road side drain), slope drainage system (catch drain), and tunnel drainage system.

b. Inspection Module

As inspection data will be the main data for maintenance planning of drainage system, the inspection module will manage the inspection data and the result of diagnosis (evaluation). Inspection items and evaluation method will be followed as provisioned in the Road Facility Inspection Guidelines which is being drafted under JICA Phase II Project in collaboration with DRVN.

c. Planning Module

The planning module will assist in preparing a maintenance plan for the drainage system by utilizing the inspection evaluation results. The planning module will produce a maintenance plan by consideration of severity of damage (damage grade B, C, D), frequency of flood incidents, importance of road sections, etc. Prioritization method should be considered in the RAMS development.

(iv) Other Asset Management System

The management system for other assets, whose deterioration prediction cannot be estimated, similar management system as recommended for drainage system will be applied. Therefore, inventory data and inspection data will be the major data requirements. The other asset management system can be developed in DRVN RAMS as necessary with the same approach as described for drainage management system.

(v) Asset Valuation System

Asset valuation system developed by MoF is to be integrated into RAMS.

(3) Data Collection Plan

Since DRVN is developing this scale of RAMS first time and past data are neither sufficient nor fully consistent with the new RAMS database framework, data collection plan for feeding data in the system are also planned. For the short term, a multi-year data collection plan is prepared to collect inventory and condition data. Once inventory data are collected and stored in the system, annual update is necessary whenever there will be a big scale repair work and new construction. Since condition of pavement and bridges will be deteriorated due to several reasons including traffic loading, a periodic condition data collection is necessary. Therefore, such periodic data collection plans are also prepared. Based on the multi-year data collection plan, data collection work is ongoing and planned to complete within next 3 years.

(4) Capacity Building / Trainings

RAMS is a complex system and users are different level varying from asset maintenance planner to field level engineer for traffic operation and management. The sustainability of the system greatly depends on the effective technology transfer during the system development phase. Therefore, attention has been paid in effective technology transfer to all level of users. Training plan is prepared for “during the project” and “after the project”. During project, seminar, workshop and On-the-Job-Training (OJT) both by using computer as well field visit are planned. The courses are designed for professionals as well as for Trainer of the Training (TOT). Training materials for all training courses are to be prepared and provided to all concerned organizations and officials.

6. CONCLUSIONS

Investment in road transport infrastructures is huge and for several decades. There is direct relationship between road transport infrastructures and socio-economic activities in the regional scale. Therefore, if road transportation system does not perform properly, there are multiple effects in various fields. Therefore, preservation of road transport infrastructures is very much important not only for preservation of road facilities but also maintain the chain of socio-economic activities. Since road network in Vietnam is expanding rapidly, many of the aged structures needs to be maintained properly and past attempts of introduction of RAMS were not successful due to various circumstances, development of a customized RAMS to make it more sustainable is necessary. Under Vietnam Road Asset Management Project funded by the World Bank (Component A-1), an integrated RAMS is designed to develop locally by customizing the foreign practices into Vietnam. RAMS under development consists of many components of RAMS including a comprehensive web-based road asset database, VBMS, VPMS, DMS, multi-year data collection plan and data collection provision and effective training programs. It is anticipated that upon completion of development of RAMS, it will be a milestone for management of road transport infrastructures effectively and efficiently in Vietnam. Also, it is expected that this approach of RAMS development will be a model for development of RAMS in Southeast Asia.

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**The Influence of Trips Generated From New Projects Development
to the Efficiency of the Road Network**

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Abstract

Urbanization has a significant impact on the future performance of surrounding road network. The Municipalities or Local Authorities who plan, manage and approve future land use, play a crucial role to address the impact of urban growth, which causing severe congestion onto the current road network. Policies and tools used for new development traffic impact evaluation are critical to determine the performance of road network for a more balanced and sustainable environment in urban area to be achieved. Thus, the present approach of approving new development project by Local Authority in relation to traffic needs is to be revisited. Trip generation of every proposed land use activities need to be evaluated on its impact to current road network capacity. The traffic evaluation should not merely focus on the proposed development site, rather it should be explored to improve surrounding impacted road network for more comprehensive solution surrounding vicinity. This paper trigger the need for new approach or better policies to ensure urban growth and road network capacity can be equally prioritised.

Keywords: Land use planning, trips generation and attraction, congestion, capacity and efficiency of road network.

Introduction

Currently, rapid development occurring along the main road network is an undeniable trend in urban area. In fact, high accessibility to main road network became the most significant reason for new development to keep encroaching to available nearby land. In the developer's perspective, every new development will be exploiting the advantage of current road network capacity towards making the most profit by having more complex development including mix used of high rise building.

However, through continuous urbanization process, the generated traffic from complex land use activities has resulted road network capacity reached to its maximum. The more traffic induced will definitely deteriorate road network performance. Thus resulting in human tension, wastage of time on road, high cost of transport and the worst, it contradicts our mission to achieve a low carbon city.

Thus, this paper focus on investigating how trips generated from new developments can influence the road capacity and road performance. In order to do this a pilot study was carried out to compare and observe the before and after situation in the Subang Jaya and Puchong Area.

Methodology

The methodology used in an effort to match the development of an area to the road capacity is as shown in four stages below:

- Stage 1: A study area with a very high rate of development was chosen
- Stage 2: Analyse satellite images of 2 different years 2006 and 2016 to look at the urbanization process and how it effect to road network.
- Stage 3: Study of the road network and its capacity
- Stage 4: General Trip generation evaluation

Nevertheless the fourth stage of computing the trips generated from the development was not carried out at this stage.

The municipality area chosen for a pilot study were Sunway, Subang Jaya and Puchong. The two areas are adjacent to one other and are flanked by a network of non-tolled and tolled urban expressways. Figure 1 shows the study area.

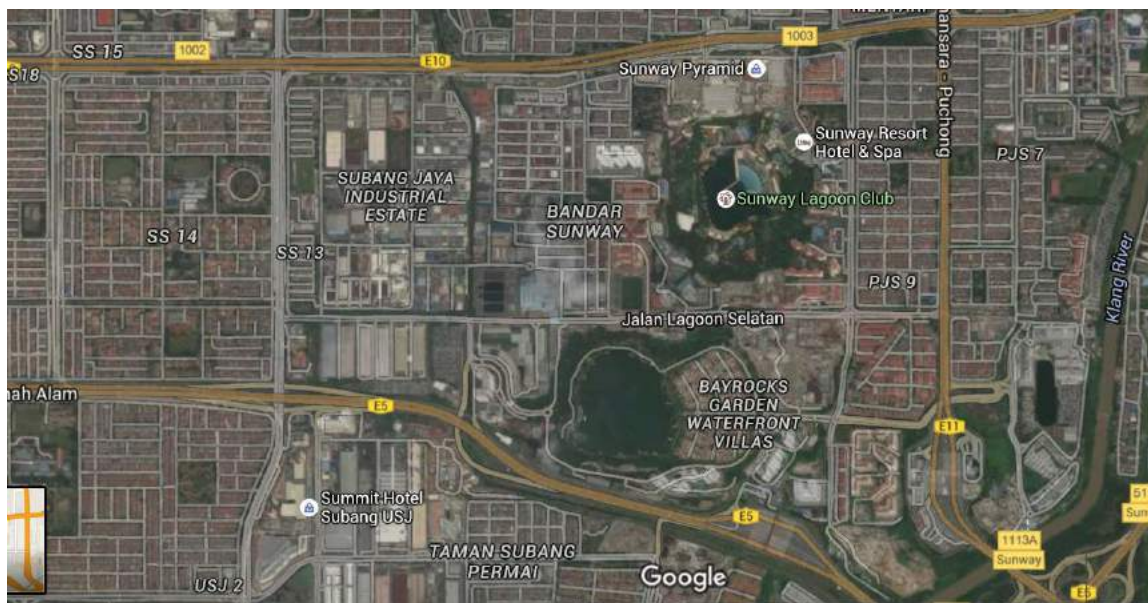


Figure 1: Map of Subang Jaya and Puchong Area
(Source Google Maps)

Both Subang Jaya and Puchong has a total area of approximately 70sq.km and 51.7sq.km respectively. Both these areas are quite similar in terms of development. The land use makes up of residential, industrial, commercial, schools and higher education, with amenities like hospitals, shopping malls, hypermarkets and many others. The ready catchment of 500,000 population within Bandar Sunway and Subang Jaya including 50,000 students coupled with the 40 million visitations per annum attests to the existing high rate of the development in the area.



Photo1: Picture of the highly developed Sunway, Subang Jaya Area

The two areas are serviced by the earlier mentioned highways which are the KESAS Highway (E5), the New Pantai Expressway (E10), the Damansara–Puchong Highway, LDP (E11) and the untolled Federal Route 2 from Klang to Kuala Lumpur. Even today, the area is still being developed, years after all the above road network has been in operation.

Analysis of Satellite Images

Satellite images of the said area were obtained from the Remote Sensing Department to see the difference in development that has occurred over the last 10 years, between 2006 and 2016. Figure 3 shows the difference in both the land use pattern and the road network. At this stage the type of land use and the computation of the trips generated was not done. However it is obvious that development in the area has increased, thus resulting in increased traffic. It can be seen that the expansion at the interchanges are also carried out. These are examples of road infrastructure improvement carried out either by the Government or by the developer themselves. However if it was done by the Government, then it is only appropriate for the developer to share the expenses on the construction of the road expansion. There is an existing policy in place where, the developers are required to pay for any intersection improvement but not on the road links. Figure 4 shows similar change that can be seen between land use and the road network.

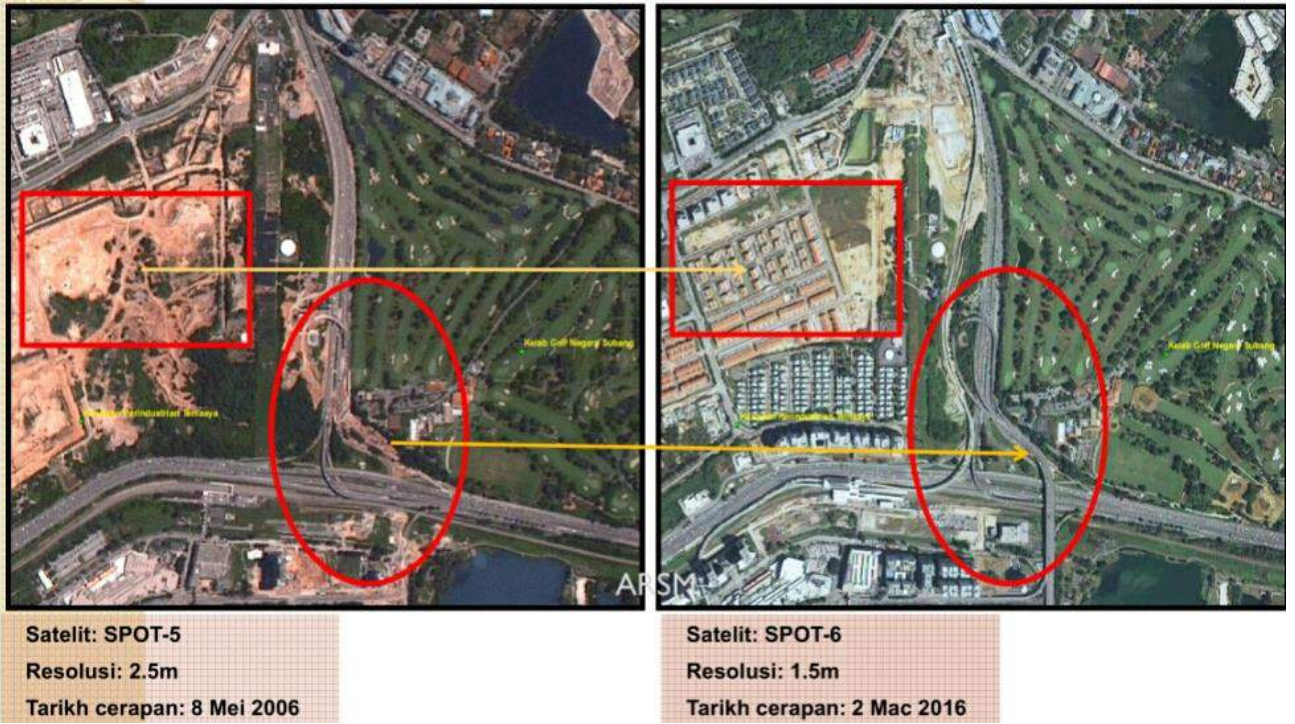


Figure 3: Change in Land use pattern between 2006 and 2016

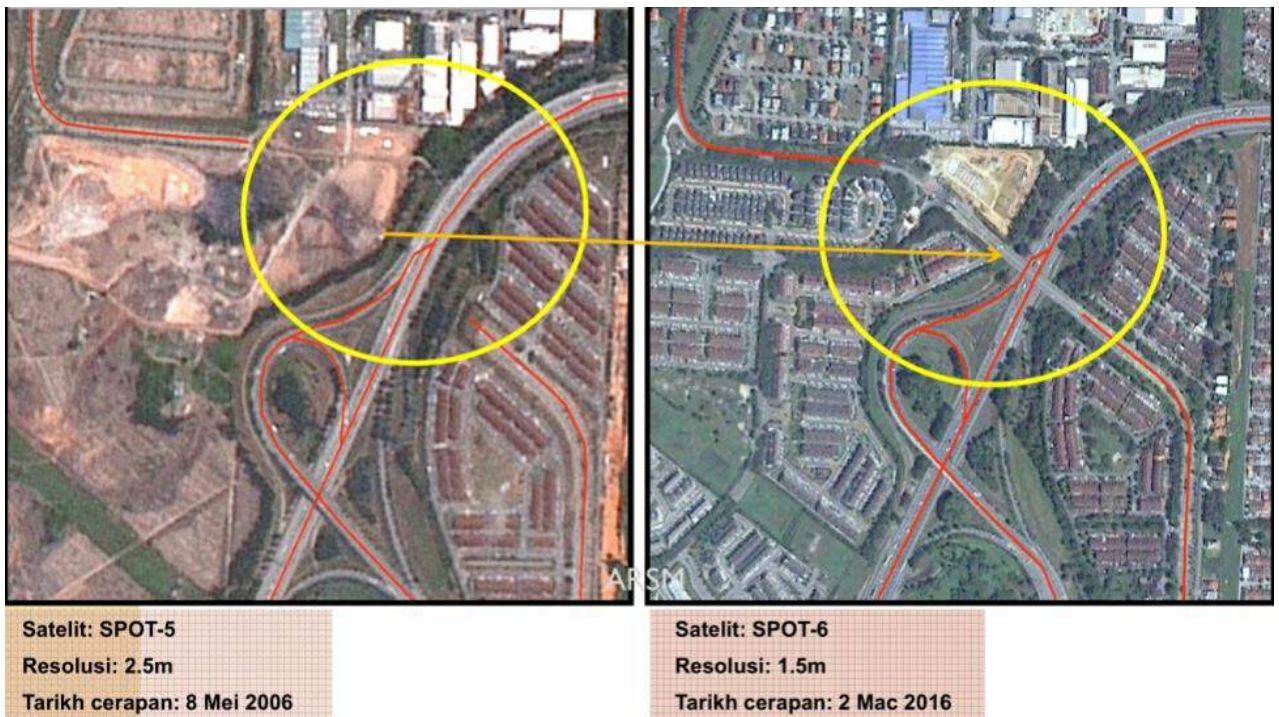


Figure 4: Change in Land Use Pattern and Road Network

However, there are many cases where the new developments are approved without any improvement to the road infrastructure. Cases where the Level of Service (LOS) has almost reached capacity, yet the Municipalities or Authorities concerned are approving projects without the traffic analysis of the road network in the vicinity. For large developments, analysing using traffic models should be imposed onto the developers for introducing new traffic onto existing highways. There are also instances where upgrading of the existing roads are simply too costly, therefore there must be a limit to the scale of project development that can be approved.

Road Network Analysis

In gathering evidence to show that there is an increase in traffic volume, the traffic along the New Pantai Expressway (E10) and LDP (E11) was used. The traffic over the 8 years more than doubled that in 2006. Similarly there is also an increase on the LDP, more than double in 6 years.

Table 1: Average Daily Traffic on NPE at Pantai Dalam (E10)
(Source: Road Traffic Volume Malaysia, HPU, KKR)

	2006	2014
Class 1	37,564	95,311
Class 2	371	950
Class 3	166	651
Class 4	2233	4,319
Class 5	135	284
Total	40,460	101,516

Table 2: Average Daily Traffic on LDP at Pencala (E11)
(Source: Road Traffic Volume Malaysia, HPU, KKR)

	2006	2014
Class 1	45,291	127,802
Class 2	1,263	1,905
Class 3	-	890
Class 4	1,313	3,517
Class 5	54	302
Total	47,922	134,416

Thus there is a relationship between development and traffic increase. In depth study on the trips generated is a pre requisite before any approval is given. For big developments a traffic modelling should be insisted, not simply a Traffic Impact Assessment study will suffice.

Figure 5 shows if not all, some of the amenities in the Puchong area. Even today there are still new development in the Puchong area despite the fact that the LDP (E11) is congested. Photo 2 shows the congestion near the toll booth on LDP.

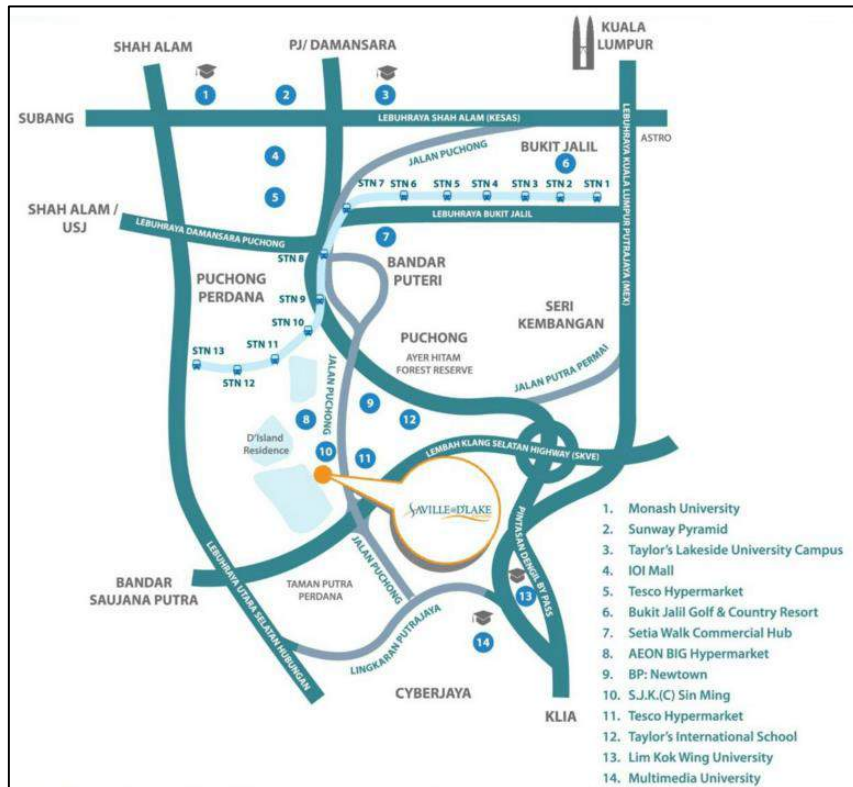


Figure 5: Amenities in the Puchong Area



Photo 2: Congestion on the Damansara-Puchong Highway (LDP)

the traffic woes. There seem to be no control on the development in Puchong especially the residential land use. Condominium after condominiums have been approved, without the Municipality nor the Federal Government charging on the use of existing roads not towards the expenditure to expand or maintain the roads.

Typically, trip generation analysis focuses on residences and residential trip generation is thought of as a function of the social and economic attributes of households. At the level of the traffic analysis zone, residential land uses "produce" or generate trips. Traffic analysis zones are also destinations of trips, trip attractors. The analysis of attractors focuses on nonresidential land uses.

There must also be a clear definition of land use categories for trip generation purposes. In Malaysia, the Highway Planning Division in the Ministry of Works has published The Trip Generation Manual to assist the planners and engineers to predict traffic from new developments to see if the infrastructure is adequate. The approving Authority must also have a clear understanding of what kind of land use development should they approve if the surrounding infrastructure has already reached its network performance without any chance of expansion. Land use change – acreage of land use for housing, industry and commercial may have to be re-studied.

Conclusion & recommendation

The municipalities or the road authorities must not be the only parties responsible in providing a smooth, comfortable and safe road facility as means of land transportation. The need to maintain road capacity by applying traffic threshold to ensure better road network performance is required. This responsibility must be shared by the developers using the road network in the vicinity as access. The possibility of traffic charges induced be chargeable to the developers, not only at intersections or access but through the use of the link itself, should be made possible with careful study and guidelines should be developed.

Traffic evaluation process for new development project need to be revisited by stressing on road network capacity and the implementation of traffic charge based on trip generation as continuous road network improvement need to be implemented.

Acknowledgement

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PAPER TITLE	Operators' and Users' Perspective on Service Quality Dimensions of Toll Road PPP		
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KEYWORDS:

PPP, service quality, toll road

ABSTRACT:

Toll road is one of the most important infrastructures in supporting population and economic growth in urban areas. Under a toll road PPP, consentient perspectives of operators and users on service quality become important to provide excellent services with reasonable price. This paper aims to identify dimensions and attributes of toll road service quality based on both perspectives. 17 attributes were analyzed and divided into seven dimensions based on TRSQ model, to better represent every aspect of toll road service quality. The survey was conducted on 2015 in 33 Indonesian toll roads. Using Importance Performance Analysis, both operators and users agree that information, accessibility, reliability, and safety & security dimensions should be prioritized. Toll road users, however, contended that mobility, which contains an attribute of travel time, is also important for toll road performance. Furthermore, these dimensions can be used to measure toll road service pricing.

Operators' and Users' Perspective on Service Quality Dimensions of Toll Road PPP

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1 INTRODUCTION

Public Private Partnership (PPP) is a new approach in providing public infrastructures, as the government tends to have limited source of funds and private sectors are able to provide them. International Road Federation or IRF (2016) defined PPP as “Alternative Financing and Procurement to maximize risk transfer and achieve value for money in the delivery of large, complex projects”. Projects under PPP’s ownership remain with the public sector, but the funding could be combination of public and/or private.

Toll road projects in Indonesia run under PPP, where the government makes toll roads plan, and private sectors are responsible to build, operate, and maintain the toll roads during concession. As the operator, private sectors have their own point of view of their toll road performance. These are often different from what users think of toll road services they perceived. On the other hand, the government as a regulator must set toll road service quality standards for toll road operators to meet.

Toll roads are built as alternatives for road users which provide better services, mainly shorter travel time. Zuna (2015) mentioned two main indicators for toll road performance, namely physical condition and travel time. For those services users must pay for toll fare, therefore they naturally have expectations of toll road service quality. In experiencing journey on toll roads, users have their own point of view of how toll road service quality should be whereas toll road operators also have their perspectives. It is the government’s job to set toll road service quality standards that meet both operator’s and user’s perspectives.

This paper aims to determine toll road service quality dimensions and attributes by comparing toll road operator’s and user’s perspectives. These dimensions can be used by the government as consideration to set standards for toll road and service pricing.

2 LITERATURE REVIEW

2.1 HISTORY OF PPP IN INDONESIA

The first toll road in Indonesia was built in 1978, funded by both government’s budget and foreign loan which granted to PT. Jasa Marga (Persero) Tbk, a government-owned company, as a paid-up capital. The toll road is 59 km long, connects Jakarta, Bogor, and Ciawi, and it is called Jagorawi Toll Road. Private sectors have been participating in toll road provision since 1987 as toll road operators. In 1997, 553 km toll road had been built, 418 km of them were operated by PT. Jasa Marga, and the remaining 135 km by private sectors. Furthermore, only 13,3 km of toll roads were built during 1997-2001 due to monetary crisis, despite 19 toll road segments had been carried out through tender. In 2005, Indonesia Toll Road Authority was established as the toll road regulator in Indonesia, and the construction of 19 toll roads was recommenced (Indonesia Toll Road Authority, 2014).

Hereinafter, Indonesian government is funding toll road constructions through three schemes, namely private full-financing scheme, Public-Private Partnership (PPP), and government-funded-construction with private-funded-operation and maintenance scheme.

2.2 INVESTMENT SCHEMES

Indonesia Toll Road Authority (2014) formed three investment schemes, they include government and business entity as stakeholders. The first scheme shows that the government conducts land acquisition and construction of toll road, and business entities are responsible for toll road operation and maintenance. The second scheme shows that land acquisition and construction of toll road are conducted by both government and business entities, while toll road operation and maintenance are conducted only by business entities. The last scheme shows that business entities

are responsible for the whole project, from land acquisition, construction, operation, and maintenance of toll road.

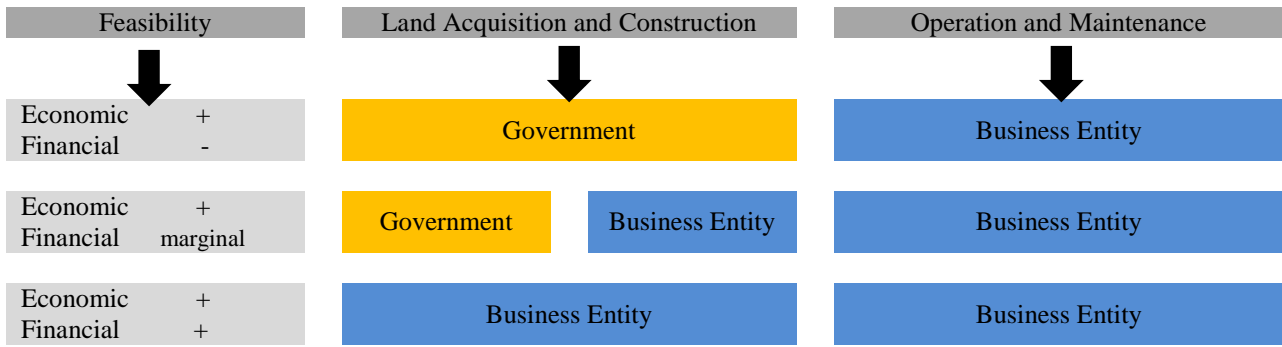


Figure 1. Investment Scheme (Indonesia Toll Road Authority, 2014)

2.3 SERVICE DOMINANT LOGIC

Vargo and Lusch (2004) introduced a new perspective of marketing, called Service Dominant Logic (SDL). In SDL, the aims of economic and marketing activities are services, while goods, money, and organization are intermediaries of service exchange. SDL was developed from Good Dominant Logic (GDL), where marketing is product oriented. Lusch (in Dakael, 2009) mentioned two types of resources, namely operand resource and operant resource. Operand resource includes machines, products, office buildings, and other objects, while operant resource includes human resources.

As mentioned, marketing used to be product oriented and focused on operand resources (Good Dominant Logic). The products distributed must deliver benefits, profits, efficient, and standardized. On the other hand, SDL sees customers as active participants in transaction process and as co-producer. Goods are intermediary used by customers to create value. Based on this perception, interactions between producers and customers are considered important, and marketing is focused more on service and the process instead of the goods distributed.

Since SDL, service quality and customer's perception are considered important in marketing. In economic activities, producers must focus on things more than just goods.

2.4 SERVICE QUALITY ON TOLL ROAD AND TRANSPORTATION SECTOR

In road sector, specifically toll road, service is one important aspect that must be considered by toll road operator. The government sets the standard for toll road service quality that the operator must meet, in order to reach users satisfaction. That is why in setting toll road service quality standards, user's point of view must be considered. SERVQUAL model by Parasuraman et al (1988) mentioned five dimensions to measure service quality, namely Reliability, Assurance, Tangibles, Empathy, and Responsiveness. This concept has been adapted in several research about service quality, including transportation sector. Randheer (2011) used SERVQUAL as the basic concept in measuring user's satisfaction on public transportation. Randheer used SERVQUAL dimensions and one additional dimension namely culture in his research. Castillo and Benitez (2012) mentioned eight dimensions to measure service quality of public transportation, for evaluating user satisfaction. Those dimensions are (1) Connectivity, (2) Accessibility, (3) Information, (4) Time Satisfaction, (5) User Attendance, (6) Comfort, (7) Safety/Security, (8) Environmental Impact. Maruvada and Bellamkonda (2012) tried to analyze main factors influencing service quality of railways in India, and they developed RAILQUAL model with six service quality dimensions. Those are (1) reservation and ticketing, (2) railway platform amenities, (3) in-train-service, (4) employee service, (5) punctuality, (6) safety and security in the journey.

Furthermore, Zuna et al (2016) conducted a research in toll road service quality model using user's perspective. SERVQUAL was used as the basic theory, it was compared to Minimum Service Standards for Toll Roads in Indonesia. The research delivered a service quality model for toll road, called Toll Road Service Quality (TRSQ) Model. It contains seven dimensions, namely information, accessibility, reliability, mobility, safety & security, rest area, and responsiveness.

This paper uses TRSQ model for toll road service quality attributes grouping.

3 DATA COLLECTING AND RESEARCH METHODS

The data used in this research is toll road users' satisfaction survey conducted by Indonesia Toll Road Authority, Ministry of Public Works and Housing, in 2015. There are 3.400 respondents from toll road operators and toll road users of 33 toll roads in Indonesia. Those respondents are divided into two groups, user and operator. There are 17 toll road attributes on the questionnaire, and the respondents were asked to evaluate every attribute in terms of importance and performance using 1-5 likert scale. This research aims to identify toll road service quality dimensions based on both perspective, including their differences and similarities. The 17 attributes are shown on table 1 below.

Table 1. Research Attributes

Attributes
(1) Toll road information
(2) Access road
(3) Entrance information board
(4) Entrance transaction
(5) Toll road condition
(6) Information board on toll road
(7) Travel time
(8) Accident handling
(9) Parking lot
(10) Toilet
(11) Gas station
(12) Restaurants
(13) Exit transaction
(14) Signage
(15) Tariff
(16) City access
(17) Response to complains

This paper uses Importance-Performance Analysis (IPA) with slight modification. The aim of this paper is to determine toll road service quality attributes that matter to both operator and user, therefore only importance of each attribute is analyzed. The mean importance level of each attribute is calculated. Attributes with importance value above the mean importance are prioritized and considered as important toll road service quality attribute. The result of this analysis is mapped in importance diagram using SPSS to show importance level of each attribute according to both point of view.

Mann-Whitney test was performed before doing the importance analysis. Based on the test, ten out of 17 attributes have significance value of below 0.05. This figure means there are significant differences between operator's and user's point of view of the attributes' importance.

4 RESULT AND DISCUSSION

Toll road operator's perspective represents toll road performance based on service providers' point of view, while user's perspective represents customer's point of view of services they received. Importance-Performance Analysis is used in this research, however only two quadrants are used, which are top and bottom quadrants (importance). Furthermore, 17 research attributes are grouped into seven TRSQ dimensions, namely information, accessibility, reliability, mobility, safety and security, rest area, and responsiveness.

Table 2. Importance of Each Attribute Based on Operators' and Users' Perspective

Dimension	Attribute	Importance	
		User	Operator
Information	Toll road information	4.074	3.983
	Entrance information board	4.166	4.071
	Information board on toll road	4.226	4.156
	Signage	4.157	4.149
Accessibility	Access road	4.154	3.989
	Entrance transaction	4.182	4.088
	Exit transaction	4.153	4.103
	City access	4.113	4.119
Reliability	Road condition	4.211	4.119
	Tariff	4.132	4.170
Mobility	Travel time	4.200	4.033
Safety and security	Accident handling	4.188	4.170
Rest Area	Parking lot	4.070	3.906
	Toilet	4.092	3.993
	Gas Station	4.126	3.912
	Restaurants	4.068	3.923
Responsiveness	Response to complaints	3.993	4.034

Table 2 shows the importance level of each attribute according to toll road operator's and user's perspective. As explained before, this research only uses importance level of the attributes. Then, all 17 attributes are grouped into two quadrants to determine important and not important attributes. Attributes with high importance level (above average) are grouped in quadrant I, and attributes with low importance level (below average) are grouped in quadrant II.

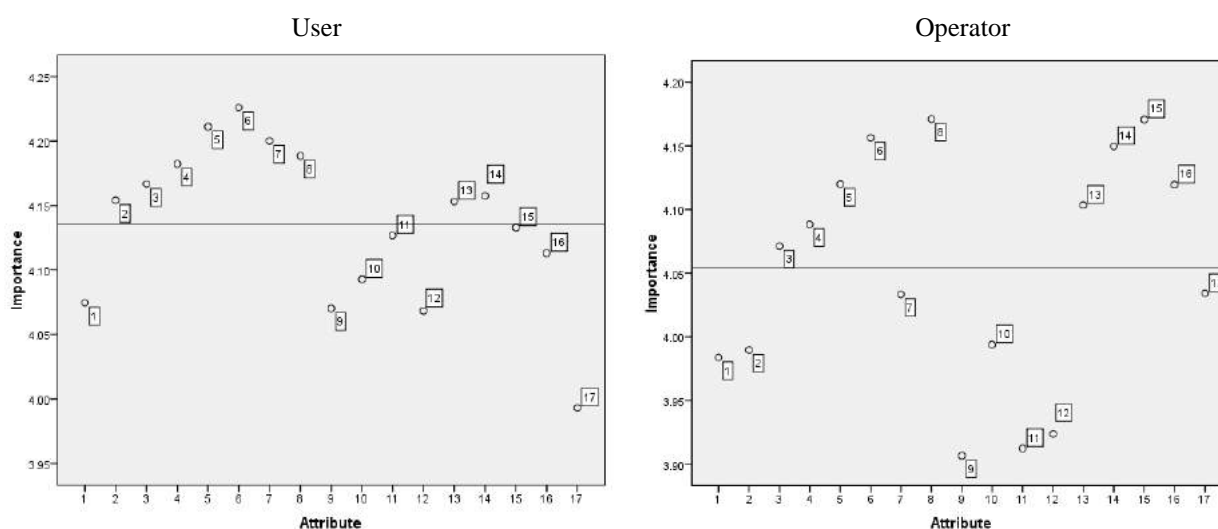


Figure 2. Attributes Grouping Based on Toll Road Operators' and Users' Perspectives

Figure 2 depicts attribute grouping based on both perspectives, using SPSS. From those pictures, it is seen that importance level based on toll road user's point of view has higher mean. Some attributes are important according to users, but considered less important to operators. This condition shows that consensus has not been reached between both parties, in terms of important toll road service quality attributes.

Based on toll road operators' point of view, there are nine important attributes, namely entrance information board, information board on toll road, signage, entrance transaction, exit transaction, city access, road condition, tariff, and accident handling. According to the dimension grouping, information, accessibility, reliability, and safety & security are important dimensions for toll road operators.

Furthermore, based on users' perspective, there are also nine important attributes in toll road service quality, namely toll entrance information board, signage, entrance transaction, access road, exit transaction, road condition, travel time, and accident handling.

Table 3. Importance Level of Toll Road Service Quality Attributes

Quadrant I		Quadrant I	
Important Attributes (User)		Important Attributes (Operator)	
Attributes	Dimensions	Attributes	Dimensions
(2) Access road	Accessibility	(3) Entrance information board	Accessibility
(3) Entrance information board	Accessibility	(4) Entrance transaction	Accessibility
(4) Entrance transaction	Accessibility	(5) Road condition	Reliability
(5) Road condition	Reliability	(6) Information board on toll road	Information
(6) Information board on toll road	Information	(8) Accident handling	Safety & Security
(7) Travel time	Mobility	(13) Exit transaction	Accessibility
(8) Accident handling	Safety & Security	(14) Signage	Information
(13) Exit transaction	Accessibility	(15) Tariff	Reliability
(14) Signage	Information	(16) City access	Accessibility
Quadrant II		Quadrant II	
Less-Important Attributes (User)		Less-Important Attributes (Operator)	
Attributes	Dimensions	Attributes	Dimensions
(1) Toll road information	Information	(1) Toll road information	Information
(9) Parking lot	Rest Area	(2) Access road	Accessibility
(10) Toilet	Rest Area	(7) Travel time	Mobility
(11) Gas station	Rest Area	(9) Parking lot	Rest Area
(12) Restaurants	Rest Area	(10) Toilet	Rest Area
(15) Tariff	Reliability	(11) Gas station	Rest Area
(16) City access	Accessibility	(12) Restaurants	Rest Area
(17) Response to complaints	Responsiveness	(17) Response to complaints	Responsiveness

Table 3 depicts the important and less-important attributes based on toll road operators' and users' perspectives. If those attributes are grouped based on TRSQ dimensions, five dimensions are important according to toll road users. Those dimensions are information, accessibility, reliability, mobility, and safety & security. Toll road operators, however, contended that mobility is less important. There are only four important dimensions namely information, accessibility, reliability, and safety & security.

5 CONCLUSIONS

Toll road is one of the most important infrastructures in urban areas, as it supports mobility of the citizens. PPP has been one of the effective solutions for toll road provision, as it involves both the government as regulator and private sectors as operator. Toll road provision under PPP could maximize risk transfer, drive innovation in scope, schedule, and budget, and also exploit the discipline of private sector capital.

As the regulator, the government must be able to set toll road service quality standards that represent toll road operator's and user's perspective. This aims to get higher toll road performance and users satisfaction. Based on the analysis results, generally toll road users and operators have the same perspective on toll road service quality. Both agree that information, accessibility, reliability, and safety & security are important dimensions. Therefore the government should focus on those dimensions in setting toll road service quality standards. However, mobility should also be considered as it is important for users. It was also mentioned earlier that travel time is one of the main indicators for toll road performance.

Based on the finding of this paper, government should involve not only toll road operators, but also toll road users, in setting toll road performance standards. Toll road user's satisfaction should also be one of toll road performance indicators. Moreover, further research can be held, if necessary, involving the government, toll road users, and toll road operators, to achieve consensus in toll road performance indicators and standards.

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7 ACKNOWLEDGEMENTS

At the end of the main text you can include a statement of acknowledgement of assistance. Grant or award numbers can be quoted as can departmental publication numbers. Acknowledgements must be brief and confined to persons and organisations who have made significant contributions. Do not include the title or rank of people.

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PAPER TITLE	The Italian ITS Logistics Platform for the management of dangerous goods transport and parking area for heavy trucks		
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KEYWORDS:

Dangerous goods transport, intelligent transport systems, logistics platform, parking area, safety & security

ABSTRACT:

Italy is one of the European countries with the highest traffic density. Road is the main transport mode used both for freight and passengers transport with congestion, safety & security and environmental pollution issues, despite the geographical features of Italy should favour other transport modes.

To make more efficient, secure and safer the national logistics system, in 2005 the Italian Ministry of Infrastructures and Transport established the UIRNet company with the aim to create, control and manage the ITS National Logistics Platform.

In the frame of this Platform, UIRNet has to contribute also at the improvement of road safety. The paper deals with the projects implemented by UIRNet to control and manage both road transport of dangerous goods and information services for safe and secure parking area for trucks, and the benefits obtained in terms of enhanced safety and efficiency that make UIRNet a best practice for any similar application worldwide.

1 TRANSPORT IN ITALY

Italy is one of the European countries with the highest density of internal traffic that is unevenly distributed across its transportation network that includes 278 ports, a rail network of 16,752 km, a road network of 255,980 km, a highway network of 6,751 km (5,724.4 km toll road) and 44 airports.

In 2014 the total cargo traffic of national carriers with origin and destination within Italy, and travelling distances over 50 km was more than 176 billion tons-km/year, with 56.51% of the demand focused on the road, while the rest was distributed between rail / pipeline (15.63%) and inland waterways (27.28%). The percentage of freight transport by air is instead irrelevant (0.59%). Furthermore, rail and pipeline traffic, the share of international traffic carried out on national territory, are comprised in the data above (see Figure 1).

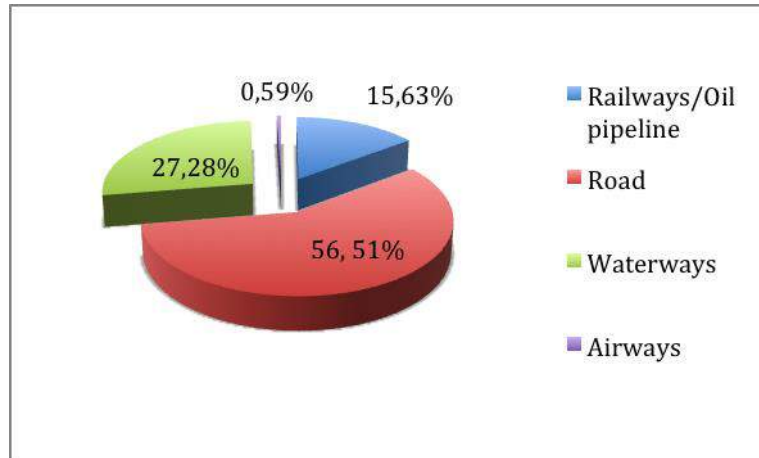


Figure 1 – Modal Shift in 2014 (% of Millions of ton-km)

Considering that for passengers transport there is the same modal shift, this data clearly outlines the absolute preference for road transport over other modalities in Italy. Unfortunately, this preference is not matched by the infrastructure currently in place, which is not yet adequate, compared to the heavy demand. This shortfall results in extensive negative externalities, in terms of congestion, environmental pollution and safety.

Furthermore, the demand for road transport is not spread evenly on the territory: traffic flows are concentrated in a few critical road segments and nodes around major metropolitan and industrial areas. In fact the problem of traffic in urban areas has become very serious as we find 50% of the population, over 70% of production activities, and 60% of circulating vehicles all converging in one area. Moreover, in the past two decades we have seen a strong tendency for people to reside outside large cities, which has consequently increased the number of commuters, which in turn has increased traffic on the already stressed urban road network.

2 FREIGHT TRANSPORT IN ITALY

Logistics is a vital and crucial sector for the national economy of the country.

The ability of a country to maximise profits on its production and to be competitive on the global arena are strictly connected to its capacity of optimize the distribution process, and thus to its logistics sector.

The most recent statistics released by the Italian Ministry of Infrastructure and Transport (2013-2014) highlight that the total transported goods in 2014 are around 176 billion ton-km, with a decrease of 25,7% compared to 2005. The available data for the 2014 (as reported in Table 1) confirm the prevalence of road transport respect the other modes with the following value of ton-km and percentage:

- ✓ 99.649 Millions of ton-km (56,51%) via road;
- ✓ 48.100 Millions of ton-km (27,28%) via waterways;
- ✓ 27.556 Millions of ton-km (15,63%) via railways and oil pipelines;
- ✓ 1.040 Millions of ton-km (0,59%) via airways, a very limited percentage.

Transport modes	Year 2005	Year 2010	Year 2014
Waterways	46.928	53.291	48.100
Railways / Oil Pipeline	33.668	28.222	27.556
Road	155.872	134.261	99.649
Airways	982	1.013	1.040
Total	237.450	216.787	176.345

Table 1 – Modal shift overview (Millions of ton-km)

Therefore, freight transport by road is the mode more used respect to the others. The main reasons can be summarised as follows:

- the average distance for freight transport in Italy is less than 300 km and therefore the road mode is the most efficient (main reason);
- the inefficiency of intermodal transport process: often it is occurred that wagons/trailers were missed during the modal change phase and also usually the modal exchange takes a lot of time;
- the trade associations of haulage freight transport are very strong and obtain subsidies/contributions for toll and fuel costs.

The challenge in Italy is to improve the efficiency of the logistics sector, favouring a more balanced modal shift in favour of other modes respect to roads.

3 THE ROLE OF ITS FOR ENHANCING LOGISTICS

In order to address the challenges of the increasing demand for transport of goods, and to be in line with other European countries, the Italian Ministry of Infrastructure and Transport has stated the need for Italy to rethink the whole transport network.

The aim is to deal with transport as an “integrated system”, where information, management and control operate in synergy, optimizing the use of vehicles and infrastructure and of the existing logistics platforms in a multimodal manner. This is possible only through the extensive use of Intelligent Transport Systems (ITS).

In particular as regards logistics, the Ministry of Infrastructure and Transport in 2005 established UIRNet company with the aim to create, control and manage the ITS National Logistics Platform.

Such need is reported in the National Action Plan on Intelligent Transport Systems adopted by the Ministry of Infrastructure and Transport as requested by the ITS Directive 2010/40/EU. In such Plan, Priority Area 2 *Continuity of ITS services for the management of traffic and goods transport* specifies that “the objective that it is necessary to achieve is the possibility of organizing integrated multimodal mobility services for people and for freight that enable the planning and management of movements in an informed and personalized way without solutions of continuity from the point of origin to the destination, using all available ways efficiently and securely through open and operable integrated platforms”.

The ITS National Logistics Platform is the implementation of what outlines the National ITS Action Plan.

4 THE ITALIAN LEGISLATIVE FRAMEWORK ON ITS

On 6th August 2010, the European Commission published the ITS Directive 2010/40/UE adopted on the 7 July 2010. This Directive establishes a framework in support of the coordinated and coherent deployment and use of Intelligent Transport Systems (ITS) within the Union, in particular across the borders between the Member States, and sets out the general conditions necessary for that purpose. According to the ITS Directive 2010/40/UE, all Member States are requested to adopt this Directive and to implement an ITS National Action Plan.

Italy adopted the ITS Directive in 2012 and the ITS National Action Plan the last 12 February 2014. The Italian Plan includes four priority areas, reflecting those indicated by the ITS Directive 2010/40/UE:

1. Optimal use of road, traffic and travel data;

2. Continuity of traffic and freight management ITS services;
3. ITS road safety and security applications;
4. Link the vehicle with the transport infrastructure.

The second priority area addresses the issues related to the achievement of safety conditions, efficiency, interoperability and continuity of ITS services for the management of traffic and transport, as well as those needed to stimulate inter-modality and co-modality in the European transport corridors and conurbations.

The goal is to achieve integrated multimodal mobility, both for people and goods, to plan and manage the movement in an informed, personalized and seamless way from the point of origin to that of destination, using all the available modes efficiently and safely.

The development of integrated mobility services for both people and goods is based on: availability, access and set up of data and information that constitutes the key to enable such services; the management and organization of this data must be integrated platforms open and interoperable; same for payment systems integrated with the transport services. It is necessary that the different operators collecting and processing information on mobility, dialogue with those platforms.

The second priority area identifies the five priority actions needed to reach the objectives listed. Priority actions 1, 2 and 4 are particularly relevant to freight transport.

Priority action 1 aims at encouraging the creation of telematics platforms in the logistics nodes of the distribution network that are consistent with the ITS National Logistics Platform. This will allow the exchange of data, information and documentation between operators, which will improve, simplify and speed up all processes and administrative issues within the complex cycle of intermodal transport (road, rail and sea).

This will necessitate the promotion of an extensive information and training campaign aimed at the real users of telematics platforms, in order to facilitate their use and encourage the development of open ITS systems, which have to be interoperable with each other and with the ITS National Platform for the Logistics.

Priority action 2 aims at encouraging the use of ITS for the management of multimodal transport systems and logistics through open and interoperable platforms. Within the second priority action, the challenge is to implement ITS systems dedicated to logistics and freight transport, by stimulating and intensifying the intermodality and co-modality of transport both at national and international level, through 21 corridors recognized at European level.

ITS systems will have to be interoperable, standardized and will use the ITS National Platform for the Logistics as reference point for road transport.

Furthermore, the continuity and the interoperability of the ITS services that facilitate the switch between road transportation to other modes in every node (ports, freight villages, stations and airports) will have to be guaranteed in terms of:

- release of information;
- simplification of administrative proceedings;
- fluidity of circulation in intermodal areas;
- reduction of waiting times;
- harmonization of interactions among the different actors involved through the National Logistics Platform, to manage:
 - freight transport information and the related e-documents;
 - tracking and tracing of vehicles transporting dangerous goods through RFID (Radio Frequency Identification), GPS (Global Positioning System) and EGNOS (European Geostationary Navigation Overlay System)/Galileo;
 - use to technologies to check status of both vehicles and goods, adopt standard protocols and open and interoperable ITS architectures for data exchange.

According to the specific characteristics of the Italian framework, special attention will be given to city logistics to control vehicle type and time of pick up and delivery of goods within the urban area.

Priority action 4 aims at guaranteeing the continuity of services on the national network and along borders. Within this priority action, the "interfacing" of national control systems of passenger and freight traffic, will be favoured at the European level to ensure the continuity of management services and information on the entire national network and along the borders, trans-borders collaborations with the other Member States will be established.

The logistic sector is crucial for Italy: this is underlined both by the ITS Action National Plan, as described above, and also by the National Plan for Logistics 2011-2020: Italy could save up to 4 billion Euro by reducing the inefficiencies of the logistics sector.

The creation of an ITS National Platform for the Logistics is a primary step to improve the efficiency of the sector and support Italian economy.

5 UIRNET: THE ITALIAN ITS LOGISTICS PLATFORM

UIRNet S.p.A. is the only entity entitled by the Italian Ministry of Infrastructure and Transport for the creation and control of the platform for the management of the national logistics network.

The ITS National Logistics Platform has been defined in 2005 and instituted to improve the efficiency and safety of the Italian logistics network favouring the interconnection of modal interchange nodes (ports, freight villages, goods centres and logistic plates).

The main mission of UIRNet is to create a network integrating the complex world of transport and logistics in a simple way, without introducing any market change and without privileging one or the other category of operators.

Such an integration can be achieved through the Platform that can interconnect the main actors of the Italian logistics system, allowing the coordination of the information flows and of the related processes. Therefore, the physical flow of goods along the roads in and out towards the logistics nodes can be better modulated.

The Italian ITS Logistics Platform has been conceived to offer services to all the logistics operators, acting as the interconnection platform for data and related processes. Nowadays, the National Logistics Platform is focused on the road freight transport and aims at regulating the interaction between transport companies and nodes operators (customers, warehouses, terminal operators, shippers, MTO).

UIRNet has developed a Platform able to offer several services to many actors of the logistics system such as transport companies, infrastructures managers, production companies and public authorities.

At the moment, the Platform is composed by integrable and scalable modules and UIRNet has primarily created the core modules, essential for the minimum services offered by the Platform:

- **SMARTRUCK:** The module addresses the freight transport companies and whoever is interested in organizing and managing a journey or mission.
- **DANGEROUS GOODS:** UIRNet, through the module Dangerous Goods (an extension of SmarTruck) adds a powerful tool of control of documents related to dangerous goods.
- **CONTROL TOWER:** the Control Tower module allows the logistics actors such as nodes, shippers, MTO, maritime actors, to receive accurate and almost in real time information on vehicles travelling towards them.
- **BOOKING:** the Booking module allows on one hand to those who offers services related to transport to make such services available for all UIRNet users; on the other hand it allows transport companies and goods settlers to use the same services by booking them.
- **FREIGHT TAXI:** the Freight Taxi module allows a transport supplier (with a cargo space at disposal) or a customer (with goods to be delivered) to gain further business opportunities.
- **GNOSCERE:** it is the Data Warehouse, a specific module based on SAP - Business Object, allowing to elaborate and analyse all the data acquired during the system operation and to show them in a simple and effective manner (both through tables and graphics).
- **SOLUTION:** concerning the needs of the main logistics nodes, UIRNet has realised a module that, through ad hoc services and processes, allows the management of the resources and of the available services at nodes, the management of the inbound queue and/or emergencies and the management of the registry of the road transport common to all nodes.

- CUSTOMS CONTROLLED CORRIDOR: to support the requests from the Agenzia Dogane e Monopoli, UIRNet has realised a module to transport and manage goods subject to customs constraints (not national or subject to customs constraints) along road stretches pre-defined by the Agency.
- PORT COMMUNITY SYSTEM INTEGRATION (PCS): it is an applicative framework able to evolve and integrate the information already in operation in the port area of reference (PCS) into a further connected system where the ITS National Logistics Platform operates.
- DANGEROUS GOODS (DG BASE): this is the module integrating the ITS National Logistics Platform allowing the development of ad hoc services for the dangerous goods transport, by calculating the dynamic risk factor, optimizing the route, to increase the land safety and, at the same time, to comply with the rules for the transport security.
- PARKING AREA AND BUFFERING: this is the module to manage the parking areas and also allowing both the planning of the ordinary stops and the management of the buffering areas in case of negative events that can prevent the node operations.
- THE REGIONAL LOGISTICS PLATFORM (PLR): an autonomous system integrated with the ITS National Logistics Platform services and managing its own users at regional level.
- MASTER DELIVERY SERVICE (MDS): the module has been developed to manage the inbound flows to the exhibition area of Expo 2015 of the logistics vehicles through a planning algorithm specifically designed, but replicable in other situations.

6 UIRNET SOLUTIONS FOR DANGEROUS GOODS MANAGEMENT

In November 2015, UIRNet ended two different projects aimed at the control and management of the dangerous goods road transport. All these projects, Picoge-MP and Module DG Base, aimed to increasing both safety and security level and achieved the expected results.

Picoge-MP project

Picoge-MP is an integrated platform for the transport and logistics governance of dangerous goods as well as a tool for emergencies management. It acts as a strategic architecture to define the guidelines to be adopted by the Italian Ministry of Infrastructures and Transports to monitor and manage dangerous goods and able to guarantee the fusion, interoperability and communication through a common language of ITS applications. The main objectives the project have been:

- To design and realize a strategic architecture able to define specific guidelines for the control and management of dangerous goods. The aim is to make the different ITS applications integrated, interoperable and able to communicate through a common language;
- To harmonize all the existing projects on the dangerous goods management;
- To create of the National Map for Risk, based on static elements and whose geo-referenced elements are available through machine-to-machine services or for consultation;
- To create a website for the dissemination of all the issues related to dangerous goods in order to communicate with users and foster their participation (www.picoge.it).

According to the above listed objectives, as a first step, UIRNet has defined and shared with the Italian Ministry of Infrastructures and Transports both guidelines and specifications to design the telematics architecture to act as a referring point for the dangerous goods transport. UIRNet has analysed in deep the state of the art of telematics platform and ITS project in the dangerous goods sector in order to collect all the existing experiences and share with the users and actors a new approach.

Finally, a website and a tool to support ITS systems design have been created. Therefore, all the gathered information are available online through the dedicated website www.picoge.it

Picoge-MP benefits

The PICOGE-MP project gathered some important results:

- The definition of a standard for ITS systems design to harmonise project proposals at least as concerns the presentation language. On the Picoge_MP website (www.picoge.it) a design tool is available based

on the standard and meant to ease and speed the project redaction. In addition, a tool for pre-validation of architectures has been inserted to be used as a support in the projects evaluation to be funded. For example, the tool would be useful to support the Italian Ministry of Infrastructure and Transport in technical tenders evaluation for ITS systems design as well as in receiving homogeneous proposals for an easier and faster evaluation.

- The availability of a geo-referenced database for vulnerable elements. A single national referring point for land maps including the evaluation of eventual incidents. The map can be consulted through the user interface or through services. The main advantage of such a referring point is the possibility to manage data in a harmonised way and to make them available for Police, Fireman, ITS systems dealing with incidents management.

DG Base project

The **DG (Dangerous Goods) Base** designed, realized, implemented, activated and managed an integrated telematics system to monitor and control both at operative and administrative level the dangerous goods transport in Calabria and Sicily Regions, involving the main ports and freight village.

The objectives of the project have been:

- To design and realize a real time control system for the dangerous goods transport on the main infrastructures at regional level, guaranteeing a higher level of safety and limiting the risk linked to vehicle transit without decreasing efficiency;
- To reduce the risk of accidents related to dangerous goods transport by monitoring both the state of the vehicle and of the goods;
- To improve the emergencies management to reduce the effects of eventual accidents;
- To improve the PA action through the identification of all the required elements for a map of risks on the territory;
- To improve the logistics standard to support drivers with effective information on the dangerous goods transport modes;
- To collaborate with other regions to ensure the interoperability of the tools used for the inter-regional control.

To reach the above listed objectives, a traffic control room has been established in Messina (South Italy). Also, video surveillance cameras have been implemented to detect both vehicles and KEMLER-ONU plates.

The services offered by the DG Base project to manage the dangerous goods transport are the following:

- Localisation of the vehicle transporting dangerous goods;
- Vehicles monitoring;
- Monitoring of the dangerous good;
- Improvement of emergencies management through a warning system;
- Real time messages to logistics chain actors;
- Use of dynamic elements on the risk maps;
- Route planning;
- Data archive and management;
- Application of intelligence business tools.

To facilitate the remote control, an App has been developed which is able to query the Logistics Platform and to collect all the information.

DB Base project benefits

The Module DG Base has gathered the following benefits:

- It enables the match between the qualified and certified consultants for dangerous goods and the companies that, according to Italian law, must use them. Therefore the module allows an automated control with relevant benefits for security both of working places and dangerous goods transport. Actually, at the moment such controls are rare and based on paper documents. It allows the monitoring of dangerous goods transport both through vehicles tracking and cameras for optical identification. The system defines both the current and expected situation for transport by calculating

the risk index related to transport. For example, if a petrol transport passes close to a liquid oxygen transport, then the system will alert the control room and operators can manage any eventual risk.

- Through an electronic brain able to record rules and experiences, the system is able to detect abnormal situations, ie. situations involving risk of deviation from the transport rules. For example, a tanker carrying substances that are not treated in the area where it moves or vehicles that take too long or perform non-linear paths between the source and destination (typical case of those who empties the street tanks).

UIRNet is discussing with the Italian Ministry of Infrastructure and transport in order to receive other information on dangerous goods transport from other petrol companies. Obviously the presented system is immediately extended to the entire country.

7 UIRNET SOLUTIONS FOR THE MANAGEMENT OF PARKING AREA FOR HEAVY TRUCKS

The project “**Remote management of the areas of buffering and parking for trucks with experimentation in the area of Catania Bicocca**” aimed at interconnecting all the parking areas, freight villages and road / highway crossings with the systems of the National Logistic Platform. The project was open only to road transport to increase the level of security in the parking areas and to ease the access to the service, supporting the respect of driving and resting time foreseen by the law for truck drivers. The truck drivers can access the service and obtaining all the needed information on parking areas availability through a dedicated App.

The project answers to a specific Delegated Regulation of the European Commission: Delegated Regulation (EU) N° 885/2013 of 15 May 2013 integrating the Directive 2010/40/UE with regard to the provision of information services for safe and secure parking places for trucks and commercial vehicles.

The main objectives of the project have been:

- To supply online services for the National Logistics Platform users;
- To ease the access to services to allow the driver the guide/rest time compliance as well as a higher comfort;
- To increase the security level in the parking areas;
- To improve the perception both for economic operators and drivers of the increased safety level and of the possibility to work in the law respect;
- To ensure a greater collaboration between the National Logistics Platform and the other actors of the system.

The achievement of the objectives of the project was possible thanks to the integration capabilities provided by UIRNet National Logistics Platform, which allows to refer to a single standard of information exchange, ensuring systems interoperability, and to use the basic services already in place.

The system is at the moment available in the parking area of Catania but can be replicated in any parking and buffering area thanks to a configuration tool able to personalise its functionalities. The module is able to plan ordinary stops and to manage the buffering areas in case of negative events that can prevent the node operation. The system converges in a unique solution the management of physical and logic safety of the stop infrastructure. Therefore execution and actions time are integrated and optimized.

Benefits gathered

The Module for parking areas control has allowed the following benefits:

- Online booking and management of parking lots within parking areas. Therefore truckers can safely plan their stops;
- Control of parking areas access by cameras with optical identification. Therefore safety is increased and gates management is improved by automation;
- Combining safety and organization, the module services allow parking areas to organize services for the passage of the drive between tractors (shifts in security) and to exchange trailer (trailers,

semitrailers, trucks ...) between tractors even at different times. This will give possibilities to small customers forms of organization at the time exclusively available only to large operators of shipments.

CONCLUSIONS

The paper presented an overview of the Italian freight transport, the ITS National Logistics Platform and some initiatives promoted by UIRNet for the management of dangerous goods transport and parking area for heavy trucks.

The adoption of the National ITS Logistics Platform can maximise the efficiency and the quality of the Italian logistics sector. The ITS Platform offers to the whole logistics chain a new strategic opportunity to improve the efficiency and security of the entire supply chain. This has a significantly positive impact on the national economy in terms of increased competitiveness, security, environmental benefits, as well as opportunities for internationalisation.

The Platform will greatly improve the efficiency and safety of the Italian freight transport: the system is tailored to address the territory characteristics and each stakeholder needs, optimizing each process, maximising assets and minimizing negative externalities.

Moreover, the ITS National logistics Platform, when it will be full operative in 2018, is expected to have huge impacts in terms of:

- Increasing national economic impacts, through the reduction of the logistics enterprises costs thanks to a better logistics efficiency: increase of intermodal transport, development of freight villages, integration with other logistics systems at national and international level, etc; this benefit is estimated in 2,5 billions of Euro/year;
- Positive impacts on employment in Italy, thanks to the reduction of the logistics performance gap respect to other European countries; this benefit is estimated between 52.000-95.000 new jobs;
- Positive environmental impacts, thanks to the optimization of the logistics flows and monitoring of dangerous goods transports; this benefit is estimated between 1,6 and 2,5 Billions of Euro/year;
- Positive impacts on GDP, thanks to the increase in competitiveness of the logistics sector; this benefit is estimated between 2,6 and 4,7Billions of Euro of GDP;
- Increasing safety through the monitoring, managing and controlling of the goods transport (and dangerous goods transport in particular), and the parking area for heavy trucks;
- Internationalisation through the integration with European and Asian ports, as well as integration with any other logistics project worldwide;

Therefore UIRNet is expected to become a pillar in the field of logistics and transport.

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KEYWORDS:

Road accident, design consistency, operating speed profile, global positioning system

ABSTRACT:

Almost 1.25 million people in a year around the world involve with road accident and suffering serious injury, slightly injury and disability based on road accident. In developing country, Malaysia is among that has the highest road accident risk and more than 50% of the road accident fatalities involves motorcyclist. Analysis shows that, the three main types of accidents in Malaysia is collision with passenger cars, collisions with other motorcycles and single-motorcycle accidents. The reasons of accident happened because lacks of road design consistency and most drivers make errors due to the geometric features on the road. Design consistency in a road section usually is results from geometric features that vary significantly and may cause drivers to make speed errors or unsafe driving leading to higher collision risk. By evaluate design consistency are through based on operating speed evaluation of 85th percentile speed of vehicles, and obtained by using continuous speed profiles (normalized area bounded of speed) for consistency value for the selected segment. These studies are to determine the integrated road design consistency (IC) of motorcycles and cars at selected segment F0050. GPS DG-200 system is used to get speed data for motorcycle and cars for develop design consistency profile. Only operating under free flow conditions were considered in this study. Geometric factors such as number of access point, tangent length, and curve length are influence for operating speed profile for developed of tangents and curves of motorcycle and cars. Use ACAD software for develop consistency model profile. The developed of consistency model are based on parameter: the bounded area between the profile and the average speed, standard deviation of speed along a segment. The motorcycle design consistency for distance of 5 km is 2.90 meanwhile; distance of 25 km is 2.30. Both distances show a good design consistency even the threshold are different. The cars design consistency for distance of 5 km is 2.41 while distance of 25 km is 1.26. Both distances shows a different threshold of design consistency: 2.41 (good consistency) meanwhile 1.26 (acceptable consistency). The integrated-consistency model is to incorporate the impact of the speed profile on design consistency in traffic and safety evaluations. As a result, Integrated consistency motorcycle and cars for distance of 5 km are 2.81 (good consistency) whereas for distance of 25 km are 2.29 (good consistency). From the consistency prediction models, integrated design consistency route F0050 of motorcycles and cars are good consistency.

Cars – Motorcycles Collisions Prediction Model Based on Road Design Consistency

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1 INTRODUCTION

Every year almost 1.25 million people around the world involve with road accident. Among 20 and 50 million people are suffering serious injury, slightly injury and disability based on road accident occurring (WHO, 2015). Besides that, WHO stated road traffic accident can causes one million people are died, three millions are severely are disabled for life, and thirty millions are injured. By 2020, road accidents will be the third leading cause of death worldwide. In developing country, every year shows that road accidents are kill more people than war and virus. Furthermore, in Asia continent only, almost 400,000 people are died on the roads and more than four million injured every year (Kareem, 2003). The causes of road accident are based on imperfect vehicles, irregular roads, careless driving, speeding, drunk driving, not enough sleep, under the influence of alcohol and other drug effect and many more. In Malaysia, road accidents are one of the most relevant issues in today's civilization and the main cause's road accidents are death and injuries. In the middle of the ASEAN, Malaysia has the highest road accident risk and more than 50% of the road accident fatalities involves motorcyclist, and the numbers of accident kept rising. The West Coast Area of Malaysia been stated as the highest motorcycle fatalities due to the highest number of registered motorcycles and population. Previous study said the entire of motorcycle billion kilometers travelled in Malaysia are increased from 1990 to 2008, therefore, motorcycle fatalities also increased as well (Abdul Manan et al., 2012 and Torregrosa et al., 2011). The increase in motorization led to consequent increase in the number of road accidents (Radin, 2005). Motorcycles represent more than half the total vehicle population and contribute more than 47% of the casualties based on deaths, serious and slightly injuries in road accidents which recorded 48%, 47.4% and 47.7% for the year 2006, 2007 and 2008, correspondingly (Yuen et al., 2014). In year 2009, total number of motorcycles in Malaysia was 8,940,230 with fatalities contributed as 60% (Abdul Manan et al., 2012). Investigation shows that, the three main types of accidents in Malaysia is collision with passenger cars, collisions with other motorcycles and single-motorcycle accidents. Generally, riding motorcycle is 17 times more dangerous than driving a passenger car and also the motorcyclist will be in front of higher risk of injury or fatality because the level of exposure to injury is higher if compared to other vehicle such a passenger car in accident (Radin Umar et al., 1995). Meanwhile, according to the study by Road Safety Research Center of University Putra Malaysia, motorcyclist represent about 55 – 57% of the total number of accidents and 60 percent of traffic fatalities in Malaysia. Their risk of injury is estimated to be 12 times higher than car passengers.

Accident occurs due to lack of road design consistency, which most drivers make, mistakes in the area of geometric features (Prasetijo J. et al., 2016). The worse the consistency, the more likely that drivers will be unexpected and an accident will happen (Lamm et al., 1999 and Juan de Oña et al., 2012). Design consistency in a road section usually results from geometric features and may cause drivers to make speed errors or unsafe driving maneuvers leading to higher collision risk. Hence, geometric design consistency is emerging as an important component in highway design relate to the safety performance. Most consistency concepts today deal with the variation of vehicles speed but less variation types (heterogenous) on road section and this speed variation is affected mainly by horizontal alignment and vertical alignment of roads (Lamm et al., 1999; Juan de Ona et al., 2012; Torregrosa et al., 2011). The most criteria used to evaluate design consistency are based on operating speed evaluation of 85th percentile speed of vehicles, and obtained by using operating speed prediction models (Gibreel et al., 1999). Some researches (Hassan, 2004; Polus et al., 2004; Mattar-Habib et al., 2008) have extended the concept by using continuous speed profiles (normalized area bounded of speed) to determine the speed variation along a road segment and determining a single consistency value for the whole road segment. In transportation engineering, GPS has been frequently used in study of travel time, route choice, car following and behaviors' of driver's speed. GPS allows

researchers to track vehicles at second-by-second resolution at any time of the day and under any weather conditions. The use of GPS technologies has used up to carry out traffic data collection for transportation studies. Field study by using GPS device with differential potential switched on and the data were examined to verify the result of theoretical analysis.

2 DATA DESCRIPTION

The models are developed based on the new method of speed profiles analysis. Some models have been developed to predict operating speed (85th percentile), by estimating the 85th speed variation along the road. By using a Global Positioning System (GPS), continuous speed profile data are using new method analysis, to develop road design consistency profile of motorcycle and cars (Prasetijo, J. et al., 2016). Only operating under free flow conditions were considered in this study. Geometric factors such as number of access point, length of tangent and length of curve are influence for operating speed profile, 85th percentile for developed of tangents and curves of motorcycle and cars. Using the Excel Software, analysis was carried out. Multiple Linear Regression method been used to develop the model for prediction of 85th percentile speed for tangent and curve. Using ACAD software to developed road design consistency model.

2.1 ROAD LOCATION

The road location selection is F0050 (Batu Pahat – Ayer Hitam), based on the highest accident by using ranking accident point system based on weightage adopted by Highway Planning Unit (HPU) and based on road accident along Johor Federal road through accident data from IPK Johor Bharu, Malaysia. The data was collected during weekdays and off-peak hours. By using Global Positioning System (GPS) that placed inside moving vehicle, the speed data were collected for two type of vehicle (cars and motorcycle). Data speed from signalized intersection been remove about 300 m to avoid the effect of traffic control devices on vehicle speeds and free flow speed (Schurr et al; 2002). The data for this research consisted of two parameter; GPS (speed) and geometric parameter (length of tangent, length of curve and access point).

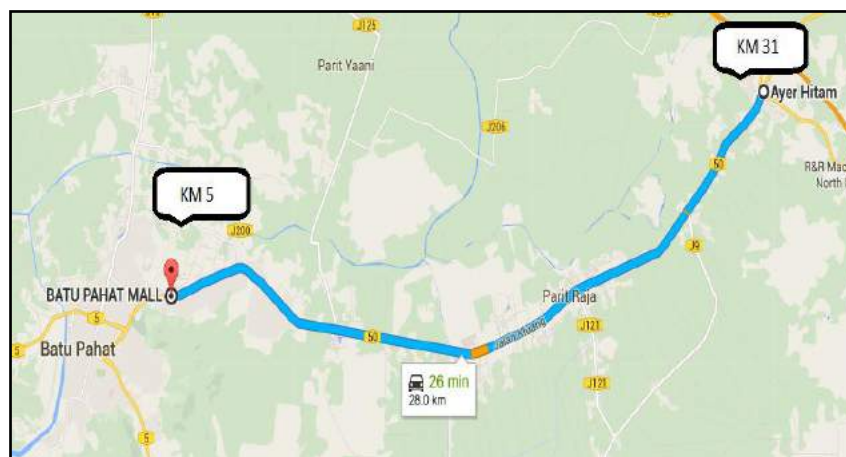


Figure 1. Road Location Batu Pahat to Ayer Hitam (31km)

2.2 DATA COLLECTION

The data was collected through test driver method using the same vehicle for cars and motorcycles. In this study, continuous speed profile (GPS DG-200) was used rather than collecting using spot speeds. GPS DG-200 was placed in the moving vehicle to record the data along a test segment. GPS are used to collect speed data of motorcycle and cars in traffic stream under free flow condition as mention early. Each data point recorded by GPS includes the vehicle position, speed, time and the distance. The results based an average speed vehicle profile that captures the whole moving process of the traffic flow travel along the road segments.



Figure 2. Global Positioning System DG-200

3 DATA ANALYSIS

Many research works defined the operating speed is the parameter of real driving performance and the speed where drivers travel in free flow conditions during daylight and calculated using a specific percentile of speed distribution, that normally used the 85th percentile (Russo et al., 2012). The profiles were obtained by considering continuous speed profile by using GPS devices from individual drivers which allowed accurate determination of the starting and ending points of all speed transitions and also the actual maximum and minimum speeds for the different road geometric elements (Torregorasa et al., 2011). According to the Mattar-habib (2004), continuous speed profiles are used to determine the speed variation along a road segment and determining a single consistency value for the whole road segment. The design consistency index is a continuous function instead of being based on ranges. Two main parameters for this model; first is equation 1; the bounded area between the profile and the average speed and the equation 2; the standard deviation of speeds along a two-lane highway segment. Based on the two independent measures, a consistency model was developed. Table 1 shows design-consistency quality (Matta-Habib et al., 2008). Threshold values state for good, acceptable, and poor design consistency was to know a consistency model.

$$R_a = \frac{(\sum a_i)}{L} \quad (1)$$

Where, R_a is relative area (m/sec) measure consistency, $\sum a_i$ is sum of i areas bounded between the speed profile and the average operating speed (m^2/sec) and L is entire segment length (m).

$$\sigma = \left\{ \frac{(V_j - V_{avg})}{n} \right\}^{0.5} \quad (2)$$

Where σ is standard deviation of operating speed (km/h), V_j is operating speed along the j^{th} geometric element (tangent or curve) (km/h), V_{avg} is average weighted (by length) operating speed along a highway segment (km/h) and n is number of geometric elements along a segment (km/h).

Table 1. Design-consistency quality (Matta-Habib et al., 2008)

Design-consistency quality		
$C = 2.808 \cdot \exp(-0.278 \cdot R_a \cdot \sigma)$		
Good	Acceptable	Poor
$C > 2$ (m/s)	$1 < C \leq 2$ (m/s)	$C \leq 1$ (m/s)

Integrated-consistency model is needed as a function of speed profile of cars and motorcycle. The model is included with the impact of speed profile of motorcycle on design consistency. The model takes into the combine effect of speed profiles of cars and motorcycles. The formula (Matta-Habib et al., 2008) from equation 3, can be used to conduct between motorcycle and cars to found integrated-consistency model.

$$IC = [2.808 * \exp(-0.278 \cdot R_a \cdot \sigma)] * \exp(-0.01 \cdot A_{CT}) \quad (3)$$

Where IC is integrated consistency of road sections, R_a is normalized bounded area by speed profile of cars /motorcycle (m/sec), σ is standard deviation of car speeds/ motorcycle and A_{CT} is normalized bounded area between the speed profile of cars and motorcycle (m/sec).

3.1 OPERATING SPEED MODEL

The speed profiles based on the models for curves were developed by (Krammes et al., 1995), while the models for tangents were developed by (Polus et al., 2008). Dependent variable selected based on speed at selected segment and independent variables are length of tangent and curve and access density. Excel Software been used to analysis multiple regressions. Operating speed model related based on 85th percentile speed, access density, length of tangent and curve. The selected operating speed model, statistical results are shows;

Motorcycle;

$$\begin{aligned} V_{85t} &= 84.13 + 0.10 L_t + 0.209 AP & R^2 &= 0.13 \\ V_{85c} &= 82.41 + 0.13 L_c - 0.798 AP & R^2 &= 0.10 \end{aligned}$$

Cars;

$$\begin{aligned} V_{85t} &= 75.03 + 2.50 L_t - 1.64 AP & R^2 &= 0.75 \\ V_{85c} &= 75.03 + 2.50 L_c - 1.64 AP & R^2 &= 0.75 \end{aligned}$$

Where;

V_{85t} = Operating speed of tangent, V_{85c} = operating speed of curve, L_t = length of tangent, L_c = length of curve, AP = access point

3.2 DESIGN CONSISTENCY

The design consistencies are calculated based on continuous operating speed profile models. The continuous operating speed profile models are plotted based on operating speed model developed. The developed of consistency model are based on parameter: the bounded area between the profile and the average speed, standard deviation of speed along a segment. The Area Bounded, R_a are calculated based on speed profile along the road. Analysis of road design consistency is a good way to reduce road accident along the segment.

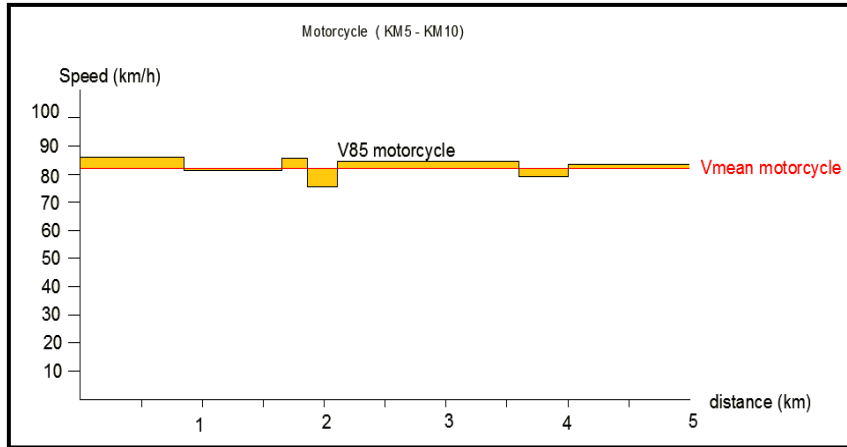


Figure 3. Design Consistency Profile of Motorcycle (KM5 – KM10)

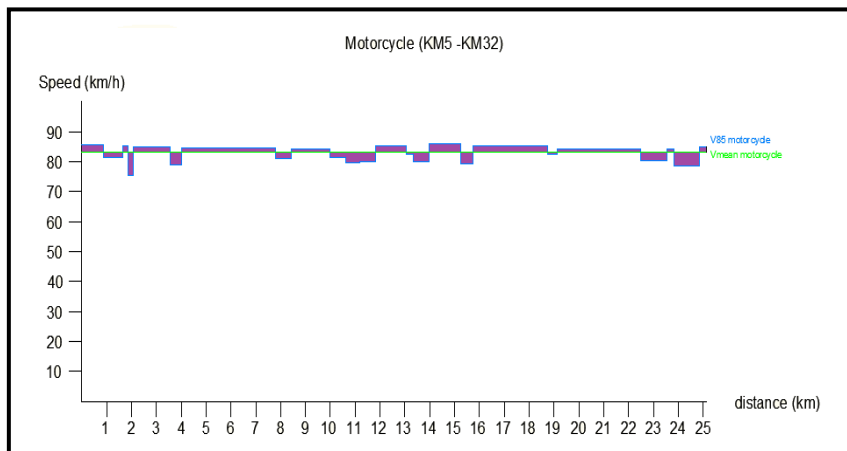


Figure 4. Design Consistency Profile of Motorcycle (KM5 – KM31)

Figure 3 shows Design Consistency Profile of Motorcycle (KM5 – KM10). The motorcycle design consistency for distance of 5 km is 2.90. Figure 4 shows Design Consistency Profile of Motorcycle (KM5 – KM31) for distance of 25 km at F0050. The motorcycle design consistency for distance of 25 km is 2.30. Both distances show a good design consistency even the threshold are different.

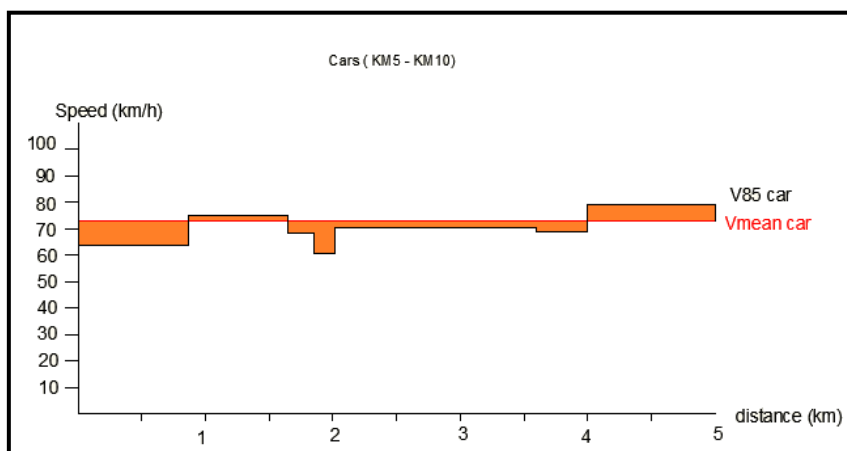


Figure 5. Design Consistency Profile of cars (KM5 – KM10)

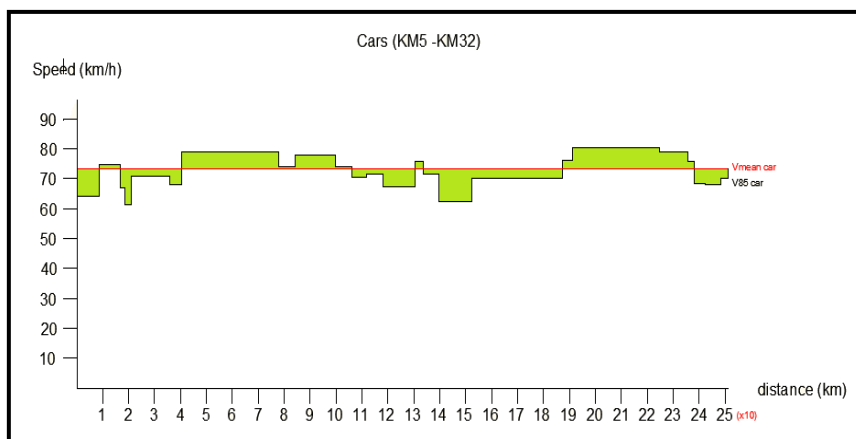


Figure 6. Design Consistency Profile of cars (KM5 – KM31)

Figure 5 shows Design Consistency Profile of cars (KM5 – KM10) for distance 5 km. The cars design consistency for distance of 5 km is 2.41. Figure 6 shows Design Consistency Profile of cars (KM5 – KM31) for distance of 25 km at F0050. The cars design consistency for distance of 25 km is 1.26. Both distances shows a different threshold of design consistency: 2.41 (good consistency) meanwhile 1.26 (acceptable consistency). Table 2 shows the summary design-consistency quality based on type of vehicle.

Table 2. Design-consistency quality based on type of vehicle

Type of vehicle	Segment (KM)	Design Consistency	Threshold
Cars	5 – 10	2.90	Good Consistency
	5 – 25	2.30	Good Consistency
Motorcycles	5 – 10	2.41	Good Consistency
	5 – 25	1.26	Acceptable Consistency

3.3 INTEGRATED DESIGN CONSISTENCY (IC)

The integrated-consistency model is to incorporate the impact of the speed profile on design consistency in traffic and safety evaluations. Figure 7 shows Design Consistency Profile of Motorcycle and Cars (KM5 – KM10) for distance of 5 km. Figure 8 shows Design Consistency Profile of Motorcycle and Cars (KM5 – KM31) for distance of 25 km. Integrated consistency motorcycle and cars for distance of 5 km (KM5 – KM10) are 2.81 (good consistency) whereas Integrated consistency motorcycle and cars for distance of 25 km (KM5 – KM32) are 2.29 (good consistency). Speed between cars and motorcycles shows a good design consistency in this segment. Table 3 shows the summary Integrated Design Consistency of Motorcycle and Cars

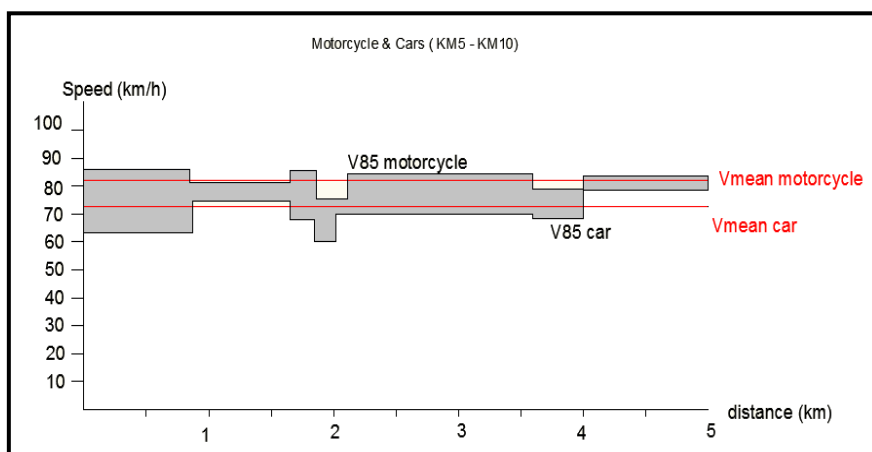


Figure 7. Design Consistency Profile of Motorcycle and Cars (KM5 – KM10)

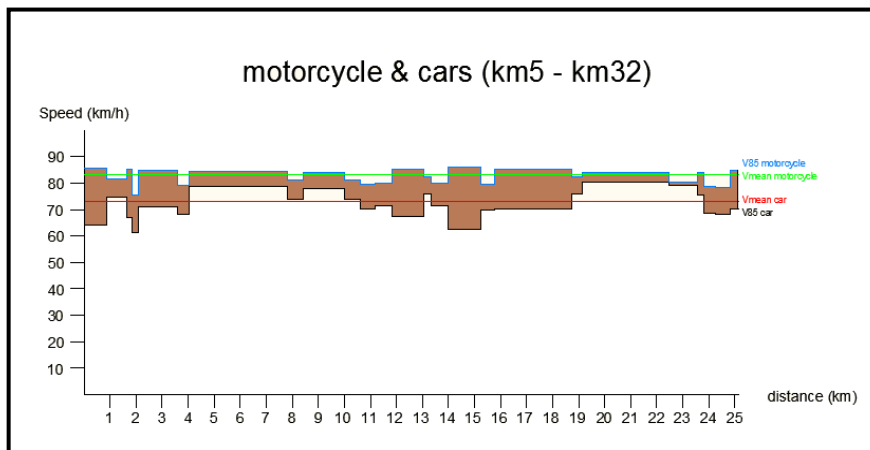


Figure 8. Design Consistency Profile of Motorcycle and Cars (KM5 – KM31)

Table 3. Integrated Design Consistency of Motorcycle and Cars

Type of vehicle	Segment (KM)	Integrated Design Consistency	Threshold
Motorcycle & Cars	5 – 10	2.81	Good Consistency
	5 – 31	2.29	Good Consistency

4 CONCLUSIONS

The consistency of the geometrical can be defined the synchronization between drivers' expectancy on the road and geometry. Continuous speed profile data that collected using Global Positioning System (GPS DG-200) equipment through under free flow condition are to develop design consistency profile of motorcycles and cars. Therefore, develop operating speed profile, 85th percentile of motorcycles and cars for tangents and curves been using speed parameter and geometric parameter. The design consistency prediction models are based on two independent, which is normalized area bounded and standard deviation of operating speeds. There are several km along the segment that has a high ranking based on police accident. From the consistency prediction models, can calculate integrated design consistency route FT0050 (KM5 – KM31) of motorcycles and cars are good consistency. This stated although the accident collision occurred between cars and motorcycles, road design consistency is still in good consistency. Factors that may occur are due to the access point along road on the route F0050, and others such as the speed of vehicles itself. Combining the results the road consistency with accidents frequency along the sections, it's expecting to produce the relationships between accidents frequency and road design consistency. For the future research, it's recommended to select more study site selection for further analysis and modeling based on road design consistency.

5 ACKNOWLEDGMENT

The authors would like to thank to the Ministry of Education of Malaysia for supporting the research through the Fundamental Research Grant Scheme (FRGS) Vot. 1452. Appreciation is also given to all individuals and organization that have made this study possible. Thank you is also extended to the Faculty of Civil Engineering and Environment, Universiti Tun Hussein Onn Malaysia.

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1 **Determining the real impact of speed limit enforcement cameras in the UK**

2 John McKerrall Lambert^a, B. Eng, Idris Francis, B.Sc. in Electrical Engineering

3 ^a Director John Lambert & Associates,

4 **Abstract**

5 The first UK fixed speed camera was installed in 1992; by 2005 there were 4000 and a peak of
6 6000. Most fixed cameras are painted yellow, and have markings on the road which provide a
7 secondary speed check. Most sites were selected based on 4 or more KSI crashes over a recent
8 three-year period.

9 Research papers since the late 1990's have estimated the effect of speed cameras on crash rates,
10 with KSI reductions ranging from 65% in early reports to 22% in Allsop (2013) report. Other
11 researchers in the UK and elsewhere considered these claims to be excessive given that police
12 analyses of contributory factors from 2005 consistently show that only ~8% of KSI crashes identify
13 exceeding the speed limit as a “likely” or “possible” contributory factor – and even then not
14 necessarily as the primary cause.

15 Idris Francis decided to undertake a thorough investigation. He obtained details of some 5 million
16 injury crashes from 1987 to 2011 from the UK Data Archive and obtained from police or UK Safety
17 Camera Partnerships the precise location and installation month for cameras in London, Wales,
18 Scotland and 19 other police areas in England, covering more than 50% of all such UK collisions.

19 The crash histories of camera sites were then compared to areas that did not receive cameras. With
20 the benefit of very large volumes of (monthly, not annual) data that minimise random effects it
21 became clear that the benefits long claimed simply do not exist and were instead the result of
22 seriously flawed analysis of insufficient and imprecise data, coupled with astonishing willingness to
23 believe impossibilities.

24 It should also be noted that if cameras reduced crashes, those effects would be bound to start at the
25 time of installation and reach a maximum within a matter of months as the proportion of drivers
26 aware of the cameras reaches a maximum – and not continue to provide increasing benefit year after
27 year. That characteristic dip at just the right time does indeed exist, but not remotely to the extent
28 long claimed, nor do those effects persist.

29 **Note: The term crash has been used throughout except where it is a title – for example Stats**
30 **19 Accident database.**

31 **Introduction – safe driving and the role of speed, speed limits and speed cameras**

32 Safe driving/ riding requires an alert driver/ rider, not impaired by alcohol, illicit drugs, prescribed
33 drugs, a medical condition or fatigue, using occupant protection equipment, and choosing an
34 appropriate speed and appropriate clearance distance to allow them to stop or swerve in time to
35 avoid a collision. Studies show distraction is a factor in around 65% of near crashes and 78% of
36 crashes (Klauer et al 2006). For Australian fatal crashes, alcohol or drugs are factors in around 30%,
37 failure to wear seat belts or helmets in around 20%, fatigue in around 18%, driving below the speed
38 limit but at an inappropriate speed in around 17%, and exceeding the speed limit in around 13%.

39 Speed has been recognised as a factor in crashes from long before the “Speed kills” campaigns in
40 the 1970's, and speed limits and tolerances have in some jurisdictions increasingly been used to
41 limit vehicle speeds. Since the early 1970's there has been a progressive increase in speed

42 detection/speed enforcement equipment - handheld radar and laser speed detectors, mobile speed
 43 cameras, fixed red light and speed cameras, and recently average speed cameras.

44 The first UK fixed speed camera was installed in 1992; by 2005 there were 4,000, and at the peak
 45 there were around 6,000. Most UK fixed cameras are painted yellow, and have markings on the
 46 road which provide a secondary speed check based on two images 0.5 seconds apart. Most sites
 47 were selected for 4 or more KSI crashes over **recent** three-year periods, though long delays between
 48 site selection and installation can cause analysts problems in research.

49 **Contributory factor “exceeding the speed limit” in KSI crashes**

50 UK police have a menu of 77 contributory factors to be applied to reported crashes – on average
 51 there are 1.95 contributory factors for fatal crashes, and 1.75 contributory factors for other casualty
 52 crashes. The contributory factors include *Exceeding the speed limit* and *Travelling too fast for the*
 53 *conditions*. These contributory factors are aggregated against fatal, serious and slight crashes and
 54 published in the Department for Transport statistics Table RAS50001. Data from those tables for
 55 the years 2010 to 2013 have been included in the table below, with the sum of the fatal and serious
 56 crash contributory factors being added together to give a figure for KSI crashes.

57 **Table 1. Table RAS50001: Contributory factors: Reported crashes by severity:**
 58 ***www.dft.gov.uk/statistics***

Year	2010		2011		2012		2013	
	Exceed limit	Too fast	Exceed limit	Too fast	Exceed limit	Too fast	Exceed limit	Too fast
Fatal	221	215	213	207	173	167	216	191
Total	1620		1663		1497		1486	
%	13.6%	13.3%	12.8%	12.5%	11.6%	11.2%	14.5%	13.9%
Serious	1179	1565	1095	1470	1041	1498	1093	1326
Total	18043		18391		18196		16974	
%	6.5%	8.7%	6.0%	8.0%	5.7%	8.2%	6.4%	7.8%
KSI	1400	1780	1308	1677	1214	1661	1309	1517
Total	19663		20054		19693		18460	
%	7.1%	9.1%	6.5%	8.4%	6.2%	8.4%	7.1%	7.8%

59 Note that despite the high emphasis on reducing speeding, the exceeding the speed limit rates have
 60 hardly changed.

61 As shown, exceeding the speed limit is a contributory factor in 6.2% to 7.1% of KSI crashes. It
 62 would be wrong to assume however that cameras could ever bring about similar reductions because
 63 even if speeding were fully eliminated – which it is not – many of those crashes may still happen
 64 due to other causal factors.

65 Note that exceeding the speed limit was stated as a contributory factor in 3.7% to 4.4% of slight
 66 injury crashes over the period.

67 The USA FARS crash database also has a speeding related data entry variable. It covers “Racing”
 68 “Exceeded Speed Limit” and “Too Fast for Conditions.” Council (2010) contains data for North
 69 Carolina and Ohio crashes shown below. Note severity categories do not allow a direct
 70 determination of the % of KSI crashes where exceeding the speed limit was a factor.

71

72 **Table 2: Frequency and number/percentage of Speed Related crashes regarding crash severity in**
 73 **North Carolina 2002-2004**

Severity	Over limit		Too fast		Total speed		Not speed		Total No
	No	%	No	%	No	%	No	%	
Fatal injury	733	24.0%	404	13.2%	1137	37.3%	1913	62.7%	3050
Disabling	835	13.8%	1076	17.7%	1911	31.5%	4156	68.5%	6067
Evident injury	2916	7.7%	6427	17.0%	9343	24.7%	28536	75.3%	37879
Totals	4484	9.5%	7907		12391		34605		46996

74 As shown in Table 2, 9.5% of all casualty crashes involved “Exceeding the speed limit.”

75 **Table 3: Frequency and number/percentage of Speed Related crashes regarding crash severity in**
 76 **Ohio 2003-2005**

Severity	Over limit		Too fast		Total speed		Not speed		Total No
	No	%	No	%	No	%	No	%	
Fatal injury	398	20.8%	156	8.2%	554	29.0%	1355	71.0%	1909
Incapacitating injury	1720	15.0%	753	6.6%	2473	21.6%	8994	78.4%	11467
Non-incapacitating injury	5255	11.0%	3668	7.7%	8923	18.6%	38995	81.4%	47918
Totals	7373	12.0%	4577		11950		49344		61294

77 As shown in Table 3, 12.0% of all casualty crashes involved “Exceeding the speed limit”

78 These percentages are higher than the UK percentages but still much lower than many of the
 79 claimed impacts of speed cameras on casualty crashes.

80 **Evaluations of the impact of speed cameras on road trauma – Australia references**

81 An evaluation of Victoria, Australia’s covert mobile speed camera program – Cameron (1992) -
 82 found a reduction in casualty crashes across Victoria of ~ 20%; in Melbourne of ~22%; in rural
 83 Victoria of ~ 18%; and on Melbourne’s arterial roads a reduction of ~ 33%.

84 And in a more general review of speed camera programs – Cameron (2006) - referenced reductions
 85 in Table 2 as detailed below:

- 86 • Fixed and/or known/signed installations –:
 - 87 ○ In Great Britain — Local reductions of 65% in serious casualty crashes for fixed
 - 88 cameras and 28% for mobile cameras;
 - 89 ○ In New Zealand – Local reduction of 28% in serious casualty crashes for mobile
 - 90 cameras, and a general reduction of 13%; and
 - 91 ○ In Queensland Australia - Local reduction of 35% in casualty crashes, and a general
 - 92 reduction of 26%;
- 93 • Unsigned sites or zones – in Victoria Australia:
 - 94 ○ General reductions of 21% for Victoria & 32% for Melbourne with mobile cameras; and
 - 95 ○ a 21% reduction in serious casualties per crash for mobile cameras in Melbourne.

96 **Evaluation of the impact of speed cameras on road trauma – formal UK research**

97 In their executive summary PA Consulting (2005) claimed that after allowing for long-term trend
 98 there was a 22% reduction in personal injury collisions (PIC) at camera sites, a 42% reduction in
 99 persons killed or seriously injured, and a 32% reduction in persons killed. Appendix H of the report
 100 noted that after using the empirical Bayes analysis to try to deal with regression to the mean
 101 (RTTM) the reduction in personal injury collisions at camera sites was 16% and the reduction in
 102 KSI collisions was 10%.

103 In Allsop (2010) a number of approaches were taken to estimating the effect of speed cameras on
 104 PIC and KSI crashes. These included adjustments for trend, the use of reductions in average speeds
 105 at camera sites plus Nilsson / Elvik power relationships to predict reductions, and the use of
 106 empirical Bayes analysis. All of these can now be shown to be seriously flawed.

107 And in Table 7 of that report it was estimated that overall the reductions were as shown below.

108 **Table 4: Allsop (2010) estimated % reductions in PIC and KSI crashes**

Type of site	Percentage reduction - PIC	Percentage reduction - KSI
Fixed urban	between 20% and 25%	between 35% and 40%
Fixed rural	between 20% and 30%	between 30% and 50%
Mobile urban	between 15% and 20%	between 15% and 30%
Mobile rural	between 10% and 15%	between 15% and 30%

109 In Allsop (2013a) the executive summary references a 25% reduction in PIC after establishment of
 110 cameras, and a 38% reduction in KSI crashes.

111 In Allsop (2013b), the executive summary references a 22% reduction in PIC after establishment of
 112 cameras, and a 32% reduction in KSI crashes.

113 All these claims much exceeded the contribution of speeding to crash causation in the first place as
 114 noted previously, even though speeding had far from been eliminated. This was primarily because
 115 of analysts' abject failure to understand and deal properly with trend and RTTM.

116 **Concerns about the formal UK speed camera research**

117 The principal concerns are the large differences between the *exceeding the speed limit* contributory
 118 factor reported by police – 11.6% to 14.5% for fatal crashes; 6.2% to 7.1% for serious crashes and
 119 3.7% to 4.4% for slight injury crashes – and the reported reductions in deaths and casualty crashes
 120 at camera sites. Note that the figures include both possible and very likely contributory factors. This
 121 led to a number of individuals undertaking their own research into the effectiveness of speed
 122 cameras. (Note – the above para arguably makes some of my earlier changes unnecessary.
 123 Idris)

124 **An alternate approach to researching speed camera effectiveness**

125 This research is based on the hypothesis that:

- 126 • Cameras cannot have an impact until they are installed;
- 127 • Any impact commences immediately after installation and rapidly reaches a maximum;
- 128 • As a result the month of installation, and monthly crash data is required for camera sites and
 129 other similar sites where cameras were not installed; and
- 130 • By comparing trends in numbers of crashes at camera sites, and trends at other similar sites
 131 from a period well before installation to well afterwards any step reduction in crashes at
 132 camera sites would be easily identified.

133 **Research by David Finney, an electronics engineer**

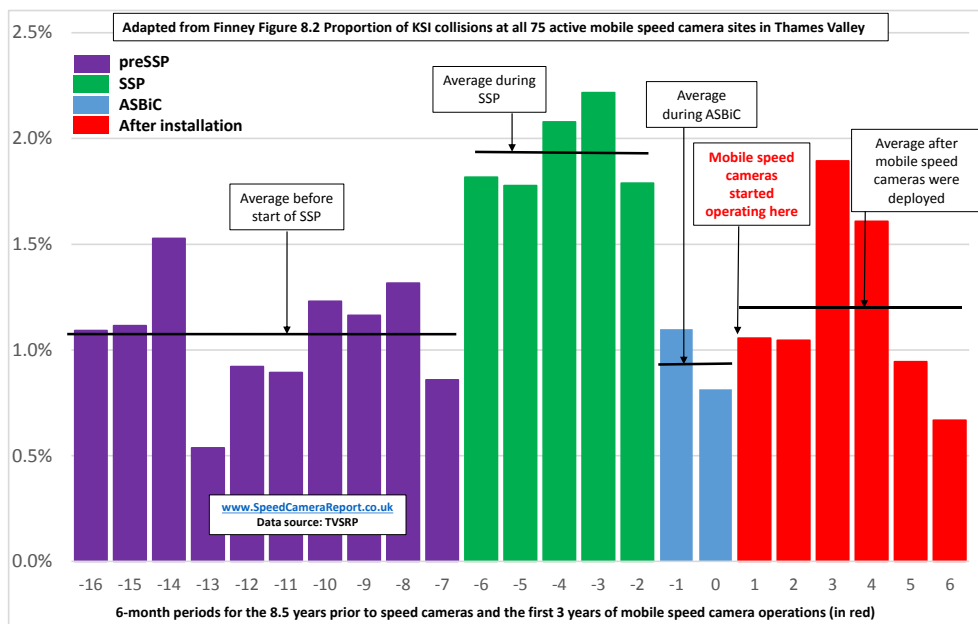
134 Dave Finney's research relates to Thames Valley speed cameras and is available at
 135 <http://www.speedcamerareport.co.uk>. The data used by David was provided and verified by the
 136 Thames Valley Safer Roads Partnership (TVSRP), the organisation responsible for the cameras. In
 137 the process of developing a robust speed camera assessment method David recognised a number of
 138 issues and developed ways to work around them. For example, to deal with any trend effects (the
 139 influence of other factors on crash numbers, his analysis used the proportion of crashes at speed

140 camera sites compared to the total number of crashes in the Thames Valley area. And to obtain
 141 meaningful results, crash histories for mobile camera sites, and for fixed camera sites were
 142 separately aggregated.

143 He also determined that there are four periods (others had used as few as two) relating to site data:

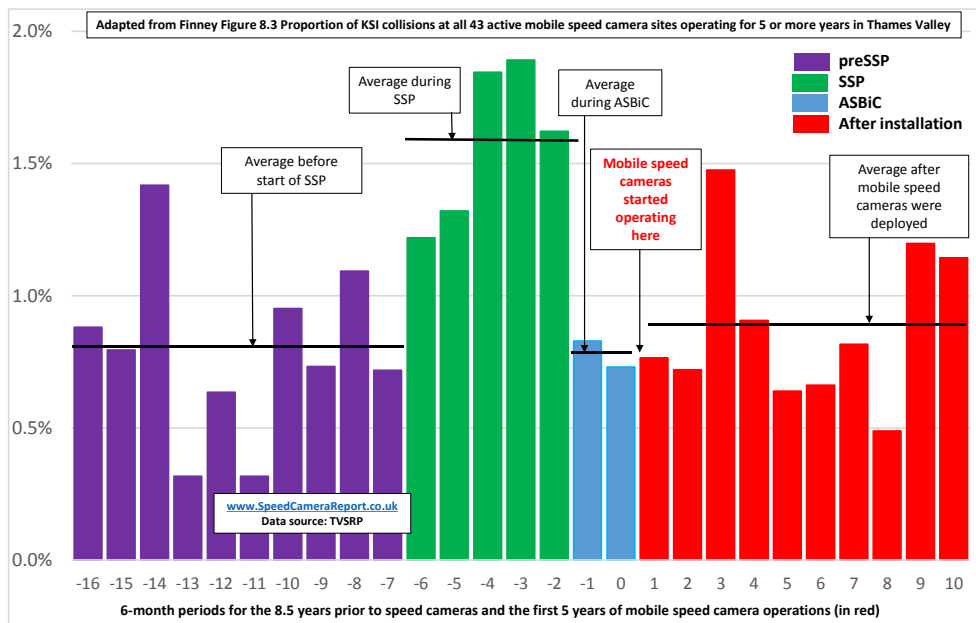
- 144 • the period prior to the site selection period (pre-SSP);
- 145 • the site selection period (SSP) –most sites have been chosen based on at least four KSI
 146 crashes over a recent three-year period. Due to chance variation in site KSI numbers, the
 147 totals are abnormally high and these are bound to return to normal the moment selection ends
 148 (RTTM);
- 149 • the period after the SSP but before the cameras were commissioned (ASBiC) when numbers
 150 have already returned to normal; and
- 151 • finally, the period during which the speed cameras were operating.

152 Often, only limited official data is to hand to allow–identification of these four periods and
 153 accordingly it is necessary to analyse site data to identify the SSP period. This is a particular
 154 problem with fixed speed cameras because the time between site selection and commissioning
 155 varies greatly due to planning and logistical problems. Mobile speed camera sites become
 156 operational as soon as the mobile speed camera vehicles are deployed, but that does not mean there
 157 are no delays.



158
 159 **Figure 1. Adapted from Finney figure 8.2 - proportion of all KSI collisions at all 75 active mobile**
 160 **speed camera sites in the Thames Valley**

161 As shown in Figure 1 in the pre-SSP period on average 1.07% of all Thames Valley KSI collisions
 162 occurred at these mobile speed camera sites. This percentage rose to 1.94% in the SSP period,
 163 dropped back to 0.96% in the ASBiC period, and rose to 1.20% over the 3 years after the mobile
 164 speed cameras started operating. Finney recognised that abnormal selection period data should
 165 never have been used as a baseline reference and accordingly sought other ways of determining
 166 normal levels..



167

168

169

Figure 2. Adapted from Finney figure 8.3 - proportion of all KSI collisions at all 43 active mobile speed camera sites operating for five or more years in the Thames Valley

170

171

172

173

As shown in Figure 2 will in the pre-SSP period on average 0.79% of all Thames Valley KSI collisions occurred at these mobile speed camera sites. This percentage rose to 1.58% in the SSP period, dropped back to 0.78% in the ASBiC period, and rose to 0.88% over the 5 years after the mobile speed cameras started operating.

174

175

In both cases the pre-SSP period figures and the ASBiC figures are similar as expected. And as expected the crash rate in the SSP period is significantly higher.

176

177

178

However in contrast to earlier claims, post commissioning crash rates are in fact higher than the prior level. Prima facie this confirms that cameras had a negative impact on reducing crashes and road trauma.

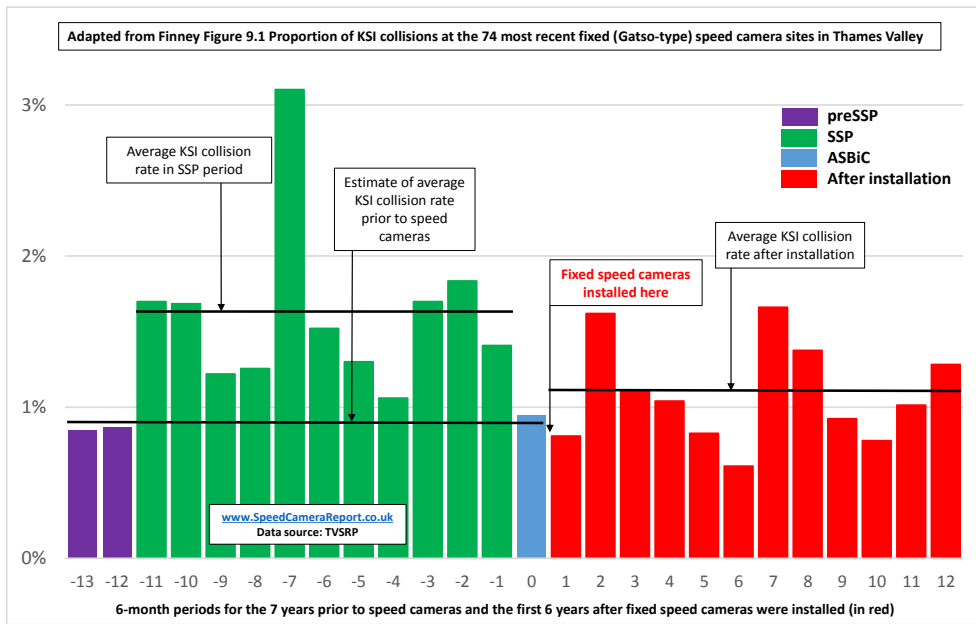
179

180

181

182

There were a total of 212 fixed speed camera sites in the Thames Valley. For many sites Finney was unable to source crash data from well before the commissioning of the cameras. As a result he was unable to identify a pre-SSP period. In order to get reliable pre-SSP data, data for the 74 most recently installed fixed speed cameras was separately analysed as shown in figure 3.



183

184

185

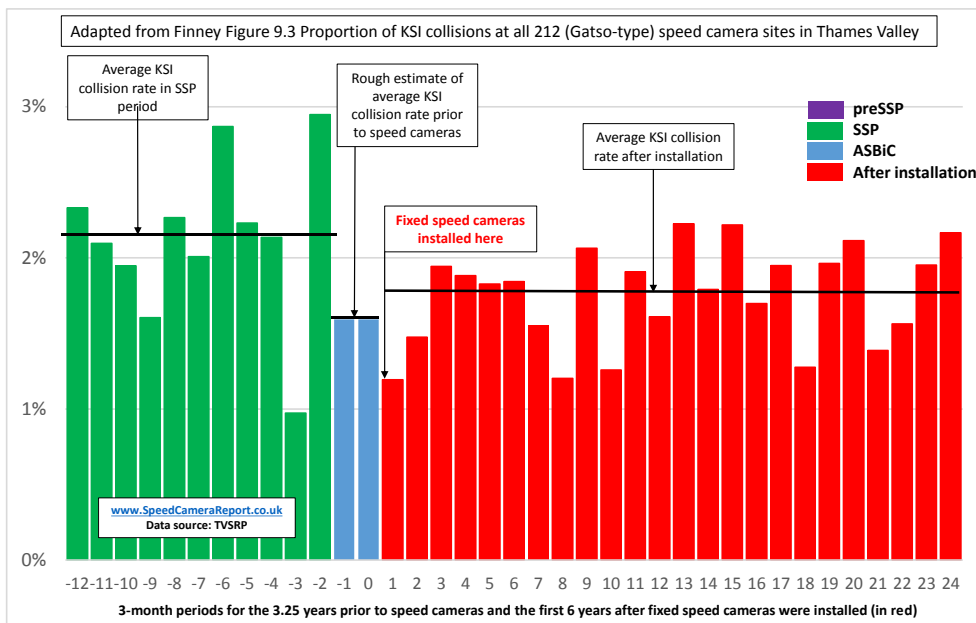
Figure 3. Adapted from Finney figure 9.1 - proportion of KSI collisions at all 74 most recent fixed (Gatso – type) speed camera sites in the Thames Valley

186

187

188

In the pre-SSP period and the ASBiC period 0.89% of all Thames Valley KSI collisions occurred at these fixed speed camera sites. This percentage rose to 1.62% in the SSP period, and was 1.09% over the 6 years after the fixed speed cameras were installed.



189

190

191

Figure 4. Adapted from Finney figure 9.3 - proportion of KSI collisions at all 212 fixed (Gatso – type) speed camera sites in the Thames Valley

192

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196

As shown in Figure 4 there is no identified pre-SSP period. However the ASBiC period, being free of selection bias, is arguably a better indication of “normal” than levels some years before that have been affected by trend effects. On average 1.59 % of all Thames Valley KSI collisions occurred at these fixed speed camera sites during the ASBiC period. In the SSP period the rate was 2.13%, and in the after installation period the rate was 1.75%.

197

Again the post-commissioning crash rates were higher than the ASBiC rate.

198

199 **Research by Idris Francis**

200 In the early 2000's statisticians claimed that speed cameras in the UK were achieving tremendous
201 road safety gains with reductions in crashes of the order of 40% to 60%. And even before the DfT
202 causal factor analysis was first published in 2006, many independent observers, mostly engineers
203 more used to working with data than probability theory, refused to believe that such large
204 reductions could ever be achieved with such modest reductions in speeds.

205 In order to determine the real effectiveness of speed cameras Francis decided to obtain a great deal
206 more than others had used, including precise locations and dates. He also obtained the precise
207 locations and installation dates of as many cameras as possible. Currently that includes details for
208 22 out of 43 police areas including the three largest areas. His aim was to compare the crash
209 histories of camera sites with those where there were no cameras, on a very large scale.

210 ***Issues – form and quality of data***

211 While some official site data has been published it is difficult or tedious to collate especially when
212 provided as separate pdf or Excel sheets for each camera site. Stats19 Police data on the other hand
213 is published annually in a consistent format so that re-formatting into convenient databases is
214 simple.

215 ***Methodology***

216 Idris obtained some 5 million Stats19 Accident and related casualty records from 1987 to 2011 from
217 the UK Data Archive, including the 6 digit Easting and Northing grid references and the date and
218 speed limit of every crash. It was then entered into a Silicon Office relational database software
219 which he had used for 30years.

220 The official data contained many errors and omissions, including absurd location codes showing
221 crashes miles out to sea! In order to address this issue the northing, easting, southing, and westing
222 coordinates of the extremities of each police area were identified and where discrepancies appeared
223 each record was reviewed. More recently, the Department of Transport issued corrected location
224 data from 2000 onwards.

225 ***Stats19 Data***

226 The police records of every reported injury road crash routinely published by the DfT do not
227 include all of the details needed for speed camera analysis, but the csv files available to researchers
228 from the UK Data Archive include all recorded Stats19 information (other than causation
229 assessment, considered too sensitive for inclusion). However there are restrictions prohibiting
230 copying the data to others unless in summary or redacted form.

231 ***Annual Crash, Casualty and Other Data used in this analysis***

232 Stats19 uses one crash file and one casualty file for each year, each casualty record being linked to
233 the relevant crash by unique codes. For the most part this analysis uses crashes not casualties.
234 However the comparative results are much the same for both.

235 Separate Excel files for Slight Injury and Fatal/ Serious injuries show as a minimum:

- 236 • Police Code 9 digits (2 being Force code, the remaining 7 are normally of no significance
237 other than for identifying particular crashes;
- 238 • Limit Relevant Speed Limit;
- 239 • EW Ordnance Survey Easting Grid Reference, 6 digits identifying the East/West location
240 within 1 metre;

- 241 • NS Ordnance Survey Northing Grid Reference, 6 digits identifying the North/South
242 location within 1 metre;
- 243 • WHEN Numerical code for date of crash, e.g. Year *12 + month;
- 244 • DIST Initially blank field to hold distance from nearest camera; and
- 245 • DIFF Initially blank field to hold the difference in months
- 246 As might be expected, crash and casualty trends are very similar so the analysis only uses crash
247 data. This cuts down the data handling requirement as only the collision files are needed.

248 *Camera Site Data (where available)*

249 These files, one per Partnership area, generally provide the following data

- 250 • Police Code 2 digits
- 251 • EC Ordnance Survey Grid Reference, 6 digits identifying the East/West camera
252 location.
- 253 • NC Ordnance Survey Grid Reference, 6 digits identifying the North/South camera
254 location within 1 metre
- 255 • WHEN Numerical code for installation month of camera, e.g. Year *12 + month;

256 To date, speed camera data for 22 police areas has been obtained and analysed. In some cases it was
257 obtained despite the apparent reluctance of Partnerships to provide it, while others claimed not to
258 hold it. Given that the Stats19 data is available to them, and they must know where and when their
259 sites were installed, the reality is that the data is available. They have not bothered to collate it to
260 date.

261 *Distances between Crashes and Casualties in Relation to Camera Sites.*

262 From this point on, the raw crash/casualty data was split up into separate files for each police area.
263 Then, given the co-ordinates of crashes and camera sites, within one area, Pythagoras Theorem can
264 find the nearest camera to each crash and enter that distance into the DIST field of each crash
265 record.

266 It was found that indexing camera locations by the first 3 digits of the Easting codes greatly speeds
267 up the processing.

268 *Months between Crashes and Casualties in Relation to Camera Site Installations.*

269 Similarly, comparing the WHEN codes of the accident and camera installation, the DIFF field of
270 the accident record can be entered and stored.

271 *Processing*

272 The resulting data may be processed to provide the following:

- 273 • A single Excel sheet showing 1987-2011 Stats19 fatal and serious crash (FSC) data
274 aggregated by **police area, year, distance from camera site if within 1km, speed limit, and**
275 **delay in months between crash and camera installation.** This allows large numbers of
276 graphs to be drawn very quickly.
- 277 • A similar sheet covers Slight Injury Collisions.
- 278 • 1987-2011 Stats19 FSC data for all police areas by area and month
- 279 • As above for each camera in 22 police areas for any desired site radius.

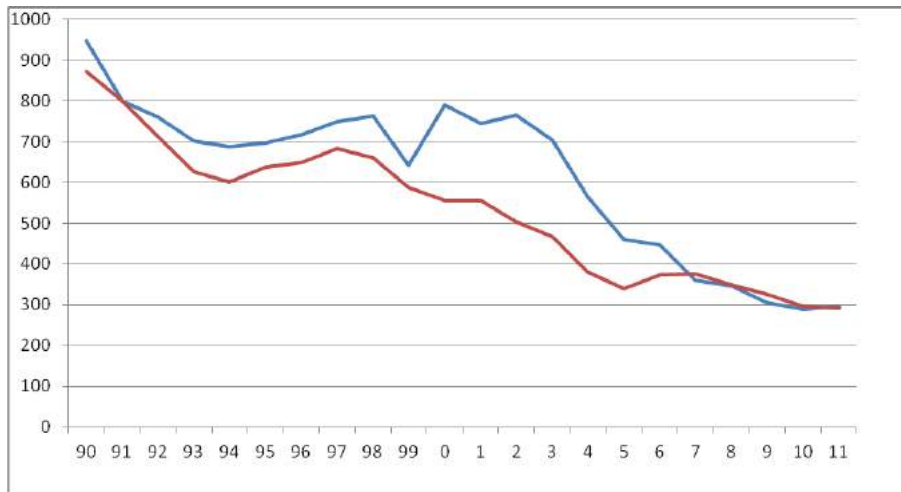
280 *Analysis*

281 Analysis only requires the application of simple arithmetic to FSC crashes near any one of 3,400
282 camera positions in the 22 police areas for which data is currently available.

283 Critically, with this method there is no need for estimates, probability theories, mathematical
284 models, or any other manipulation of the data in order to make comparisons between sites where
285 cameras have been installed, and similar sites where cameras have not been installed.

286 And if a camera or group of cameras have any beneficial effect, it will appear as a quite sudden
287 reduction in accidents their reach camera from the month of installation onwards.

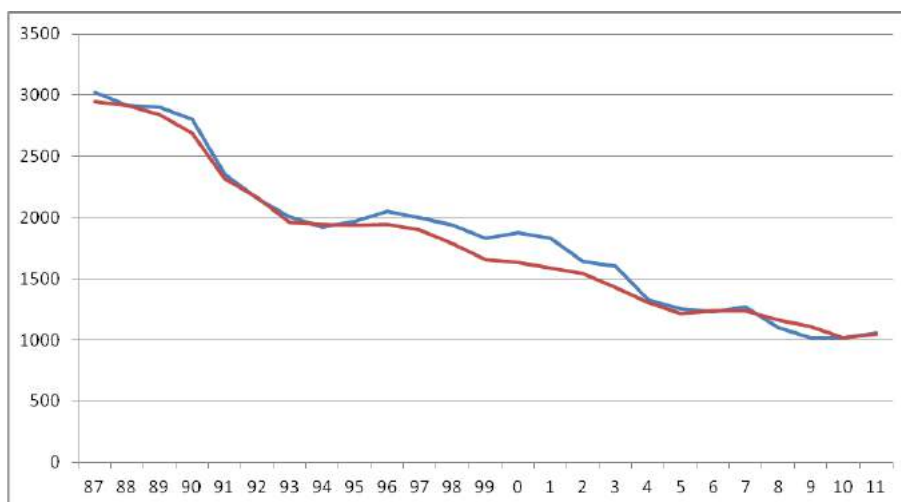
288 In the first two of the following graphs, to make comparison easy, the numbers of non-camera site
289 KSI have been scaled down to equal the number of KSI at camera sites in 1991. The first method
290 (Fig.5) compares FSC rates near cameras with rates elsewhere in the same police area(s) Site
291 selection based on high numbers of crashes causes previously common trends to diverge and after
292 selection this effect ends, leading to convergence. Once past that transient effect, camera benefit, if
293 it existed, would be confirmed by crash numbers falling further than non-site data. It is clear that
294 they do not.



295

296 **Figure 5. Method 1 – London camera site KSI in blue (475 Sites -last one installed in 2008)**
297 **versus non-camera site KSI crashes in red**

298 After the separation in the two graphs caused by camera sites being selected for high crash rates, the
299 two graphs converge from 2007 onwards – there is no difference in the trends in the number of
300 crashes, the scaled trendlines overlapping in 1991, and then again overlapping from 2007 through to
301 2011.



302

303 **Figure 6. Method 1 – KSI in England and Wales within 500m of camera locations on 30mph**
 304 **roads - camera site KSI crashes in blue versus non-camera site KSI crashes in red**

305 There is no discernible difference in the scaled trendlines in numbers of crashes in the two graphs
 306 either side of the period in which most cameras were installed - around 1995 to 2004. This graph of
 307 sites on 30 mph roads represents about 60% of all camera sites identified in the 32 police areas.

308 The above two graphs and some others use Transport for London data published in July 2014.
 309 Unfortunately London is one of only 10 or 11 areas to have published sensibly usable data, and the
 310 only where the volume of data is in any case large enough to allow accurate analysis. For that
 311 reason the second method used in this analysis is based in circular sites centred on cameras, for
 312 these reasons:

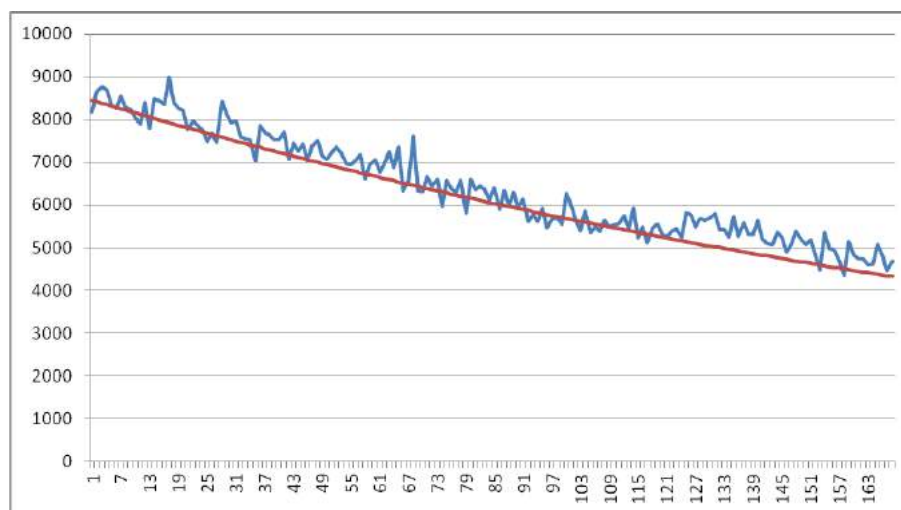
- 313 • It bypasses the failure of so many partnerships to publish usable data and so thereby allows a
 314 much larger scale analysis which is inherently more accurate.
- 315 • It allows monitoring of the distances over which camera effects extend, which partnerships
 316 are unable to do with their fixed and necessarily subjective site boundaries.

317 Doubtless some who would prefer not to believe the results provided, will claim that they are
 318 invalid because they do not conform to a view of where site boundaries should be drawn. Research
 319 known to the authors tends to show that the effect of speed cameras on speed behaviour typically
 320 extends around 700 m either side of the camera. For that reason boundaries have been chosen up to
 321 1000 m from the camera location.

322 Also, if crash numbers appear to fall within narrowly defined boundaries, they should also fall
 323 within wider circular boundaries. If they do not, that can only be because the cameras are merely
 324 shifting crashes from one place to another (by for example by causing drivers to divert) rather than
 325 reducing them

326 It brings consistency to the results, as opposed to the variations arising from the different ways in
 327 which site boundaries are determined by different partnerships.

328 This second method uses monthly data for cameras installed between 1994 and 2008, adjusted
 329 relative to camera installation month (in this case set as month 85) to identify and quantify any
 330 reductions in KSI following installation..



331
 332 **Figure 7. Method 2 –FSC within 500 m of cameras for 22 Police Areas—with installation**
 333 **adjusted to month 85. Average Trend in red > 1km from cameras, * 0.996/ per month**

334 There is no identifiable drop in KSI within 1km of the cameras after installation in month 85.

335 Hundreds of graphs can be drawn from the each single Excel sheet of crash data, for different
336 combinations of police area, speed limit, radius from camera and camera type. Every graph drawn
337 to date shows the same results – minimal and short-lived reductions close to cameras and significant
338 increases further away. The same applies to Slight Injury Collisions as to FSC.

339 **Conclusion**

340 Previous research suggests that speed cameras reduce crashes by 22% - 65%. However UK and US
341 contributory factor data shows that *exceeding the speed limit* is only a factor in around 11% to 24%
342 of fatal crashes; 6% to 10% of KSI crashes and 4% to 5% of slight injury crashes. And logic
343 suggests that significantly lower figures than these should be the limit to the effectiveness of speed
344 cameras.

345 In an effort to get around this apparent contradiction both Finney and Francis used unadjusted data
346 (no statistical tools such as the empirical Bayes method were used) to directly determine the impact
347 of speed cameras on crashes. Both their analyses showed that speed cameras have no significant
348 impact on road trauma.

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368 *report*. London: PA consulting group or

1 **Do crash rates really increase with increases in average speed?**

2 John McKerrall Lambert^a, B. Eng,

3 ^a Director John Lambert & Associates,

4 **Abstract**

5 For more than two decades speed limit enforcement has been supported by research that "shows"
6 that crash rates increase with increasing average speed.

7 Safe driving is primarily determined by being alert, being unimpaired, and driving at an appropriate
8 speed and with an appropriate clearance distance for the environment at the time. And note speed
9 limits are not set based on maximum safe speeds.

10 This paper reviews the papers by Nilsson, TRL, Kloeden et al and others that show benefits from
11 reducing average speeds, and shows they contain errors or are inconclusive. It supports that
12 MUARC report 307 correctly determines enforcement cameras have virtually no effect on road
13 trauma even though studies show average speeds are reduced at camera sites. And it reviews
14 Allsop's 2013 report which claims speed cameras reduced KSI crashes, shows no correlation
15 between the change in average speeds and the change in crash rates at camera sites.

16 It reviews research on crash rates by road type and speed limits, and finds no correlation between
17 crash rates and average speeds.

18 This paper shows that it cannot be asserted that crash rates increase with average speed. This has
19 serious implications for the reports that show speed limit enforcement reduces speeds and then use
20 Nilsson, TRL, Kloeden or other results to claim there would be a reduction in casualty crashes.

21 **Introduction – safe driving and the role of speed and speed limits**

22 Safe driving/ riding requires:

an alert driver/ rider, not impaired by alcohol, illicit drugs, prescribed drugs, a medical
condition or fatigue,

the use of occupant protection equipment, and

choice of an appropriate speed and appropriate clearance distance to allow them to stop or
swerve in time to avoid a collision.

23 Speed has been recognised as a factor in crashes from before the "Speed kills" campaigns in the
24 1970s. And speed limits and speed limit tolerances have in some jurisdictions been increasingly
25 used to control vehicle speeds. Since the early 1970's there has been a progressive increase in speed
26 detection/speed enforcement equipment. And in evaluating the effectiveness of these technologies it
27 has been a fairly common practice to measure the effect of the devices on average speeds and then
28 use relationships between average speeds and crash rates to predict changes in crashes.

29 **Speed and crashes – a conceptual model**

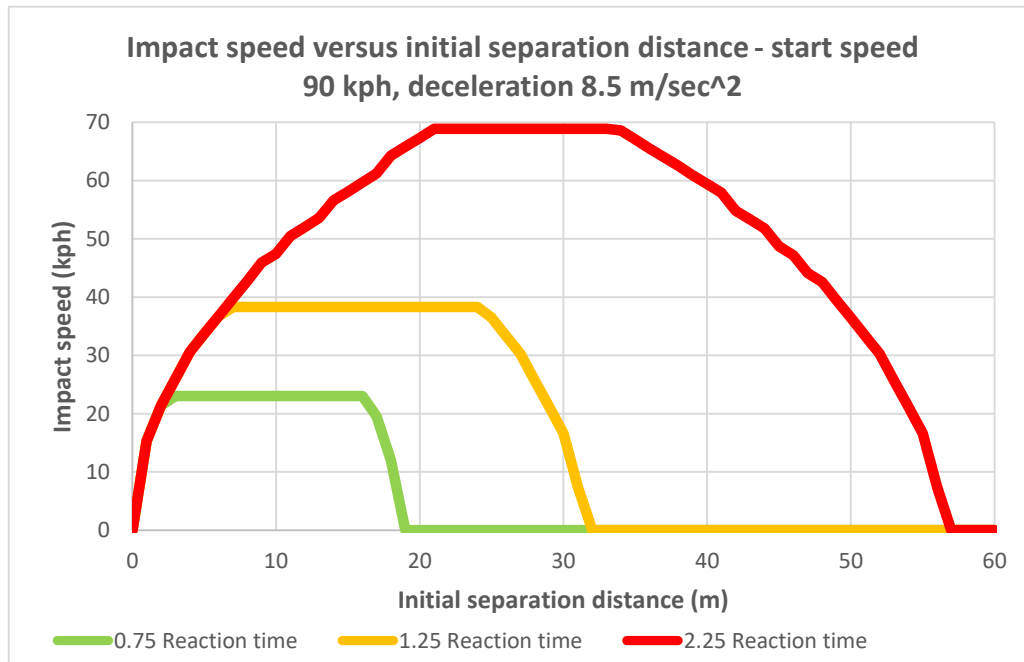
30 For a particular vehicle, driver and situation the chance of a crash is highly variable. Take two
31 vehicles, travelling at 90 km/h, with deceleration rates of 8.5 m/s^2 , and reaction times of 0.75, 1.25
32 and 2.25 seconds, with the front vehicle suddenly braking. The impact speed vs separation distance
33 can be modelled.

34

35

36

Figure 1. Impact speed versus initial separation distance



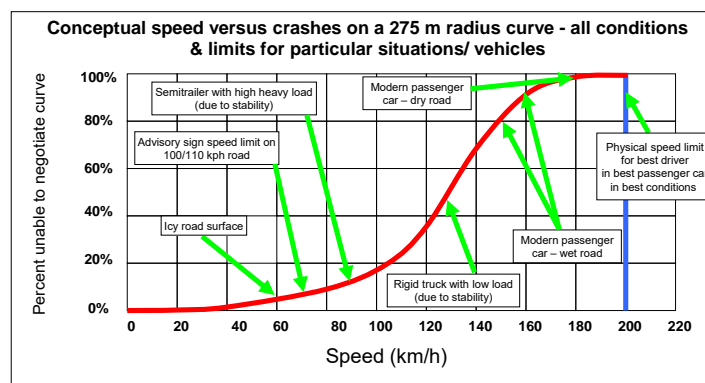
37

38 The results are shown in Figure 1. The reaction time determines the maximum impact speed. Each
 39 graph comprises 4 zones: A period where the following car has not yet applied its brakes; a period
 40 where both cars braking; a period where lead car has stopped; and a period where both cars have
 41 stopped without colliding..

42 For a particular on road situation the overall crash risk factor will depend on the crash risk factors
 43 for the range of vehicles and conditions that occur at that location. For example consider a 275
 44 meter curve with 4.5% superelevation. This curve would be likely to have a 70 km/h advisory speed
 45 limit.

46

Figure 2. Likely distribution of crash risk on a 275 m curve



47

48 In icy conditions vehicles would slide and crash off the road or into each other at around 60 km/h. A
 49 semitrailer with a high heavy load would rollover at around 90 km/h and so on. The crash risk for
 50 the population of vehicles over a range of weather situations approximates an S-curve. And in a
 51 100/110 km/h speed zone the relevant crash risk curve would be the section from 0 to ~120 km/h.
 52 To this crash risk would need to be added risks associated with distracted, impaired or fatigued
 53 drivers crashing off the curve at any speed.

54 **Driver populations and crash rates**

55 Based on all the data I have seen over my 50 years of involvement in road safety and using
56 Australian data I have been able to determine the following approximate crash causation rates.

57 **Table 2: Crash causation rates per years of driving – Australian data**

Crash type	Most responsible 30%	Next 50%	Less responsible 12%	Irresponsible 8%
Fatal	140,000 yrs	42,000 yrs	10,500 yrs	1800 yrs
Serious	8,700 yrs	2,500 yrs	950 yrs	155 yrs
Other injury	500 yrs	150 yrs	90 yrs	15 yrs

58 As shown the responsible 80% of drivers have very low crash rates. The chance of them causing a
59 crash where someone is injured is around once in 2 to 3 driving lifetimes. And necessarily these
60 drivers must be skilled at choosing appropriate speeds and clearance distances to avoid crashes,
61 must not drive when they are unimpaired, and must use occupant protection devices.

62 And being the majority of drivers they have a huge impact in controlling driving behaviour on
63 roads. Except where traffic is light this group basically controls vehicle speeds.

64 **Driver behaviour – speed versus speed limits**

65 **Figure 3. Average speeds versus speed limits on a length of UK B class road**

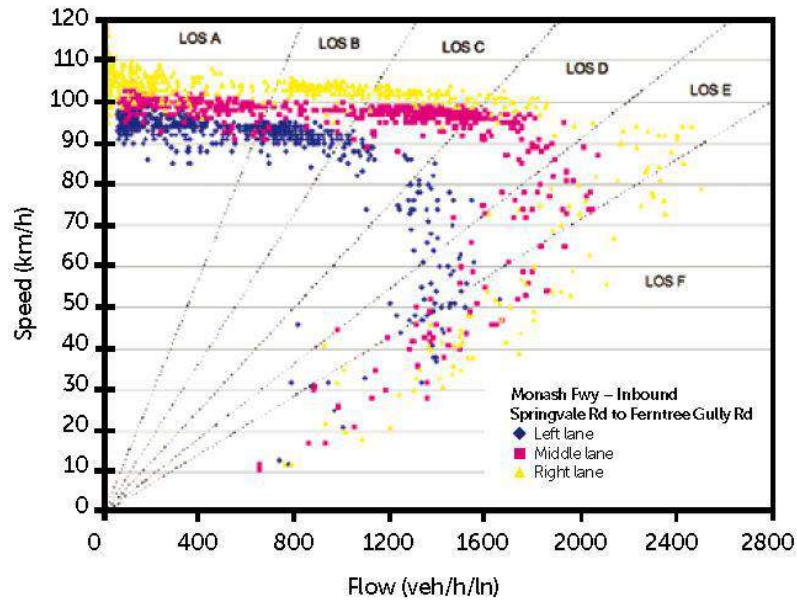


66

67 The Figure 3 is from UK DfT Traffic Advisory Leaflet 2/06. These average speeds are basically
68 controlled by the 80% of responsible drivers. It is interesting to note the degree to which these
69 drivers follow the actual speed limits. In the initial 40 mph zone the drivers chose generally to travel
70 above the speed limit; in the following 50 and 40 mph zones the drivers chose to follow the speed
71 limits with one significant variation; and in the 60 mph zone the drivers chose to travel below the
72 speed limit. Note that the enforcement tolerance in the UK is 10% of the speed limit +2 mph so that
73 except in the initial 40 mph zone the average driver would not be at risk of being infringed.

74 **Average speeds reflect driver's responses to perceived varying levels of risk. And in the 60**
75 **mph zone, average speeds along this 7 km length varied from 40 mph to 60 mph. In a study of**
76 **average speed versus crashes average speed measurements could be highly variable.**

77 **Figure 4. Average speeds versus traffic flow on a section of the Monash Freeway in Victoria**
78 **Australia (speed limit 100 kph)**



79

80 Figure 4 is VicRoads Managed Freeways Handbook (2013) Figure 24

81 In Australia drivers drive on the left side of the road, so the left lane is the slowest lane and the right
 82 lane is the fastest lane in a three lane freeway situation. In the uncongested situation speeds in left-
 83 hand lane average 90-95 km/h; 95-100 km/h; in the middle lane and 100-105 km/h in the right lane.
 84 Once a lane reaches saturation the speeds rapidly drop. And lower speeds equal lower capacity.

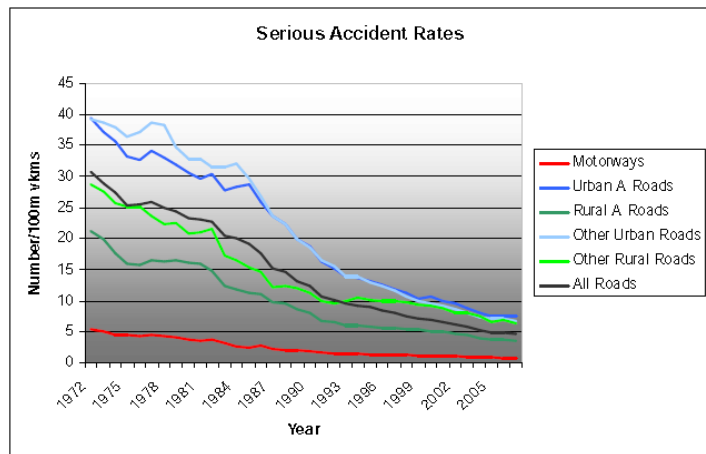
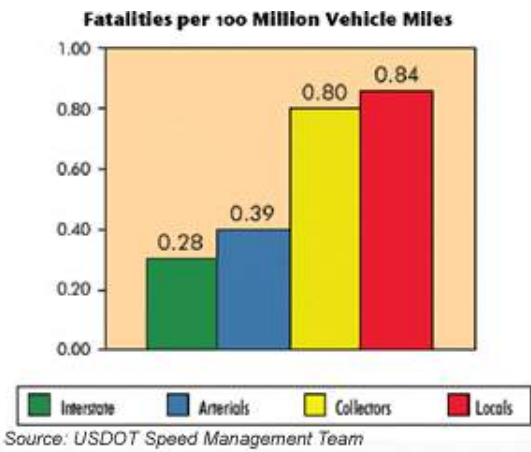
85 As shown in this figure at 400 vehicles per hour in the uncongested situation vehicles are travelling
 86 at about 100 km/h, or 100,000 m/h. Assuming the average vehicle length was about 4.5 m then the
 87 400 vehicles would occupy 1800 m of the 100,000 m. Hence the average distance between them
 88 equals $(100,000 - 1,800) / 400$ or of the order of 250 m. And at 800 vehicles per hour in the
 89 uncongested situation vehicles are travelling at around 96 km/h, or 96,000 m/h. The same
 90 calculation gives a separation distance of the order of 120 m. It would therefore be expected that
 91 crashes would be rare in mid blocks situation. However low traffic flow would allow less
 92 responsible drivers to travel faster than the general traffic stream, and create a risk of serious
 93 crashes. In comparison in the saturation zone and congested zone, the average separation distances
 94 will be 25 metres to 55 m (1.2 to 2.5 seconds) with significant risk of nose to tail crashes especially
 95 at intersections. However because vehicle travel speeds before braking will be similar and lower,
 96 crash severity is likely to be low.

97 **In summary figure 4 reflects a large range of speeds and clearance distances and varying**
 98 **crash type risks. For the purpose of average speeds and crash rates, a single value for average**
 99 **speed for this segment of road or any other high volume road would be a nonsense.**

100 **Crash rates versus speed limits by road class**

101 If crash rates increased with average speed in an absolute sense, it would be expected that the road
 102 classes with the highest average speed would have the highest crash rates.

103 *Figure 5. Trauma rates versus road category USA & UK*



104

105 As shown in the diagrams above (USDOT(2003) Figure 4 & Bayliss (2009) Figure 6) to a large
 106 extent the reverse is true – the roads with the highest speeds – the motorways and freeways – have
 107 the lowest crash rates. And the roads with the lower speeds have the highest crash rates.

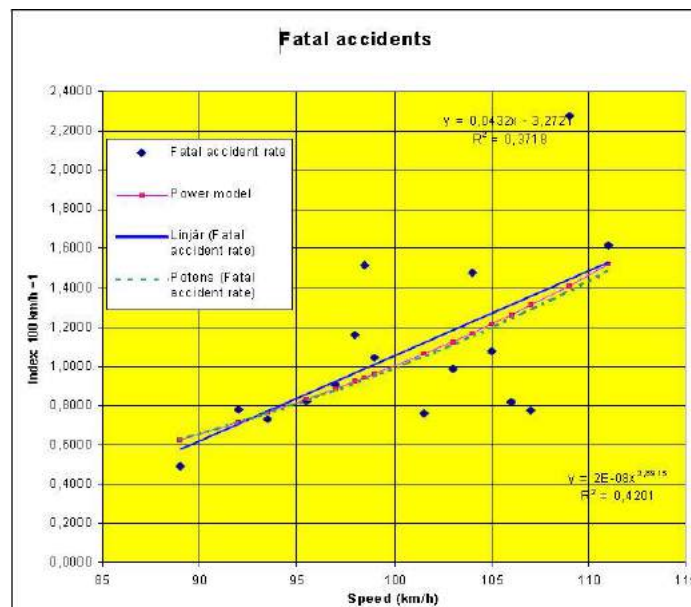
108 **Research by Nilsson (2004)**

109 Nilsson’s research used Swedish National Road Administration 1997 mean speed data for two lane
 110 roads with a road width of 13 m - 43 road sections had a speed limit of 90 km/h and 62 Road
 111 sections had a speed limit of 110 km/h. Crash data was for the period 1991 to 1997.

112 The data was grouped by average speed to give reasonable number of crashes per group. In the 90
 113 km/h zones the groupings were 87-91 km/h (94 crashes), 92 km/h (154), 93-94 (200), 95-96 (144),
 114 97 (190), 98 (190), 99 (116) and 100-112 (165). In 110 km/h zones the groupings were 97-100
 115 km/h (53), 101-102 km/h (63), 103 (163), 104 (136), 105 (104), 106-108 (118), 109 (94) and 110-
 116 112 (96).

117

Figure 6. Nilsson (2004) Figure 28

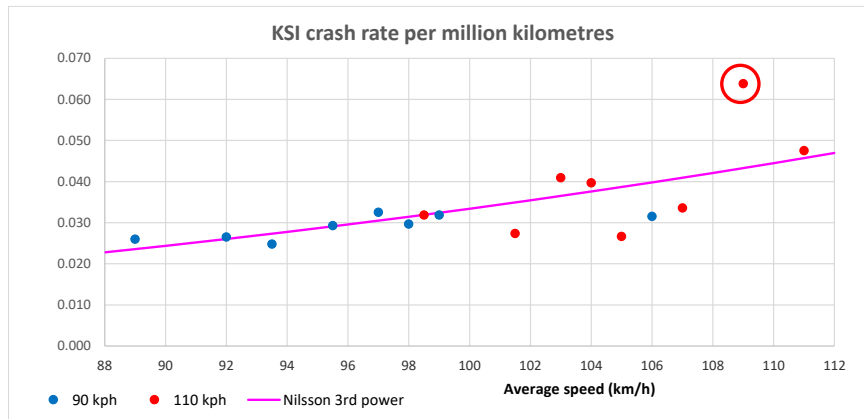


118

119 Figure 6 above shows the 16 fatal accident rates together with Nilsson’s power model curve in pink,
 120 plus a linear model and a power model from be accident rate data. What is not highlighted is the
 121 fact that these data points relate to road sections with different speed limits. And the fact that the
 122 road managers had specified different speed limits is prima facie evidence that they perceived the
 123 risks with the two road types to be significantly different.

124

Figure 7. Based on Nilsson (2004) Figure 28

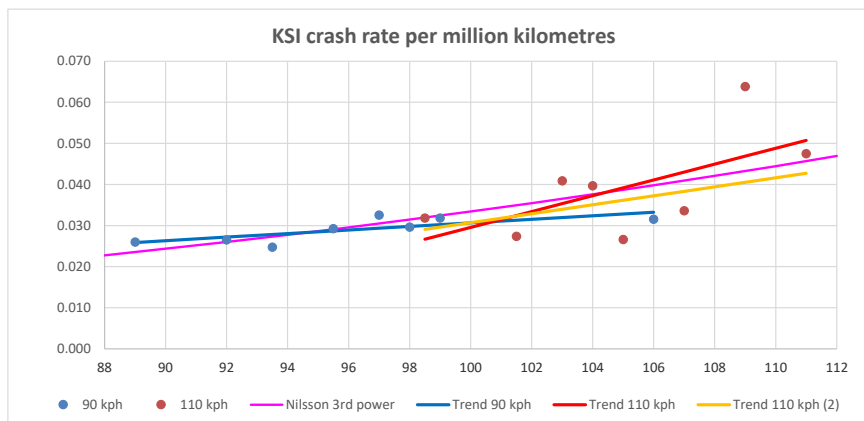


125

126 Figure 57 shows the two groups of data highlighted separately, plus one outlier value circled in red.

127

Figure 8. Trend lines based on Nilsson (2004) Figure 28



128

129 Three linear trendlines have been added in figure 8 above - for the 90 km/h data; for the 110 km/h
 130 data; and for the 110 km/h data less the one outlying value. Note that the linear trendlines are
 131 significantly different in slope to the Nilsson third power model except for the adjusted trend for the
 132 110 km/h data. Similar results are found for other crash types. This brings into question the validity
 133 of Nilsson’s power model.

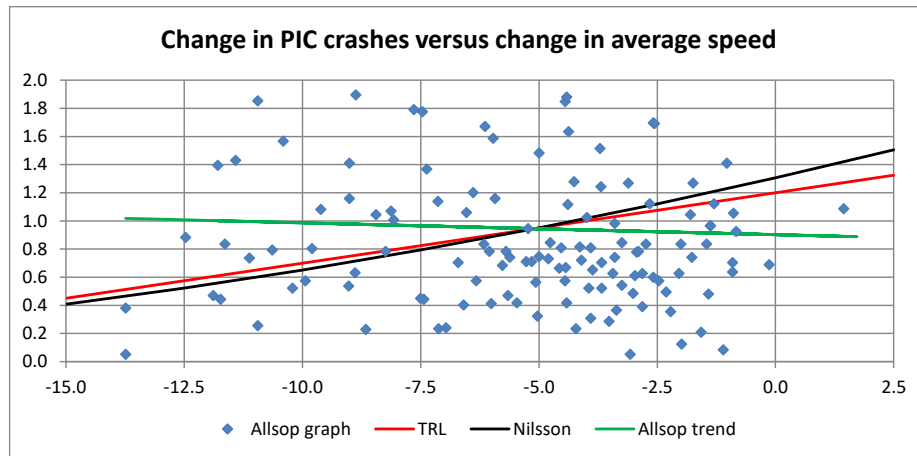
134 **Research by Allsop (2013)**

135 In Appendix 4 - Joint Analysis of Collision and Speed data, data was examined for eight UK Speed
 136 Camera Partnership (Partnership) areas. Where it was clear that one or more observations were
 137 made before establishment of a camera and one or more afterwards, the observations of mean speed
 138 before and after establishment were each averaged, and the difference between the two averages
 139 was taken as an estimate of the change in mean speed in the vicinity of the camera following its
 140 establishment.

141 Changes in mean speed were estimated in this way for 132 cameras in these eight Partnerships, and
 142 ranged from a reduction of 13.7 miles/h to an increase of 1.7 miles/h. All but three were reductions.

143 The change in collision occurrence at the camera concerned was measured by number of personal
 144 injury crashes (PIC) per year in the vicinity of the camera in years throughout which the camera
 145 may have been in operation.

146 **Figure 9. Change in PIC crashes versus change in average speed – 132 UK speed camera sites**



147

148 Allsop's data is shown above with the trend line shown in green. Note he found a slight increase in
 149 PIC crash crashes with reductions in average speed. I have compared Allsop's data and trend with
 150 the predicted trends by TRL and Nilsson based around the average of all Allsop's data. As shown
 151 Allsop's data does not support the TRL or Nilsson trendlines at all.

152 **Research by Kloeden and McLean (1997) – Urban roads**

153 This study was a case/ control study

154 ***Case vehicles***

155 The following criteria were used for the selection of case vehicles: Crash was in the Adelaide
 156 metropolitan area, with a 60 km/h speed limit, not on a section of road with an advisory speed sign
 157 of less than 60 km/h, case vehicle was a car or car derivative, at least one person was transported
 158 from the crash scene by ambulance, case vehicle had a free travelling speed prior to the crash, was
 159 not executing an illegal manoeuvre prior to the start of the crash sequence, the case vehicle driver
 160 did not suffer from a medical condition that caused the crash, and had a zero blood alcohol
 161 concentration (BAC), there was sufficient information was available to carry out a computer-aided
 162 crash reconstruction, the case vehicle did not roll over, and crash did not occur while it was raining

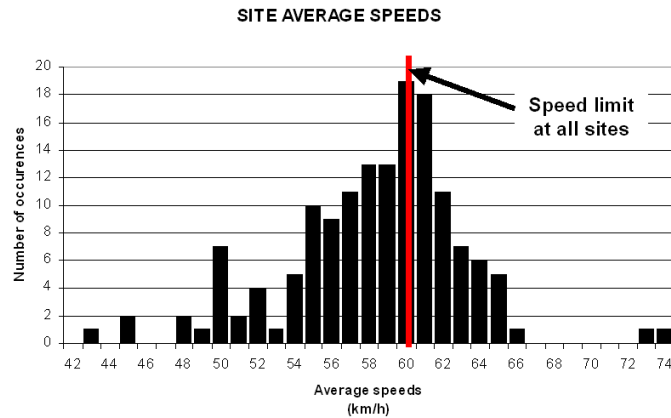
163 Cases were restricted in the interest of uniformity. Higher speed zones would have had
 164 fundamentally different speed distributions which would have made the case-control analysis more
 165 complicated to perform and the results harder to interpret.

166 The end result was that 28% of the notified crashes were selected for analysis. Those crashes were
 167 disproportionately intersection crashes between cross traffic or turning traffic (60% of cases versus
 168 expected frequency of around 17%-20%).

169 ***Control vehicles***

170 The selection of 4 control vehicles were based on same location, weather conditions, day of week,
 171 and time of day as the crash; same direction of travel as the case vehicle; car or car derivative, free
 172 travelling speed, and most were checked for zero BAC.

173 ***Figure 10. Distribution of average speeds of groups of control vehicles***



174

175

As shown average speeds of the control vehicles varied dramatically from 43 km/h up to 74 km/h.

As these average speeds are the result of decisions made the responsible group of drivers, it must be

the case that the risks vary dramatically between sites. Reference to the actual Kloeden and Mclean

data shows that the high speeds were recorded on roads where the pavement width in one direction

was 12 m wide or more, whilst in the low speed situation the pavement width had been restricted to

6 ½ m wide in one direction using traffic calming methods.

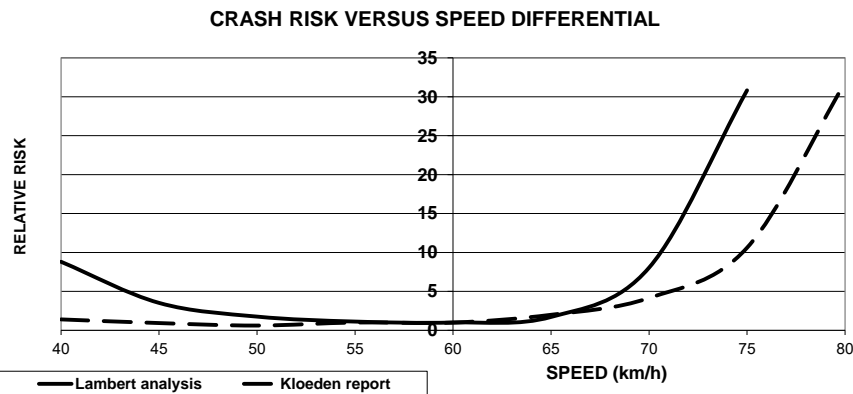
Kloeden and McLean professionally used available information to determine the speed of the case

vehicles prior to the crash. However inexplicably when they analysed the data to determine a crash

risk, they used 60 km/h as the reference speed instead of the average speed of the control vehicles.

184

Figure 11. Crash risk versus speed differential



185

And in figure 11 above the dashed line shows the results of their analysis. Note that there is hardly

any increase in crash risk for vehicles travelling much slower than 60 km/h. This is in conflict with

the general experience in road safety that shows that vehicles travelling much slower than the traffic

stream represent a significant crash risk – hence all the warnings required with slow moving

vehicles.

In my paper Lambert (2000) I reanalysed the data using the control vehicle speeds as the reference

speed. The results of my analysis are shown in figure 11 in the full black line (I have overlaid the

Kloeden and McLean graph placing the zero value at the 60 km/h value of the original graph).

The implications of my analysis is that it's not the average speed differential from 60 km/h that

controls crash risk, it is the variation from the average speed that the 80% of responsible drivers

choose that is the critical factor. And note that as expected vehicles travelling much slower than the

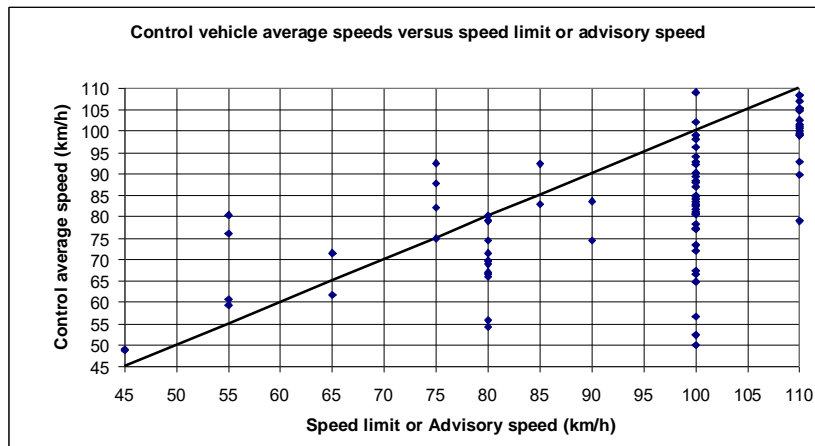
average travel speed also generate a significant increase in crash risk.

In summary the crash rate does NOT double for every 5 km above 60 kph.

199 **Research by Kloeden and McLean (2001) Rural roads**

200 This research project was of a similar design to the previous research project but for rural roads.
 201 The differences were that rather than limiting cases to a single speed limit zone, it covered 80 kmh,
 202 100 km/h and 110 km/h speed zones. In addition the analysis followed the approach in Lambert
 203 (2000).

204 **Figure 12. Control vehicle average speeds versus speed limit or advisory speed**



205

206 As is shown above at most sites the average speed for control vehicles are significantly less than the
 207 speed limit or advisory speed. In only 14 (8.4%) of the 167 cases is the average control group speed
 208 at or above the speed limit or within 5 km/h of the speed limit. Prima facie this reflects that
 209 responsible drivers perceive these sections of road as being of higher risk and therefore reduce their
 210 speed to control that risk. And the implications of this are that the data in this research project is
 211 only appropriate in relation to speed limits at problem locations in the rural road network.

212 The results showed that the crash risk increases significantly where vehicles travel faster than the
 213 speed responsible drivers would choose – it does not show that crash risk varies **with** average speed.

214 **Research by Taylor et al (2002) TRL Report TRL511**

215 This Research Report aimed at determining the relationship between speed and crash rate on UK
 216 Rural roads with 60 mph speed limits.

217 Four Groups of roads were identified that can be broadly described as follows:

218 *Group 1: Roads which are very hilly, with a high bend density and low traffic speed - low quality*
 219 *roads.*

220 *Group 2: Roads with a high access density, above average bend density and below average traffic*
 221 *speed - lower than average quality roads.*

222 *Group 3: Roads with a high junction density, but below average bend density and hilliness, and*
 223 *above average traffic speed - higher than average quality roads.*

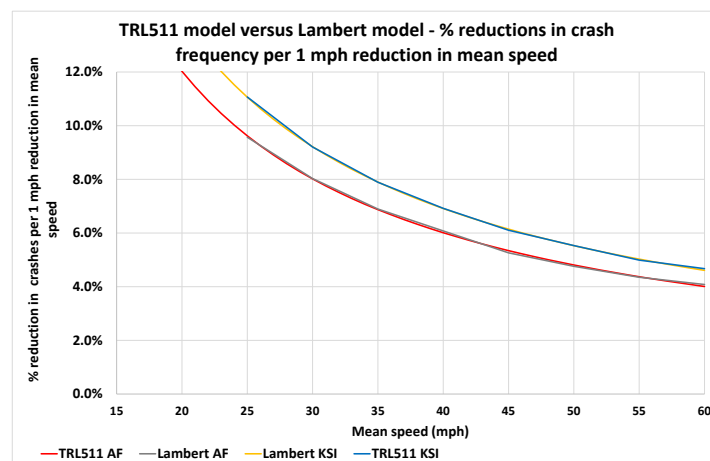
224 *Group 4: Roads with a low density of bends, junctions and accesses and a high traffic speed - high*
 225 *quality roads.*

226 Unfortunately nowhere in the paper is the base data displayed so that readers are faced with a black
 227 box analysis. Two model forms were developed – Level 1 which was of the form accident count =
 228 Function(years of accident data; AADT flow; link length; mean speed); and Level 2 which was of
 229 the form accident count = Function(years of accident data; AADT flow; link length; mean speed;
 230 road geometry). Model results were presented in Figure 3 of the report.

231 The Report is puzzling to the writer for a number of reasons. Firstly it contains none of the base
 232 data. Figure 2 represents the model for a very specific situation, and figures A1 and A2 are a
 233 synthesised construction to demonstrate a masking situation.

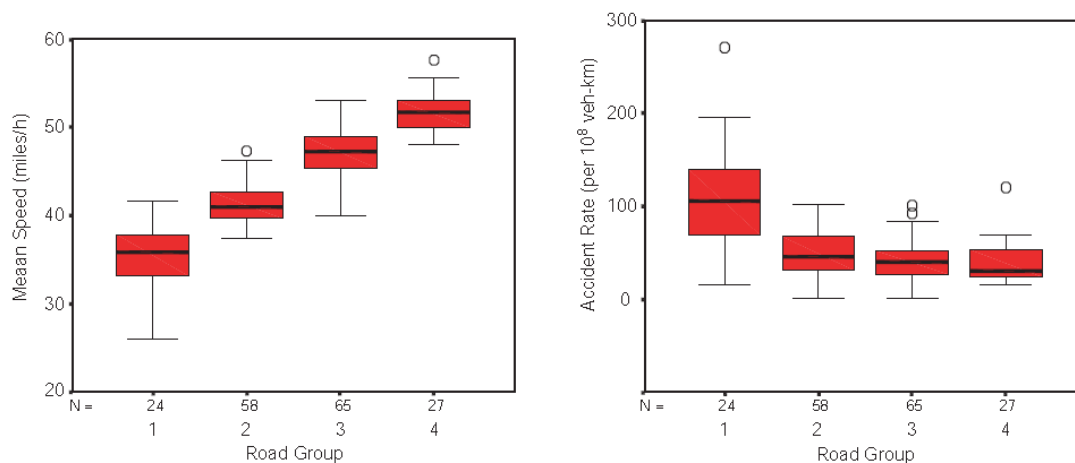
234 Further the trends shown in Figure 3 of the report did not make sense to the writer in relation to any
 235 hypothesis as to what factors would drive reductions in crash frequency or KSI crash frequencies
 236 versus mean travel speeds. This is especially so given that responsible drivers are very good at
 237 adjusting speeds and mean speeds to maintain a high level of safety. I decided to analyse the trend
 238 lines in Figure 3 of the report. That analysis showed the crash frequency trend line $\approx 2.405 \times (1/V)$;
 239 and the KSI crash frequency trend line $\approx 2.765 \times (1/V)$. The correlation between the TRL511 report
 240 data and my model is shown in figure 13 below.

241 **Figure 13. Comparison between TRL511 trend lines and my model**



242

243 The authors offer no hypothesis as to why % reductions are proportional to the inverse of the mean
 244 speed on rural single carriageway roads.



245

246 As shown in the segment of Figure 1 of the TRL report, low quality group 1 roads are associated
 247 with low mean speeds and high crash rates, whilst high-quality group for roads are associated with
 248 high speeds and low crash rates. Hence the relationship shown in figure 13 above may be the result
 249 of road standard rather than mean speed.

250 Conclusion

251 Almost universally studies into mean speeds versus crashes have failed to recognize that a) speed
 252 alone never defines safe driving – it is speed and clearance distance that underlies safe driving; b)
 253 that on heavily trafficked roads traffic flow has a complex and dramatic impact on speed and on

254 types of crashes; c) that on lightly trafficked rural roads crashes are mostly concentrated at “black
 255 spots” connected by safe sections of roads where – yet the safe sections are where speeds are
 256 checked; and d) that responsible drivers continually adjust speeds even within a speed zone so that
 257 any mean speed reading is highly likely to not represent the mean over that section.

258 There is little consistency in the various models. The slope of the Allsop trend line is opposite to the
 259 slopes of other models, and the Kloeden serious crash trend is very different to the other trends.

260 In summary based on my analysis of the reports above, and the concerns stated, there is no robust
 261 model that can be used to predict reductions in crash frequency with reductions in mean speed.
 262 Further data from various jurisdictions show that the highest speed roads have the lowest
 263 fatality/crash rates per 100 million km, so there is no underlying relationship between speed and
 264 crashes that would indicate a reduction in crashes with a reduction in mean speed. And finally given
 265 that speed alone never describes safe driving, it is not unexpected that any research aimed at
 266 relating speed alone to crashes is likely to be inconclusive.

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PAPER TITLE	New Low-Activity Nuclear Gauge for Soil Wet Density Measurement with Low Regulatory Burden		
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KEYWORDS:

Include up to 5 keywords

soil, density, moisture, nuclear, gamma-ray

ABSTRACT:

For decades, measuring soil density and moisture content using nuclear gauges has been a well-accepted testing method for road and embankment construction. The ownership and use of nuclear gauges come with some regulatory burdens such as the need for licensing, special storage, transportation requirements, gauge operator training, and personal dosimetry. As a result, the industry has been searching for alternatives to nuclear gauges. So far, no such non-nuclear solution has been found.

Recently, in the United States, the Nuclear Regulatory Commission declared a nuclear gauge exempt from licensing due to its safe design and use of an extremely low-activity radioactive source. The availability of such an instrument allows many countries to accept a proven testing method where often little or no testing was completed. The outcome will be a longer lasting road foundations and safer embankments. We will present the technical aspects of this nuclear gauge and its performance.

New Low-Activity Nuclear Gauge for Soil Wet Density Measurement with Low Regulatory Burden

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1 INTRODUCTION

Soil compaction is essential to increase the load bearing capacity during construction of roads, airfields, building sites, embankments, and parking lots. Soil compaction is verified by measuring the soil density which is defined as mass per unit volume. The resulting density measurement is a measure of the degree of compaction. During the entire process of constructing the road foundation, dry density and moisture content of the compacted soil and aggregate base are monitored by the direct measurement of wet density and moisture content of soil.

Soil wet density can be measured by the direct determination of mass and volume of a sample. When using this measurement methods, soil mass is removed in the measurement area creating a small hole. Although the mass of the soil can be measured accurately, the volume measurement is challenging due to the irregular geometry of the excavated hole. Several methods based on this concept are used in the industry. They are the sand cone method (ASTM D1556), the rubber balloon method (ASTM D2167), and the drive cylinder method (ASTM D2937). Although these methods can be accurate, from early on, the construction industry understood the drawbacks of using these methods such as long measurement time and operator errors.

Nuclear techniques for soil density and moisture content measurements were introduced in the 1950s (Krueger 1950; Pieper 1949; Belcher and Cuykendal, 1950). Historically, this soil density measurement method is based on gamma-ray scattering and this moisture content measurement method is based on neutron scattering. Gauges based on nuclear techniques, which are called nuclear density gauges (NDG), have been in use worldwide as a soil compaction measuring device for over five decades. These gauges are widely used in the industry because of the ease of use, accuracy of density and moisture measurements, large measurement volume, and short measurement time. The accuracy of measuring density and moisture by NDGs has been well-established and in many places the nuclear techniques are used as the standard method for quality control or quality assurance (ASTM D6938-10).

The NDGs use radioisotope sources of gamma-rays and neutrons. The source activity or strength are less than 370 MBq (10 mCi) for density gauges and less than 1850 MBq (50 mCi) for combined density and moisture gauges. Although such levels of source activity is not high and gauge design provides adequate shielding from radiation for the gauge operator, regulatory agencies worldwide consider them as controlled devices. These regulations impose certain burdens for ownership and operation of nuclear gauges such as the need for licensing, special storage, transportation requirements, gauge operator training, and personal dosimetry. On account of these burdens, many non-nuclear technologies, based on soil electrical properties, stiffness, and modulus, have been evaluated as alternatives to the nuclear techniques but with limited success (Berney and Kyzar 2012; Mejias-Santiago et al. 2013; Rose 2013; Wen et al. 2015).

In order to lessen the regulatory burden for gauge users without sacrificing the accuracy and precision of the measurement, a new nuclear density gauge has been developed to be used in the United States and its territories with the expectation of the acceptance of its use by other countries in the near future. The new gauge uses a low-activity gamma-ray source of strength less than 3.7 MBq (0.1 mCi) and measures the soil wet density by the gamma-ray scattering method. The new low-activity nuclear density gauge (Low-activity NDG) is exempt from nuclear regulations in the United States and its territories. Furthermore, with the use of a reliable non-nuclear soil moisture measurement method, the Low-activity NDG can obtain the dry density of soil and perform measurements comparable to a traditional NDG.

This paper describes design features and the measurement properties of this new Low-activity NDG and presents the results of several field studies conducted in North Carolina, USA.

2 LOW-ACTIVITY NUCLEAR DENSITY GAUGE FOR SOIL WET DENSITY MEASUREMENT

2.1 General Description

The Low-activity NDG for soil wet density measurement (Figure 1 and 2) is based on the physics of gamma-ray interactions with matter. The primary interaction mechanism of gamma-rays with energies in the range from 0.1 to 2 MeV with atomic electrons of chemical elements in typical soils is Compton scattering. The Compton scattering rates depend on the density of electrons in the material and therefore depend directly on the bulk density of the material.

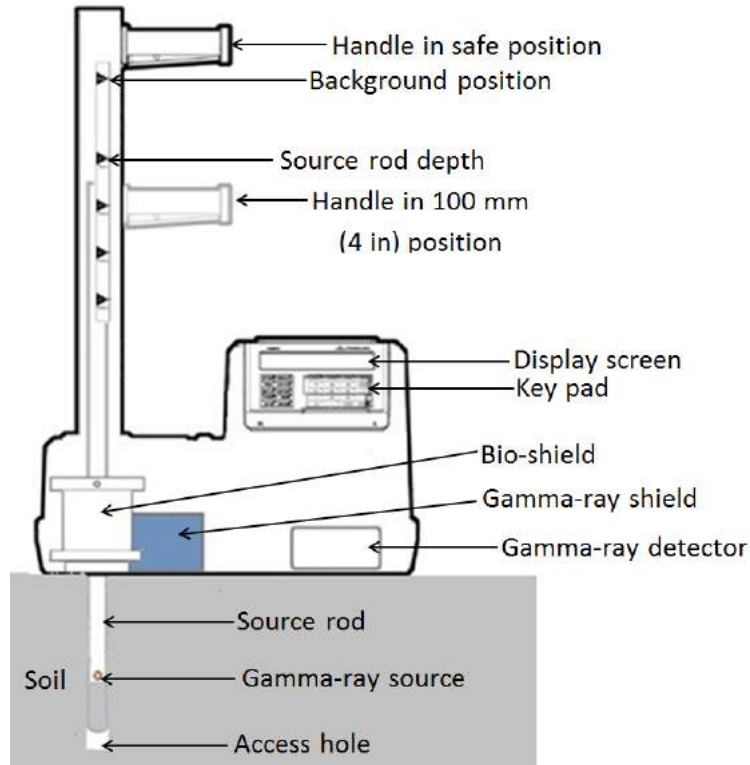


Figure 1. The Low-activity NDG for soil wet density measurement: Troxler Model 4590.

The gauge contains a radioisotope source of gamma-rays from a Cesium-137 source of primary energy of 0.662 MeV and an energy selective high efficiency gamma-ray detector. The gamma-rays traversing through the test sample that reach the detector having energies in the Compton range can be counted exclusively by the energy selective high efficiency detector. The scattered gamma-rays also scan a large volume of the sample enabling the estimation of densities of a heterogeneous material like soil. The instrumentation of this Low-activity NDG is similar to that of a laboratory nuclear density gauge for measuring bulk density of cylindrical compacted asphalt specimens, which is also exempt from regulations in the United States and its territories (Dep and Troxler 2000; Malpass and Khosla, 2001). Table 1 provides specifications and features of this Low-activity NDG and a conventional NDG for comparison.

The operation of the Low-activity NDG is similar to that of a conventional NDG except for one aspect, a short background count is required occasionally (Troxler and Dep 2009). The background count includes gamma-rays originated from the soil due to the natural radioactivity. For conventional NDGs, this background radiation is negligible when compared to the radiation from the gamma-ray source in the gauge. Since the Low-activity NDG uses nearly 1% of the source activity as that of a conventional NDG, the background radiation is measured and accounted for in the density determination. To measure the background radiation, a gamma-ray count is taken by placing the gamma-ray source in a special position where the source is still inside the gauge but is shielded by radiation shielding material in the tip of the source rod, bio-shield, and the extra shield between the bio-shield and the detector (Figure 1).

2.2 Measurement Process

The measurement process for the Low-activity NDG starts with the preparation of the test location by leveling the soil using the scraper plate, drilling a 16.5 mm (0.65 inch) diameter access hole about 50 mm (2 inches) deeper than the planned measurement depth, and then positioning the gauge directly on the soil. If this test is the first test for the day, the daily standard count (C_{std}) is taken first by taking two gamma-ray counts: one with the source at the safe position and the other with the source at the background position (Figure 1).



Figure 2. The Low-activity N for soil wet density measurement: Troxler Model 4590 (left) and the non-nuclear moisture probe: Troxler Model 6760 (right). The moisture probe communicates wirelessly with the gauge.

After accepting the standard count, the measurement is made by lowering the source rod to the desired depth i and taking the measurement count (C_{mi}) followed by moving the source rod up to the background position and taking the background count (C_b). The gamma-ray count-density relationship is given by

$$\frac{(C_{mi} - C_b)}{C_{std}} = A_i \exp^{B_i \rho} + C_i \quad (1)$$

where A_i , B_i , and C_i are the calibration constants which are determined during the factory calibration of the gauge. The gauge computes the wet density ρ using the measured counts and the pre-determined constants.

2.3 Safety and Security Features

The Low-activity NDG comes with innovative safety features incorporating a very low activity or strength Cesium-137 gamma-ray source. The radiation shielding for the source during storage and transport reduces the annual radiation exposure to the gauge operator to be less than that of the natural terrestrial radiation- 0.12 mSievert (12 mrem) vs. 0.28 mSievert (28 mrem). The self-shielding source rod design eliminates the need for a spring loaded shutter mechanism as is used in the conventional NDGs. This also eliminates the need for cleaning of the shutter area due to accumulation of soil. The source rod removal requires a special proprietary tool. The source rod tower and bio-shield are ruggedized against accidental damage to the gauge in the field (Troxler Electronic Laboratories, 2016).

3 GAUGE MEASUREMENT PROPERTIES

3.1 Density Calibration

The wet density calibration procedure for the Low-activity NDG is similar to that of a conventional NDG. Since the gamma-ray source decays with time, the gauge response or the gamma-ray count decreases with time. Therefore, to eliminate the time dependence, the gauge count is standardized by dividing with a count that only depends on time (C_{std}). Since the Low-activity NDG uses a low strength gamma-ray source, gamma-rays detected other than that from the source traversing through the test sample, C_b , are also excluded.

The Low-activity NDG is calibrated using the same three density calibration standards in specially designed bays that are used to calibrate the conventional NDGs. These standards are made out of solely magnesium,

combinations of magnesium and aluminum, and solely aluminum. They have different densities covering a density range from 1780 to 2690 kg/m³ (111 to 168 lb/ft³).

Table 1. Gauge properties for Low-activity NDG and a conventional nuclear density gauge

Property	Details	Low-Activity Nuclear Density Gauge (Troxler Model 4950)	Conventional Nuclear Density Gauge (Troxler Model 3440) ¹
Soil Property		Wet Density	Wet Density and Moisture
Technology	Measurement Method	Gamma-ray Compton Scattering	Gamma-ray Compton Scattering
	Gamma-ray Source	Cesium-137	Cesium-137
	Source Strength	3.33 MBq (0.09 mCi)	286 MBq (8 mCi)
	Detector	High Efficiency (solid phase)	Low detection efficiency (gas phase)
	Depth modes	Up to 200 mm (8 inch)	Up to 300 mm (12 inch)
	Analysis	Spectrometric	Counting
Accuracy & Precision of Wet Density Measurement		Meets Industry standard	Meets Industry standard
Security		Lower Activity Source Less risk	Higher Activity Source Greater risk
Safety	Radiation Dose	Lower doses 0.1 μSv/hr (0.01 mrem/hr) at 1 m	Higher doses 3 μSv/hr (0.3 mrem/hr) at 1 m
	Radiation safety Training requirement in US	Not Required	Required
Regulatory Burden	Burden	Very low	High
	Licensing in the US	Not Required	Required
	Shipping in the US: Type A packaging	Not Required	Required

¹Troxler Electronic Laboratories (2015).

Calibrating the Low-activity NDG requires gamma-ray counts taken by placing the gauge on the three calibration standards for each measurement depth. For each calibration standard i of density ρ_i and the gamma-ray source at depth j , let the gamma-ray count be C_{mij} . Let C_{bi} be the gamma-ray count for the background. The calibration constants for the depth j , A_j , B_j , and C_j are determined using the measured counts and the densities of standards. Note that, when calibrating the new gauge, the count time is selected to minimize the natural error from randomness of radioactive decay of the source.

The calibration curves for the measurement depths 50 mm (2 in), 100 mm (4 in), 150 mm (6 in), and 200 mm (8 in) for a Troxler Model 4590 Low-activity NDG are shown in Figure 3. The figure also contains, for the same depths, the calibration curves for a Troxler Model 3440 conventional. For comparison purposes, curves for Model 3440 gauge was scaled so that both types of gauges have the same count ratio at the lowest density. The instrument sensitivity to soil density, the change in gauge count to the change in density as given by the slope of the curve, is slightly better for the Low-activity NDG especially at higher densities. The sensitivity at 2480 kg/m³ (155 lb/ft³) for the 150 mm (6 in) for Low-activity NDG is -0.00136 per kg/m³ and for the conventional NDG is -0.00102 per kg/m³.

3.2 Density Measurement Precision

The measurement precision is a measure of repeatability of a density measurement. The precision varies with measurement depth and density, and depends on the uncertainty of the measurement count and uncertainty of the background count. We have estimated the density measurement precision for the four depth modes 50 mm (2 in), 100 mm (4 in), 150 mm (6 in), and 200 mm (8 in) at the three calibration standard densities for the Low-activity NDG. The precision estimates were made using a Monte Carlo method allowing random variations for measurement count and background count using actual data collected from a gauge calibration. For these estimates, we have considered the variability of a 2 minute measurement count and a 1 minute background count for each depth mode. Figure 4 provides the results. The measurement precision at 2000 kg/m³ (125 lb/ft³) for the Low-activity NDG is 3 kg/m³ (0.18 lb/ft³) and for a conventional NDG, for 1 minute count, is 3.4 kg/m³ (0.21 lb/ft³).

3.3 Density Measurement Precision: Repeatability and Reproducibility

For our preliminary study of determining the repeatability and reproducibility, we used four Troxler Model 4590 Low-activity NDGs. Three gauge operators conducted the test each operating a separate gauge. The fourth gauge was operated at different times by one of the same operators. We selected four test locations covering a 192 kg/m^3 (12 lb/ft^3) density range in a road construction project in Roxboro, NC. We selected the 150 mm (6 inch) depth mode of operation. For all measurements, a measurements count time of 2 minute and a background count time of 1 minute were used. On each test location, three measurements were obtained with each gauge. The repeatability of the measurement using the new nuclear gauge in the density range of 1955 to 2160 kg/m^3 (122 to 135 lb/ft^3) was $(0.3$ to $0.4 \text{ lb/ft}^3)$ and the reproducibility was $(0.5$ to $0.6 \text{ lb/ft}^3)$ (Table 2). For comparison purposes, Table 2 also shows the results of a more extensive test conducted for the conventional NDGs (ASTM D6938-10) where the measurement time of 1 minute, which is sufficient for the application, was used. The repeatability and reproducibility for the Low-activity NDG is similar to that for the conventional NDG.

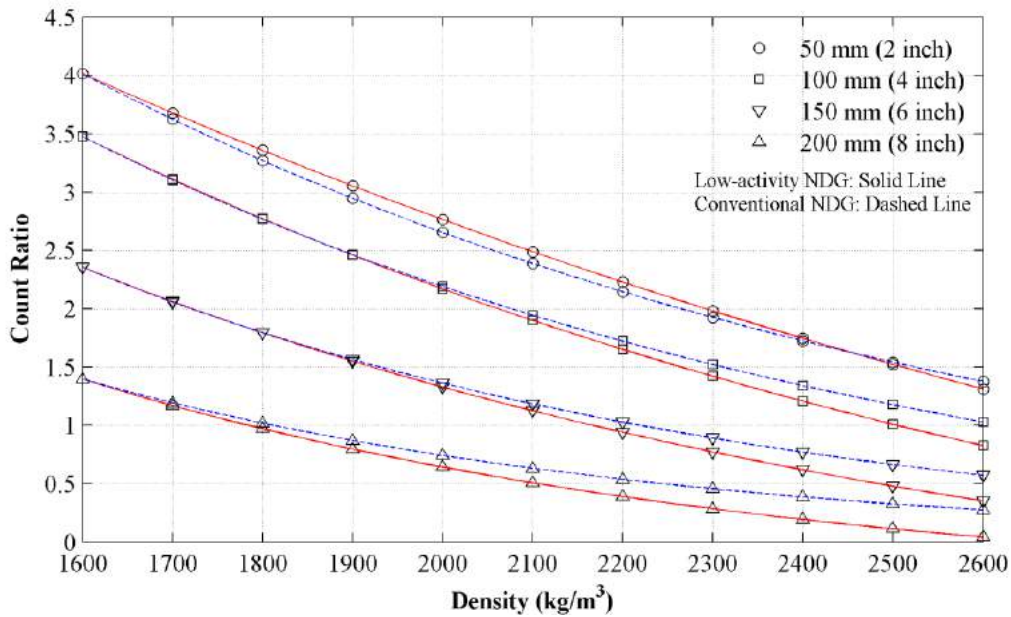


Figure 3. The wet density calibration curves for the Low-activity NDG (solid line) and a conventional nuclear density gauge Troxler Model 3440 (dashed line).

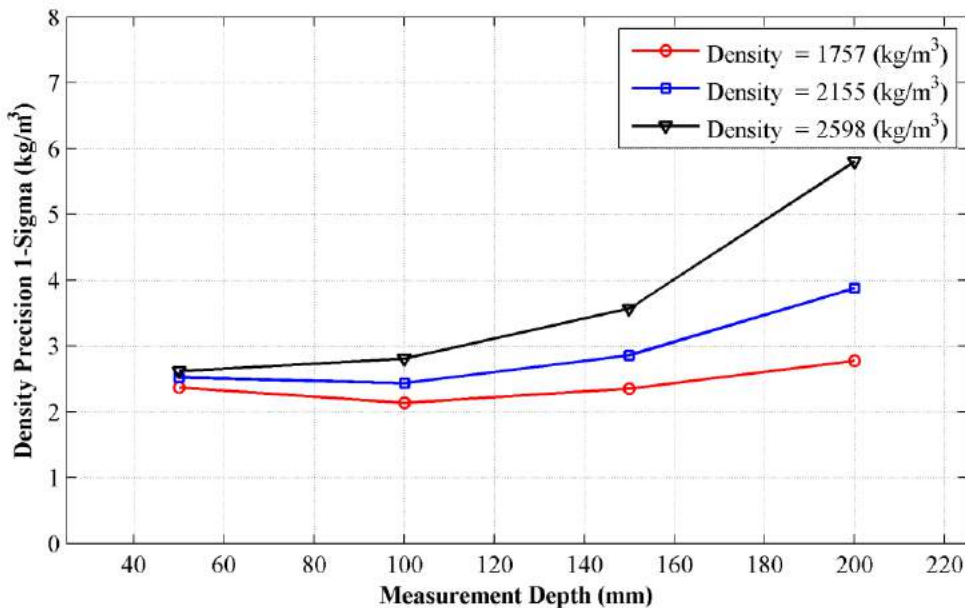


Figure 4. The Low-activity NDG density measurement precision for various measurement depths. Here, the precision is estimated for a measurement count time of 2 minutes and background count time of 1 minute.

For Low-activity NDG, the precision values estimated considering only the uncertainty of the gamma-ray counts from Poisson nature of radioactive decay as shown in Figure 4 was about 3 kg/m³ (0.2 lb/ft³) at 2162 kg/m³ (135 lb/ft³). This low value is expected since for this estimate gauge operator errors present in an actual measurement was not considered.

Table 2. The repeatability and reproducibility of wet density measurements¹ for Low-activity NDG and conventional nuclear density gauge

Test Location	Low-activity NDG (Troxler Model 4950)			Conventional Nuclear Density Gauge (Troxler Model 3440)
	1	2	3	Three locations (three soil types)
Average Density	1967 kg/m ³ (122.8 lb/ft ³)	2036 kg/m ³ (127.1 lb/ft ³)	2156 kg/m ³ (134.6 lb/ft ³)	1837-2084 kg/m ³ (114.7-130.1 lb/ft ³)
Repeatability at 1-σ	6.4 kg/m ³ (0.40 lb/ft ³)	7.0 kg/m ³ (0.44 lb/ft ³)	4.6 kg/m ³ (0.29 lb/ft ³)	4.2-7.4 kg/m ³ (0.26-0.46 lb/ft ³)
Reproducibility at 1-σ	8.7 kg/m ³ (0.54 lb/ft ³)	9.0 kg/m ³ (0.56 lb/ft ³)	9.3 kg/m ³ (0.58 lb/ft ³)	10.6-12.3 kg/m ³ (0.66-0.77 lb/ft ³)
Difference Between Two Test Results at 95% Limit for Repeatability	17.6 kg/m ³ (1.1 lb/ft ³)	19.2 kg/m ³ (1.2 lb/ft ³)	12.8 kg/m ³ (0.8 lb/ft ³)	11.2-20.8 kg/m ³ (0.7-1.3 lb/ft ³)
Difference Between Two Test Results at 95% Limit for Reproducibility	24.0 kg/m ³ (1.5 lb/ft ³)	25.6 kg/m ³ (1.6 lb/ft ³)	25.6 kg/m ³ (1.6 lb/ft ³)	30.4-33.6 kg/m ³ (1.9-2.1 lb/ft ³)

¹The depth mode for both types of gauges is 150 mm (6 inch). The measurement time for Low-activity NDG is 3 minutes and for the conventional nuclear density gauge is 1 minute.

3.4 Source aging effect on density measurement precision

The gamma-ray source in the new gauge, Cesium-137, has a 30 year half-life. Since the gauge uses a low activity or strength source, the change in source activity with time due to natural radioactive decay can degrade the density measurement precision. The degradation of density precision was estimated by using a Monte Carlo method allowing the decrease in the source activity with time and the random variations for measurement count and background count using data collected for gauge calibration. The degradation of density measurement precision for the first ten years is shown in Figure 5 (left). The Low-activity NDG can hold the required density measurement precision for about 8 to 10 years after that it requires a gamma-ray source replacement. In addition, the source aging effect can add a significant bias to the density reading Figure 5 (right). To avoid addition of a bias to the density measurement, the gauge requires a calibration every 1 to 2 years.

3.5 Bias on density measurement due to variability in the background

In a given project, if the natural variability of the composition of the soil, for example the sand to clay ratio, is minimal, the magnitude of the background count remains a constant. However, due to the randomness in the radioactive decay process, the background count follows a Poisson distribution and a one-time determined background count can have a magnitude two standard deviation from the average count. Therefore, when using the background count measured at the first test site for all subsequent counts, a bias can be introduced to the density measurement.

When count time for the background is long enough to reduce the standard deviation, the uncertainty in the background count can be lowered thus reducing the possibility of a bias to the density value. We have estimated the density bias by using a Monte Carlo method allowing for the random variations for measurement count and background count using actual data collected from a gauge calibration. For this study, we have estimated the bias for measuring the density of a sample having a density of 2155 kg/m³ (134.5 lb/ft³) with a 2 minute measurement and 1 minute background count. The density bias estimate for 150 mm (6 inch) and 200 mm (8 inch) measurement depths was found to be than 8 kg/m³ (0.5 lb/ft³). Therefore, if the background count is measured at one test location and used for all subsequent measurements in that project, the bias in the density reading were small.

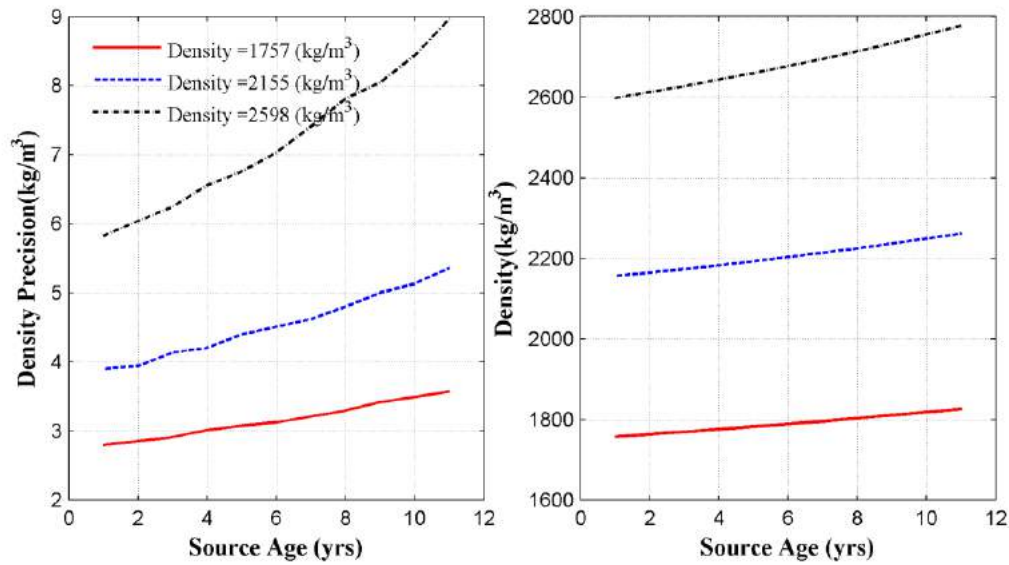


Figure 5. The gamma-ray source aging effect for the Low-activity NDG. Here, the source aging effect on precision and density are estimated for the 200 mm (8 inch) measurement depth and for a measurement count time of 2 minutes and background count time of 1 minute.

3.6 Field density measurement in comparison to a conventional gauge

Three Low-activity NDGs and one conventional NDG were used in a road construction project in Wayne County, North Carolina. Measurements were taken at four locations with each location wet density was determined for two depth modes 150 mm (6 in) and 200 mm (8 in). The results are shown in Figure 6. As expected, the three Low-activity NDGs showed good repeatability and reproducibility for wet density measurement with a range of 6 to 24 kg/m³ (0.4 to 1.5 lb/ft³) for the 150 mm (6 inch) measurement depth and 18 to 27 kg/m³ (1.1 to 1.4 lb/ft³) for the 200 mm (8 inch) measurement depth. The difference in the wet density values for the two types of nuclear gauges is less than 18 kg/m³ (1.1 lb/ft³) for both measurement depths.

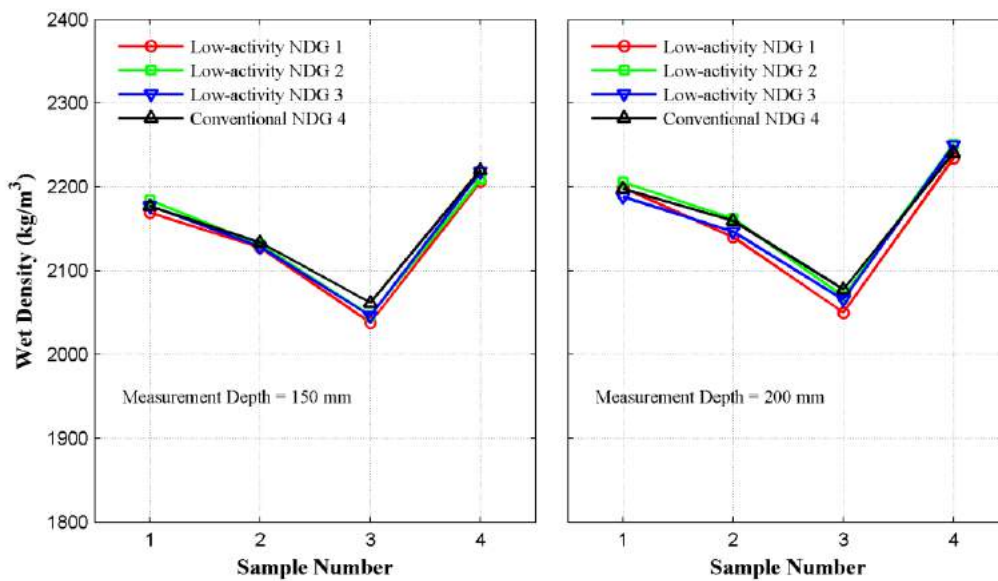


Figure 6. Low-activity NDG vs conventional nuclear density gauge density measurements for four test locations in a project. For the Low-activity NDG, the count time for measurement is 2 minutes and for the background is 1 minute.

The gauge was tested at eight different road construction projects in North Carolina covering a wide range of soil types including aggregate base courses. One to two new nuclear gauges and one conventional gauge were used in

density measurements done by multiple operators. Measurements were made at 6inch and 8inch depths. The new gauge measurement time was 2 minute with a one minute background count whereas the conventional gauge measurement time was 4 minutes. The results are shown in Figure 7 for 150 mm (6 inch) and Figure 8 for 200 mm (8 inch) measurements. For both depth modes, the new nuclear gauge showed a high correlation to conventional gauge measurements with a 95% reproducibility limit on the difference between two readings of about 38 kg/m^3 (2.4 b/ft^3). This proves to be close to the difference between two readings from two conventional gauges as stated for reproducibility in ASTM D6938-10.

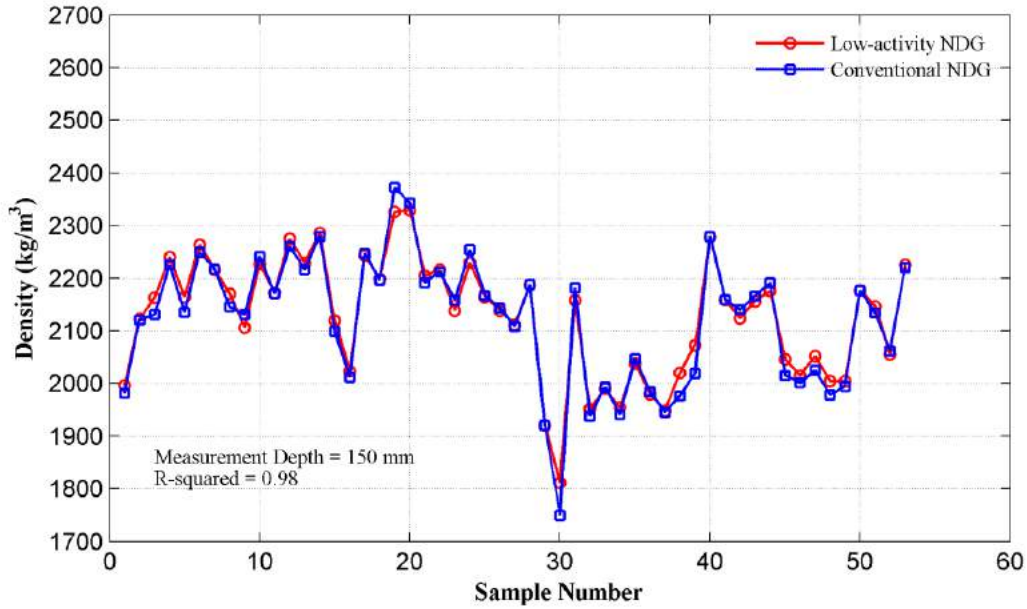


Figure 7. The Low-activity NDG and conventional nuclear density gauge density measurements for multiple locations in several projects. Here, the measurement depth is 150 mm (6 inches). For the Low-activity NDG, the measurement count time is 2 minutes and background count time is 1 minute.

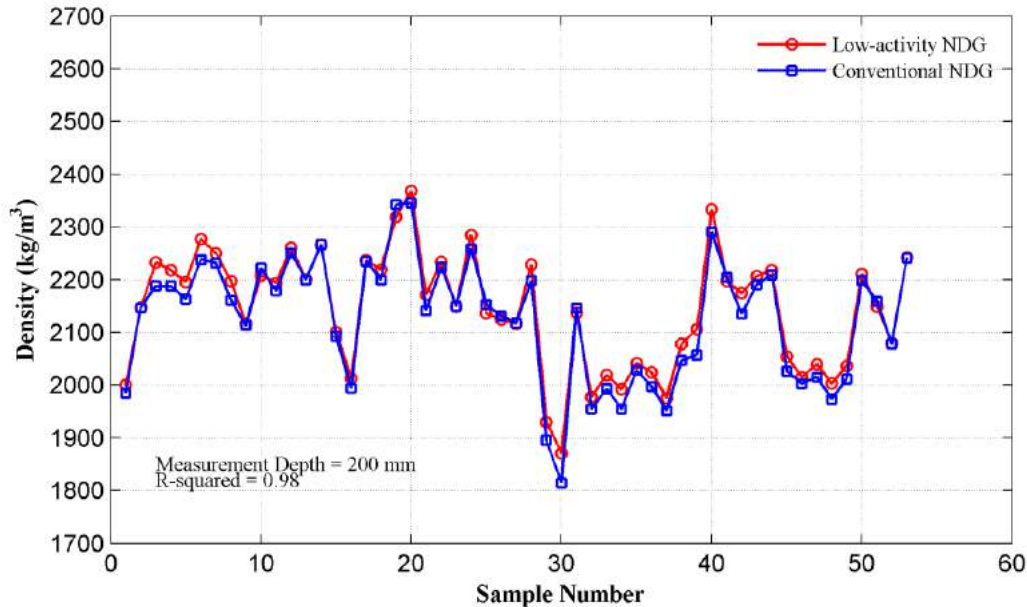


Figure 8. The Low-activity NDG and conventional nuclear density gauge density measurements for multiple locations in several projects. Here, the measurement depth is 200 mm (8 inches). For the Low-activity NDG, the measurement count time is 2 minutes and background count time is 1 minute.

4 SOIL DRY DENSITY MEASUREMENT

The Low-activity NDG only measures the soil wet density (WD). In order to determine the soil dry density (DD), it uses the moisture density (M) measured by a Troxler Model 6760 Moisture Probe. This probe uses electromagnetic technique for moisture measurement. The probe is inserted to the same access hole prepared for the Low-activity NDG when measuring the soil wet density. The dry density of soil is given by

$$DD = WD - M \quad (2)$$

Table 3 provides the results for the road construction project in Wayne County, North Carolina. Also shown in the table are the results from a conventional NDG. The results show a good agreement of the dry density determined by the two types of nuclear gauges with a maximum difference of 10.4 kg/m³ (0.65 lb/ft³).

Table 3. Soil dry density measurement by Low-activity NDG and a non-nuclear moisture probe. For comparison, also included are the measurements using a conventional nuclear density gauge. Here, WD, M, and DD are soil wet density, moisture density, and dry density respectively.

Sample	Conventional NDG			Low-activity NDG and Moisture Probe		
	WD kg/m ³ (lb/ft ³)	M kg/m ³ (lb/ft ³)	DD kg/m ³ (lb/ft ³)	WD kg/m ³ (lb/ft ³)	M kg/m ³ (lb/ft ³)	DD kg/m ³ (lb/ft ³)
1	2177 (135.9)	170 (10.6)	2007 (125.3)	2177 (135.9)	180 (11.2)	1997 (124.6)
2	2134 (133.2)	186 (11.6)	1948 (121.6)	2147 (134.0)	189 (11.8)	1958 (122.2)
3	2061 (128.7)	155 (9.7)	1906 (119.0)	2055 (128.3)	149 (9.3)	1907 (119.0)
4	2220 (138.6)	191 (11.9)	2029 (126.7)	2225 (138.9)	184 (11.5)	2041 (127.4)

If the gauge operator wishes to use another method for measuring the moisture content of soil, this moisture value can be entered into the gauge using the gauge key pad. Then, the gauge can compute the soil dry density using the wet density determined by the gauge and the moisture content.

5 INTERNATIONAL REGULATIONS FOR NUCLEAR GAUGES

The conventional NDGs are used in most countries around the world with the requirement of a permit or license and other restrictions such as training, monitoring badges, shipping paperwork, etc. However, there are a few countries which will not allow these nuclear gauges inside their borders at all due to the activity of the sources. It is likely that the new Low-activity NDG will be allowed in these areas due to the extremely reduced activity or strength of the Cesium-137 gamma-ray source, elimination of the Americium-241-Beryllium neutron source from the gauge, and the innovative safety features incorporated to the gauge.

Troxler and our international partners are targeting some countries, which currently have similar regulations to the United States, as potentially open to the same exempt designation for the new Low-activity NDG as the United States. This would allow gauge use in those areas with no permit or license requirements. This will take some time and assistance from our partners located in those areas as well as from our friends in the construction industry. As advocates for proper quality control / quality assurance in the road building industry, we can see this opportunity to make it easier to perform the required density measurements. When little or no density and moisture testing is performed on a prepared roadbed, the life of the road is diminished. Great improvements in the quality of roads constructed around the world would result.

6 CONCLUSIONS

- 1) The measurement sensitivity of the Low-activity NDG is slightly better than a conventional NDG especially in the high density region and is acceptable for the soil compaction application.
- 2) The measurement precision for the Low-activity NDG when taking measurement counts for 2 minutes and a background count for 1 minute meets the industry requirement. However, the total measurement time for this gauge is longer than that for a conventional NDG: 3 minutes compared to 1 minute.

- 3) The repeatability and reproducibility of density measurement for the Low-activity NDG when taking measurements with measurement counts for 2 minutes and a background count for 1 minute is comparable to that of a conventional NDG using a count time of 1 minute.
- 4) The Low-activity NDG can hold the required density measurement precision for about 8 to 10 years after that it requires a gamma-ray source replacement. To avoid addition of a significant bias to the density measurement, the gauge requires a calibration every 1 to 2 years.
- 5) For a given project, when the soil type is not significantly changing, the measurement time can be shortened by taking the background count only at the first measurement location. The bias for density is small and can be ignored in this case. Whenever the accuracy of the density measurement is most essential, a separate background should be taken for each measurement.
- 6) The density measurements taken with the Low-activity NDG for 17 different sites at 150 mm (6 inch) and 200 mm (8 inch) depth agreed well with that of a conventional NDG. This high degree of correlation between the density values determined by the two types of gauges (R-squared is 0.98) is due to the fact that both type of nuclear gauges are based on the same measurement method- gamma-ray scattering. The small bias between the density values is due to the fact that both gauge types were calibrated on the same set of calibration standards.
- 7) The wet density measured using the Low-activity NDG together with the moisture content measured by other reliable methods should show sufficient accuracy for determining dry density for QC/QA purposes.
- 8) In the USA and its territories where the new Low-activity NDG is exempt from regulation, the gauge user has no additional regulatory burden in purchasing, storing, transporting to the test site, shipping the gauge to the manufacturer for calibration, and using the gauge in multiple states.
- 9) The new Low-activity NDG will allow reliable quality control of road foundations to be possible in parts of the world where it is currently not. Fewer, less restrictive regulations will make it easier to obtain and use the nuclear density measurement methods around the world extending the life and improving the safety of roads everywhere.

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Probability of Crash Involvements based on Drivers' Characteristics and Traffic Rule Violation Records

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Abstract

This paper aims to investigate the factors associated with driving risk potential to be involved in traffic crashes. The relationships between the driver's historic records of traffic crashes, traffic rules violation and demographic characteristics have been explored. The individual driver's data were collected from four different databases during five years (2010-2014). A total of 713,783 individual driver's records were employed in the analysis process.

The Logistic regression modeling approach was applied to determine the most appropriate variables that can be used as predictors for crash involvement risk. Several variables are examined in the model including the number property damage only (PDO) crashes records, total number of traffic violations, speeding-related violations, unsafe driving behavior, number of violation with penalty points, number of penalty points and the demographic characteristics of the drivers (age, gender, years of experience and nationality). The results showed that violation records related to drivers' behavior during driving, driver demographic characteristics (age, gender, nationality), and property damage only related crashes records can be used as good predictors for identifying the risky drivers.

Key words: *Driver behavior, crash rates, road safety in Abu Dhabi, traffic rule violations, driver demographic characteristics*

INTRODUCTION

Factors that affect the occurrence of the traffic crash are fundamentally classified into three groups: human, vehicle and environmental. However, the human factors are considered as the main cause of the traffic crashes in the majority of the reported crashes (97%-98%) (Karscasu and Er 2011). Accordingly, examining the contributing factors regarding the drivers' behavior has received a significant attention in prior studies. Early on, Peck et al. (1971) stated that it is difficult to accurately predict drivers who will and will not be involved in crashes. However, one year after, Stewart and Campdell (1972) investigated the probability of future crashes in terms of historical crash and/or violation records. The study observed a four-year history of crash and traffic violation records of North Carolina drivers to predict the future crashes. After that in 1990s' and more recent, several studies had been published to suggest that the involvement in a traffic crash is not a random event and it can be predicted. This suggestion gives hope that interventions can be found to prevent traffic crashes occurrence and reduce the severity and number of casualties. These interventions might range from simple driver improvement letter, to changes in the existing strategies and polices of traffic safety to implementation of new strategies and polices. Accordingly, the term of "traffic accidents" in the field of road safety researches has been gradually changed to "traffic crashes".

In general, there is no much information in regard of driver behavior in the Gulf Corporation Council (GCC) countries. Hence, the interaction between drivers' behavior in terms of their traffic rule violation records and their involvements in traffic crashes has not been investigated yet or published in any prior studies. In Abu Dhabi (AD) Emirate, the capital of UAE, a tangible effort has recently been done to improve the road safety through the three main strategic approaches: Engineering, Education and Enforcement. Accordingly, road crash fatalities have reduced from 376 in year 2010 to 245 in year 2015 (about 35% reduction). Despite the increasing number of registered vehicle increased by 46% and licensed drivers by 45% during the same period.

Understanding the driver's behavior is the key factor to maintain a proper reduction rate of fatalities and improving the road safety strategies. Especially in the case of UAE where more than two hundred different nationalities are residing in the country along with different education levels, culture, language, and driving skills background which creates a considerable challenge for different agencies that are responsible for road safety especially the department of traffic police.

Therefore, the primary objective of this study is to investigate the contributing factors affecting driver risk potential to be involved in a traffic crash. Relationships between the at-fault drivers involved in traffic crashes and their historical records of traffic rules violations were investigated in terms of demographic characteristics of drivers.

LITRATURE REVIEW

Among the huge number of prior studies that examined the contributing factors that affect the occurrence of the traffic crashes, relatively few studies were tackled the interrelationship between the driver's behavior in terms of his/her historical records of traffic rule violations and his/her involvement in traffic crashes. In this topic, two different data were employed in prior studies: historical recorded data and self-report survey data of a sample of drivers.

Based on driver's data of Californian, Gebers (1999) developed 17 logistic regression models to test the most significant factors affecting drivers' involvement in crashes. After several tries the final model could correctly classify crash-involved drivers up to 27.6%. Gebers and Peck (2003) found that the driver's violation recorded is better than crash history record to predicate crash-prone drivers by using equation constructed modeling approach. Chandraratna et al. (2006) studied Kentucky drivers to develop a crash prediction model that can be used to estimate the likelihood of a driver being at fault for a near future crash occurrence by using logistic regression modelling technique. The significant association between the subsequent frequency of crash involvements of Canadian drivers and their crash and violation records in the previous period was also confirmed in three studies (Boyer et al., 1990; Hauer et al., 1991; Chen et al., 1995) by using multiple regression models. Blasco et al. (2003) conducted a follow-up study for a sample of drivers over 8 years to investigate the probability to be involved in a traffic crash after a crash occurrence. The study founded two different style of driving related to the accumulation versus the non-accumulation of crashes in short intervals.

Through examining drivers' age impact, previous violation records and crash data, Daigneault et al. (2002) concluded that prior crashes would be a better predictor for crash risk than prior violations. It was also found that this relationship increase with elder drivers (drivers have more than or equal 65 years old). Gerber and Peck (1992) showed a significant effect of age and prior violations on subsequent of crashes, especially for elder groups of drivers (aged 60-69 and above 70).

More recent studies focused on analyzing at-fault driver's data. Zhang et al. (2013) analyzed crash severity and violation data of Chinese drivers. The results established the role of traffic violations as one of the major risks threatening road safety. In addition, specific risk factors associated with traffic violations and accident severity were determined. The authors suggested that to reduce traffic crashes and fatality rates, measures such as traffic regulations targeting different vehicle types/driver groups with respect to the various human, vehicle and environment risk factors are needed. Some studies proved the significant effect of specific drivers' behavior such as speeding and drunk and drug usage on the prediction of the crash-prone driver (Watson et al. 2015; Das et al., 2015; Kim 2015). Nishida (2015) analyzed crash and violation data of 81 million licensed drivers in Japan (approximately 1 million crash record and 10 million violation records per year). It was found that drivers who experience accidents drive more carefully immediately after an accident, revealed high accident rates among drivers who have experienced certain violations, and produced other findings that could constitute a

foundation for developing individual driver-targeted measures. In addition, the drivers with a history of numerous accidents or apprehensions/violations are more likely to cause accidents.

Self-report survey technique was also intensively used by a number of researchers to identify the interaction between drivers' history of violations/behavior and his/her crash involvements (for example, Winter and Dodou 2010 (in USA); Constantinou et al., 2011 (in Greek); Illiescu and Sarbescu 2013 (in Romania); Pearson et al. 2013 (in USA); Sumer, 2003 (in Turkey); Stanojević 2013 (in Serbia and Kosovo), Xu et al., 2014 (in China), Ona et al., 2014 (in Italy and Spain)). These studies proved the same tendency of drivers who frequently take traffic rule violation tickets is more likely to be involved in traffic crashes. In addition, Berdoulat et al., (2013), investigated the aggressiveness and impulsiveness in the prediction of risky drivers. It was found that aggressiveness and impeded progress were the best predictors of violations and aggressive violations. The results supported that transgressive driving behaviors are relevant indicators of aggressive driving. The same finding was found by Bachoo et al. (2013) from a sample of post graduate students in Durban, South Africa. A new definition of the impulsivity in driving context was suggested by Bıçaksız and Ozkan (2015) during analyzing 288 student self-reported questionnaires in driving behavior, violations and accident involvements. Machado-Leon et al. (2016) investigated crash risk perceptions in an inter-city, two-way road context of 492 drivers by using a Stated Preference ranking survey. The study that all risky driving behaviors showed a significant potential effect on crash risk perceptions, and model's results allowed to differentiate more important from less important unsafe driving behaviors based on their weight on perceived crash risk. Furthermore, the study analyzed the potential differences in risk perception of these traffic violations between drivers of different characteristics, such as driving experience, household size, income and gender.

DATA COLLECTION AND PREPARATION

The employed data in this paper were extracted from four different databases of Abu-Dhabi (AD) traffic police during five years (from 2010 to 2014). These databases are; 1) traffic violation system, 2) property damage only (PDO) crashes system, 3) severe crashes system and 4) driving license information system. Then an integration process among the different systems was carried out to define the historical date of individual drivers in a new integrated dataset system to be used in this study. As shown in Figure 1, the integration process was carried out by utilizing a unique traffic code that is given to each licensed driver when he/she issues a driving license in AD. This code is called Unified Traffic Code (UTC) in the traffic police database systems.

Around 1.1 million registered driving licenses exist in the driving licenses database by the end of 2014. Data filtration was conducted to exclude the drivers have missing information and drivers issued driving licenses during the period of the study period. As a result, about 713,583 drivers' records were settled in the integrated database system. The integrated database for five years (from 2010 to 2014) includes a total number of traffic rules violation of 2,762,275 and total

PDO crashes of 682,136 and total severe crashes (i.e., any crash with at least one injury or fatality) of 8,604. All of these records were employed in the analysis and modeling process in this study.

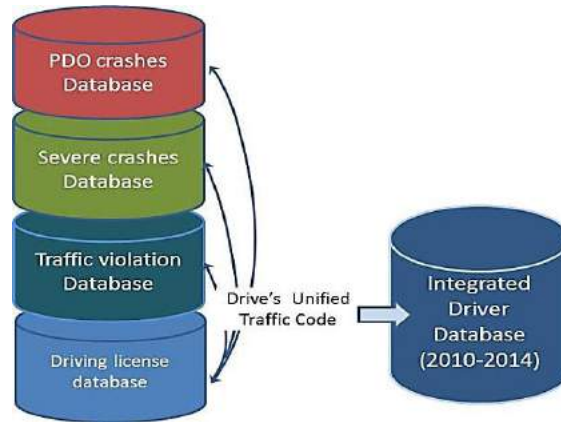


Figure 1. Integrated driver's database system in AD (crashes, violations, and demographic information)

Table 1: Examined variables with respect to traffic crashed records of the driver

Variable class	Variable name	Variable description
I	Number of Violation Records of the Driver	the total number of violation tickets over five years of the driver
	Number of Face to Face Traffic Rule Violations	the violations that he driver are taken on street by the police officers
	Number of Absent Violations	the violations that were recorded by automated enforcement devices such as: speed camera, radars and red light violation camera
	Number of Violations That Have Traffic Points	about 73 violation have a traffic points range between 2 to 24 points based on AD traffic violation law
	Number of Traffic Points of the Driver	the sum of the traffic point of the driver over the five years of the study
	Number of Hazard Violations	about 25 type of violations defined by AD traffic police including speeding when the speed exceeds 60 km/hr over the speed limit, red light violation, tailgating, mobile use, alcohol use, etc.
	Number of Class (A) Violations	The speed-related violations
	Number of Class (B) Violations	about 32 type of violations that related to drivers' behavior during driving and that may affect road safety such as: tailgating, red light violation, traffic rule and signs violations, overtaking violations, mobile use, alcohol or drug use, set bet violation, reckless driving, sudden lane changing, illegal U-turn priorities violations, etc.
II	Number of Class (C) Violations	the other types of violations out of class (A) and (B) listed in the traffic violation law such as: illegal parking, drive with expired license, made changes in the vehicles, illegal horn use, vehicle plat number missing or unclear, increasing in vehicle exhaust, over loaded, etc.
	Driver's Demographic Characteristics	Variables of driver's characteristics such as: age, gender, nationality and years of experience in driving.

Variables Description

Based on the developed integration database system, more detailed information regarding drivers' records were extracted and used as independent variables to investigate the chance of driver involvement in future traffic crashes. All variables are driver-based information during five years from 2010 till the end of year 2014. The examined variables can be classified into two groups as shown in Table 1. Class I variables include the violation variables categories and class II variables include the demographic characteristics of drivers. Table 1 shows the name and description of each examined variable in this study.

DATA ANALYSIS

Crash Rates of Different Driver's Groups

All drivers were classified into groups based on the examined variables shown in Table 1. For example; class (A) violation group includes all drivers who got at least one violation in that group. Then, a crash rate of the drivers has been calculated (crash rate = total number of crashes / total number of the driver) at each aforementioned group. Figure 2 shows the crash rate of drivers who belong to Class I variables. It shows that the drivers who experienced any type of violations have the highest crash rate during the five years of the study data period followed by drivers who got face to face (present) violation types. This figure reveals that drivers who get presents violation ticket are more dangerous than who got absent violations. In addition, drivers who got violations of type (B) are more dangerous than who got violations of type (A). From this result, the hazard violations list should be revised to take violations from type (B) in order to cover all violations that have high risk relationship. So more violation types should be added to this list and more enforcement in such violation types should be taken by face to face tickets (present violations).

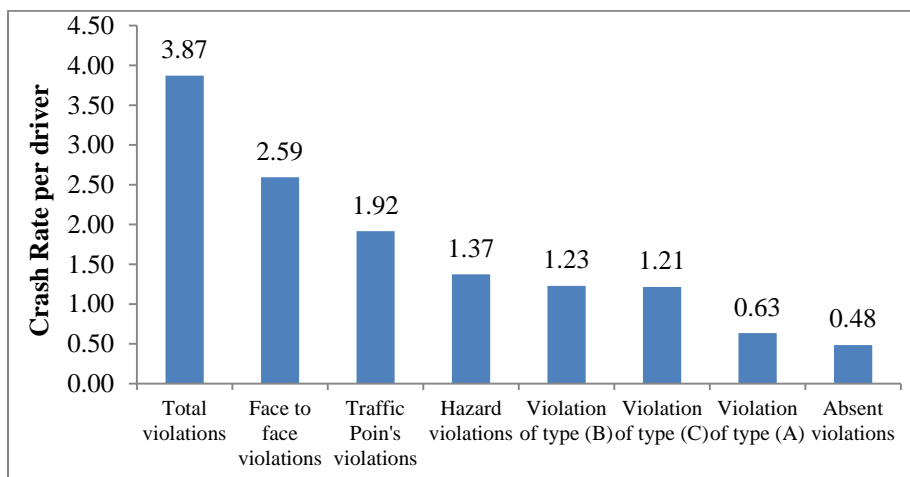


Figure 2: Crash rate of the driver's group belonging to class (I) variables

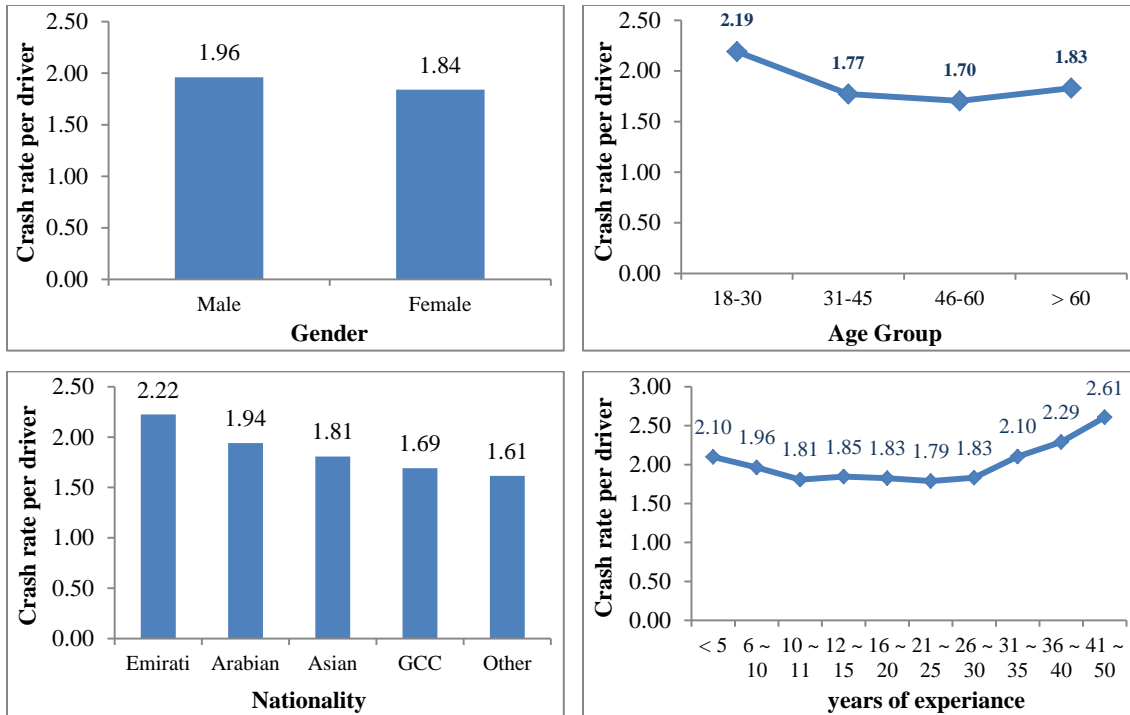


Figure 3: Crash rate of the driver’s group belonging to class (II) variables

Figure 3 shows the crash rate of the drivers who belong to Class II variables (driver’s demographic characteristics). This figure shows that the male drivers have slightly higher crash rate than females and UAE local drivers (Emirati) have higher crash rates than other nationalities. In addition, younger drivers have the highest crash rate and the crash rate slightly increases for elder drivers compared to mid-age drivers. Furthermore, the crash rate decreases with increasing the experience years of driving up to 30 years. After 30 years of experience the crash rate increases that can be explained due to the elder age of the drivers.

Driver’s Percentage Involved in Crashes

Percentage of driver’s involvement in crashes was calculated for each driver’s group. The driver percentage values can be calculated as follows:

$$\% \text{ of Drivers involved in crashes} = \frac{\text{number of drivers involved in crashes}}{\text{total number of drivers}} \dots\dots\dots (1)$$

Figure 4 shows the percentage of drivers involved in crashes with respect to the number of violations the driver received during the five years of the study. This figure shows a strong correlation between the percentage of drivers and their violation records. This result reveals that the likelihood to be involved in a crash increases with increasing the number of traffic role violations of the driver. Also, it is clear that the percentage of drivers to be involved in a future crash is higher for the drivers who were involved in at least one historical violation. On the other hand, Figure 5 shows the percentage of driver involved in severe crash only (i.e., crash with at least one injury) with respect to drivers’ violation records. This figure shows that the percentage of drivers involved in severe crashes increases with increasing violation record of the drivers,

however; the percentage value is very low compared to involved in any crash that shown in Figure 4.

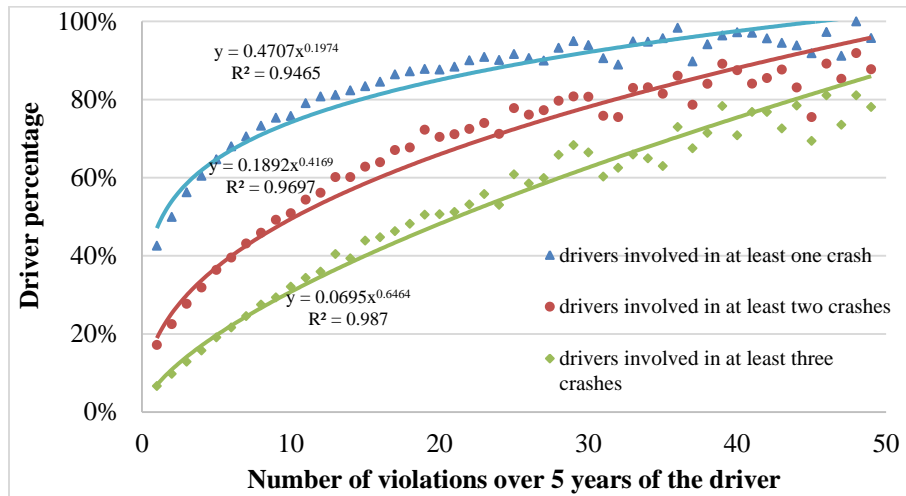


Figure 4: Percentage of drivers involved in crashes (PDO or severe)

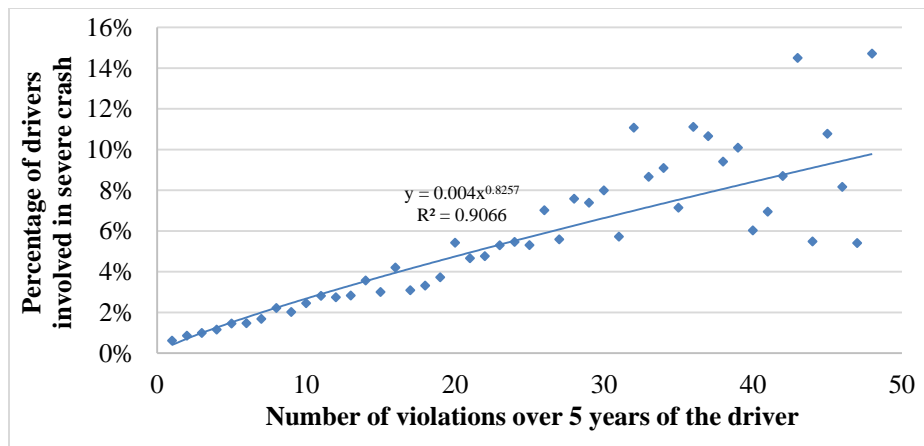


Figure 5: Percentage of drivers involved in a severe crashes only

Driver’s Involved in Crashes Model Development

In this section, the authors have attempted to study the previous variables that affect the crash involvement and form them in a defined model to predict the likelihood of drivers to be involved in at-fault-crash. The response/ dependent variable is a binary nature (1= involved in at least one PDO/ severe crash, 0= not involved in a crash). Binary logistic regression has good potential to examine the independent variables (predictors) of the dataset that were shown in Table 1. The logistic regression model was utilized to predict the occurrence of an at-fault crash for the i^{th} case that is can be calculated from the following equations:

$$P_i = \frac{e^{u_i}}{1+e^{u_i}} \dots \dots \dots (2)$$

$$U_i = a_o + b_1Y_{1i} + b_2Y_{2i} + \dots + b_nY_{ni} \dots\dots\dots (3)$$

Where: P_i is the probability of involved in at-fault crashes by crash-prone drivers

U_i is the linear combination of predictor variables

b_i is the coefficient estimated using the maximum likelihood method

Y_i is the explanatory variables as listed in Table 1.

For the logistic model, Eq. (2) can be rewritten as follows:

$$\ln\left(\frac{P_i}{1-P_i}\right) = a_o + b_1Y_{1i} + b_2Y_{2i} + \dots + b_nY_{ni} \dots\dots\dots (4)$$

The linear regression is simply the natural logarithm of the probability of having one crash divided by the probability of having no crash. The followed procedure for estimating the model coefficients is maximum likelihood where the best combination of the dependent variables is to be found that maximizes the likelihood of obtaining the observed outcomes.

The first step is to find the significant variables that maximize the likelihood of the estimation model outputs. In this regards, several trails were conducted to estimate the model parameters by utilizing all variables that are listed in Table 1. The results showed that most of the variables that refer to the violation records of the drivers are significantly affect the crash involvement. However, the statistical modelling process excluded many variables due to its internal strong correlations.

Table 2 shows Pearson correlation matrix between the violation-related variables. This table indicates that all the variables refer to violation records are significantly correlated to crash involvement of the drivers and corrections among each other's. The highest five variables correlated to the crash involvement of drivers are Violations Type B, Total violations, Traffic point violations, Present violations, Total points. On the other hand, there are strong correlations among these five variables which affected the goodness of estimated model in the initial trails.

By looking at the correction value “R²” in Table 1, it can be clearly concluded that the violation type B is the best representative variable among the violation’s variable group due to its strong correlation to the dependent variable “crash involvement” and other violation related independent variables. Thus, this variable has been selected to represent the other types of the examined violation records of the driver in the developing process of the regression model.

Table 3 shows the estimate parameters of the developed regression model. It is shows that, among the tested explanatory variables, five variables are significantly affect the likelihood of drivers to be involved in crashes at 95% significant level. These variables can be taken as the best predictors to identify the risky drivers or drivers that have high potential to be involved in traffic crashes in the future. Three variables related to drivers demographic characteristics (i.e. gender, age group, and nationality) are shown as significant variables in the estimated model. In addition, the results show that the drivers that have historical records in PDO crashes have a higher probability to be involved in future crashes by 5.6 times than other drivers.

Table 2: Pearson correlation matrix of dependent variables related to violation records

Violation-related Variables		Crash Involvement	Total Violations	Traffic Point Violations	Total Points	Hazard Violations	Violation Type(A)	Violation Type(B)	Violation Type(C)	Present Violations	Absent Violations
Crash Involvement	<i>Pearson Correlation</i>	1	.412**	.408**	.372**	.326**	.211**	.542**	.305**	.386**	.208**
	<i>P-value</i>		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total Violations	<i>Pearson Correlation</i>	.412**	1	.815**	.753**	.704**	.512**	.663**	.790**	.903**	.465**
	<i>P-value</i>	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Traffic Point Violations	<i>Pearson Correlation</i>	.408**	.815**	1	.903**	.822**	.397**	.811**	.613**	.814**	.445**
	<i>P-value</i>	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total Points	<i>Pearson Correlation</i>	.372**	.753**	.903**	1	.761**	.518**	.667**	.534**	.703**	.558**
	<i>P-value</i>	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00
Hazard Violations	<i>Pearson Correlation</i>	.326**	.704**	.822**	.761**	1	.254**	.863**	.431**	.706**	.310**
	<i>P-value</i>	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00
Violation Type(A)	<i>Pearson Correlation</i>	.211**	.512**	.397**	.518**	.254**	1	.183**	.190**	.382**	.742**
	<i>P-value</i>	0.00	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00
Violation Type(B)	<i>Pearson Correlation</i>	.542**	.663**	.811**	.667**	.863**	.183**	1	.372**	.728**	.241**
	<i>P-value</i>	0.00	0.00	0.00	0.00	0.00	0.00		0.00	0.00	0.00
Violation Type(C)	<i>Pearson Correlation</i>	.305**	.790**	.613**	.534**	.431**	.190**	.372**	1	.845**	.224**
	<i>P-value</i>	0.00	0.00	0.00	0.00	0.00	0.00	0.00		0.00	0.00
Present Violations	<i>Pearson Correlation</i>	.386**	.903**	.814**	.703**	.706**	.382**	.728**	.845**	1	.227**
	<i>P-value</i>	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		0.00
Absent Violations	<i>Pearson Correlation</i>	.208**	.465**	.445**	.558**	.310**	.742**	.241**	.224**	.227**	1
	<i>P-value</i>	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

** Correlation is significant at the 0.01 level (2-tailed).

Table 3. Results of logistic regression model

Dependent variable	Regression Coefficient	Standard error	Wald χ^2	P-value	Odds ratio	Odds ratio 95% confidence limits	
						Lower	Upper
Intercept	0.023	1.465	.000	0.098	1.02		
Gender	3.776	0.593	40.565	0.000	1.80	13.7	139.5
Age Group	-1.865	0.354	27.799	0.000	0.15	0.1	0.3
Nationality	1.939	0.976	3.947	0.047	6.95	1.0	47.1
Violation Type B	1.090	0.436	6.236	0.013	2.97	1.3	7.0
PDO Crash binary	10.935	1.022	114.411	0.000	5.60	0.8	41.6

The estimated coefficients of the regression model variables are seen to be logic indicating since all variables have positive impact of crash involvement probability except the driver's age. That

means the local residents and mal drivers have potential be involved in crashes more than other. It shows that the male drivers are 1.8 times more to be involved in future crashes than females. This particular result can be explained since about 85% of licensed drivers in AD are males. Regarding the age, the negative since of the coefficient mean your drivers are more likelihood to be involved in crashes and the probability decreases with increase the driver's age; however the analysis shown in Figure 3 showed that the this deceasing trend starts to be increases for elder drivers (age > 50 years old).

CONCLUSION

The prime goal of this study is to explore the factor associated to drivers who have high potential to be involved in traffic crashes (i.e., property damage only PDO and severe crash types). Several variables related to the historical records of drivers' violation types and the demographic characteristics of the drivers are examined. Five years (2010-2015) of historical data of individual drivers are employed. The data analysis showed that driver's crash rates and the percentage of drivers involved in crashes increases are different among the examined violation types. Young drivers (aged between 18 and 30 years), male and low experiences have higher rate of crashes compared to others. In addition, older drivers (greater than 60 years old) have higher crash rate than younger drivers.

Logistic regression model was developed to estimate the associated variables of the drivers' crash involvements. After several trials the results shows that five variables have significantly affect the crash involvement of drivers: Violations Type B (about 32 types of violations refer to un safe driving behavior), PDO crashes history, driver's age, nationality and gender. These variables could be taken as the best predictors to identify the risky drivers in the future.

An immediate implication is: if the traffic rule violation records could be reduced or controlled successfully, then the rate of crashes and resulted in injuries and fatalities would be reduced accordingly. Further research aiming to investigate specific violation types in terms of crash rates is currently conducted by the author for deeply understanding the interaction between drivers' behavior in terms of their violation records and road safety indicators.

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Road Tunnel Inspection with High-Performance Devices

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1 ABSTRACT

The management and maintenance of the tunnels currently in service require the application of techniques for the inspection of linings and facilities, using high performance systems. Besides, many of the contracts of maintenance of the infrastructures, including those of tunnels, are managed through contracts of service quality indicators. In order to assess the different indicators of the state of the tunnels, it is necessary to have objective inspection tools (not only visual inspection performed by surveyors) to measure the evolution of deterioration.

The paper presents different automated techniques, some of them already fully implemented in the infrastructure maintenance market, such as the Lidar, and some of them which are brand new and make it possible to complete the information for the inspection of deterioration in all kinds of lining. We will present how the results obtained in different tunnels through to visual inspections performed by specialist surveyors can be completed with the information obtained of them through a combination of techniques mainly based on imaging and laser.

The automated inspection offers a better knowledge of the state of the tunnel and avoids subjectivity problems associated to traditional inspection. The paper will show also how these techniques may function as a basis for accepting new tunnels.

2 NEED TO INSPECT ROAD TUNNELS

It is currently necessary to inspect road tunnels in order to maintain an inventory focused on preventive maintenance, so the real condition of the infrastructure is known and it is possible to take the necessary actions in case critical issues are detected. In addition to this, the tunnel inventory must be precise enough so as to assess which investments are necessary to ensure the operation safety and to prevent structural, geotechnical or infrastructure/operation issues or failure.

Scheduled routine inspections are key to detect safety issues and to prevent failure. Thus, it is necessary to devote time and resources to acquire the best possible information that allows us to plan the necessary repairs to prevent structure failure and, what is worse, loss of life.

It should be considered that the external signs of deterioration that appear on the lining surface –such as humidity, corrosion, concrete cracking (face loss or spalling)– are indicative of possible structure problems. Hence the importance of conducting such inspections.

3 VISUAL INSPECTION OF THE STRUCTURE BY MEANS OF INSPECTORS

More detailed inspections conducted with non-destructive methods present the drawback of being slow, entailing complex tests and demanding many material resources, so they are usually replaced by visual inspections conducted by staff checking the tunnel. However, this method does not ensure having the most comprehensive and swift information.

In order to solve both issues, new systems have been under development for these last years so as to conduct quick and, most importantly, accurate inspections regarding the tunnel condition that facilitate using non-destructive techniques in more specific areas.

The data analysis provided by these systems can be automatically processed at a later stage, where visualization, analysis and management tools can be included.

We will now see which techniques are achieving greater progress.

4 HIGH-PERFORMANCE AUTOMATED INSPECTION TECHNIQUES

The most commonly used techniques for continuous automatic visual inspection up to now are using linear CCD cameras with an adequate lightning, and using the LIDAR. 3D cameras that were used for other purposes are also being used in tunnels. These 3D cameras use a linear laser spotlight and make it possible to increase the inspection speed while offering maximum resolution. The deformation of the projected light of the spotlight is monitored with a camera to check for defects on the surface.

As it will be shown in the paper these 3D cameras –used in combination with LIDAR technology subsystems and visible infrared light subsystems– are the basis to conduct an accurate tunnel inspection. This information that can be later used and focused or directed in analysis tools -later inspections with indirect non-destructive methods such as Radar, Thermography, Impact Echo Test, etc.

The combination of techniques offers the best possible results in terms of visual inspection; it is possible to apply data analysis to show where to use slower, more expensive techniques requiring more resources, such as geophysical methods. This combination of techniques will depend on the needs and the type of the tunnels to inspect, and the available inspection time, among other constraints. This paper presents a solution mainly based on the use of 3D cameras to detect deterioration on the supports, together with other sensors such as LIDAR or inertial units to know the tunnel geometry (slope, profile, radius, sections, clearance...), and the visible infrared light subsystem and the panoramic cameras to obtain global images of the tunnel course.

The 3D cameras will be used, as mentioned, to inspect the support deterioration but also to survey the pavement to check for its texture and the eventual existence of cracks, potholes or other deformation.

5 SYSTEM DESCRIPTION

The **Tunnelings system**, used to conduct high-performance, automatic visual inspections, is based on 3D cameras that allow for the high-resolution, high performance inspection and analysis of the defects in tunnel supports. These are 3D cameras, each of them consisting of an area-scan camera and laser line spotlight, which are monitored by a control unit.

The Tunnelings system also includes the following subsystems:

- A LIDAR system able to obtain a point cloud that represents in 3D the surroundings of the area surveyed;
- An inertial motion unit attached to each 3D camera, which provides 3D orientation and position of the vehicle and makes it possible to obtain a 3D virtual model of the infrastructure;
- An inertial motion unit to control the position of the vehicle and to correct it if needed;
- A GPS that provides the global coordinate references at the beginning and the end of the tunnel;
- A visible 360° camera that takes panoramic images every few meters in order to visually locate the area surveyed;
- A set of illuminators and visible infrared cameras able to acquire high-resolution images in dark environments;
- An odometer that provides the system with sufficient accuracy to trigger the 3D camera every 1mm, i.e. to obtain profiles with a spacing of 1mm.

All the sensors are precisely integrated and synchronized. The odometry system (DMI) is used to synchronize the information of the data perception sensors (3D cameras, LIDAR and panoramic cameras) with the sensors

used for positioning and localization purposes (GPS and the inertial motion units). Thus, all the data of the components of the system can be unequivocally referenced between them.

The **3D laser cameras** used in Tunnelings apply the triangulation principle to extract 3D information from the sensors, as shown in Figure 1. A pattern of known light, a line in this case, is projected from the laser onto the object to be inspected. The line is recorded by an area-scan camera positioned away at a fixed distance, at an oblique angle relative to the projected light. The relative position (rotation + translation) between camera and laser is obtained by means of a precise calibration process.

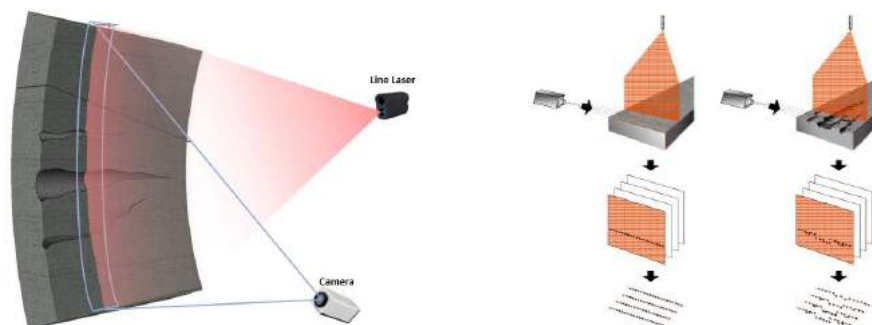


Figure 1 - Triangulation measurement principle for high-speed acquisition of 3D information

The 3D sensor cameras used in Tunnelings (Figure 2) are the LCMS camera-laser units originally developed for road surface inspection tasks by INO (National Optics Institute, Canada), and marketed by Pavemetrics. These sensors have been adapted and integrated for tunnel inspection within the Tunnelings system.

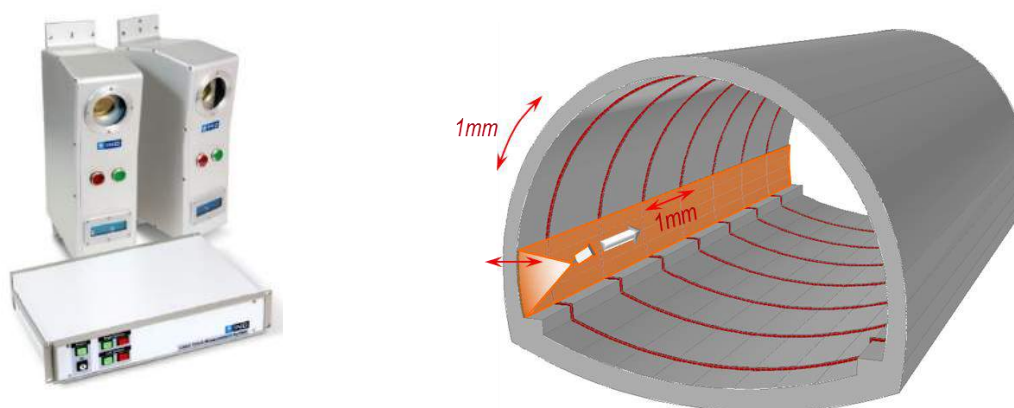


Figure 2 - 3D cameras: a) Camera-laser units and control unit; b) Surveying scheme

The 3D laser cameras are triggered by a Distance Measurement Instrument (DMI) to acquire profiles of points, and then images are created by means of profiles aggregation. Their most relevant features are the following:

- Speed: 5600 profiles per second
- Number of points per profile: 2352
- Maximum resolution:
 - Longitudinal: 1 profile every 1 mm
 - Transversal: 1 point every 1 mm
 - Radial (depth): 0.5 mm

Each camera of the *Tunnelings* system captures an array of 2352 points per profile in a width of approx. 2 m with three-dimensional information. The profiles are acquired every time the system receives a signal from the Distance Measurement Instrument (DMI) of the vehicle. The DMI measures the displacement in the

driving direction of the survey with millimetre resolution and, for example, every 1000 consecutive profiles are merged, so that images with a 2352 x 1000 mm resolution are obtained and saved in a common file for each camera.

The image shown in Figure 3.a corresponds to the intensity image of the lining surface, whereas the registered depth map of the lining surface (also referred to as range data) is shown in Figure 3.b. The brighter pixels in the depth image indicate more depth, i.e. these points are farther from the sensor. Finally, the 3D reconstruction can be seen in Figure 3.c, where the lining surface and its defects can be examined on-site in close detail.

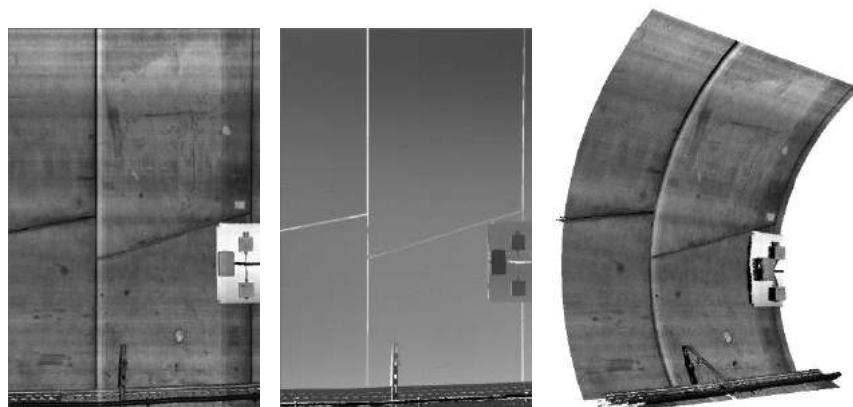


Figure 3 – 3D sensors data: a) Intensity image b) Depth Image c) Point Cloud

Tunnelings subsystems

The *Tunnelings* system has two LIDAR installed, which are electro-optical laser measurement sensors that work under the principle of pulse propagation time measurement: the LIDAR scans its perimeter in a plane using laser beams, and measures its surroundings in two-dimensional polar coordinates. The laser beams are reflected using a mirror, and they scan the surroundings in a circular manner. The measurements are triggered using an angular encoder. When a laser beam strikes an object, its position is determined. The distance to the object is calculated from the propagation time that the light requires from the emission to the reception of the reflection at the sensor, and the direction is obtained from the angular encoder information. Figure 4 displays all the subsystems that are installed on a truck in order to conduct the inspection.



Figure 4 – Subsystems (Lidar-Infrared Visible cameras) assembled together with 3D cameras on a truck

Figure 5 shows the results these subsystems are able to offer. An example of the data retrieved by this LIDAR sensor type: a profile every 10 cm when the platform is driving at a speed of 18km/h (5m/s), and the panoramic images in well-lit tunnels.



Figure 5 – Road tunnel reconstruction from LIDAR data and panoramic cameras

In order to obtain a panoramic image of the tunnel, different sensors are used –depending on the lightning conditions. If the tunnel is well lit, 360° cameras are used to obtain a full picture that can be rotated. In case the tunnel is not sufficiently lit, the visible infrared cameras are used. The visible infrared system includes a set of illuminators and infrared cameras (Figure 6) able to acquire high-resolution images in completely dark environments.



Figure 6 – Visible infrared cameras and illuminators

The cameras have a CMOS sensor of e2v which features a high quality and sensitivity at the near-infrared. Besides, these cameras also include a global shutter that allows for accurate moving snapshots and reproduction of sharp contrasts.

6 INSTALLATION FOR ROAD TUNNELS

Using the most common setting (trade-off between transverse accuracy and transverse field of view), each camera-laser unit described in the previous section is able to inspect a 2 m wide section with an accuracy of 1 mm. Therefore, in road tunnels that usually have large sections it is necessary to assemble several sensors to inspect the full section of the tunnel. In large tunnels that include a number of traffic lanes, this generally involves several runs. Most road tunnels require surveying first one half of the tunnel and later the second half. A scheme of six sensor assembly for 4 lane road tunnel inspection is depicted in Figure 6.

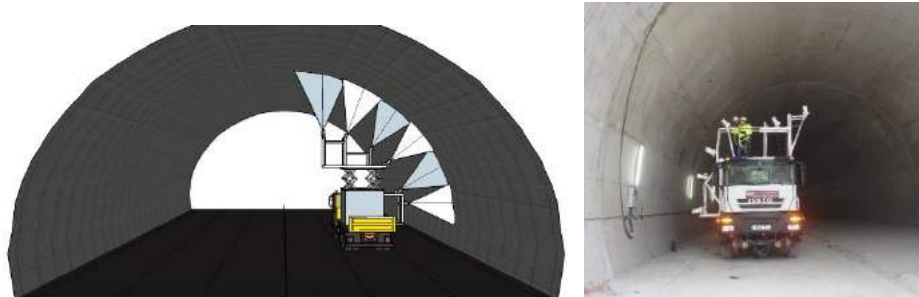


Figure 6 - Scheme of six sensor assembly for a road tunnel inspection and installation.

The laser cameras should be installed at certain distances from the tunnel surface and their laser beams must be projected perpendicularly towards the tunnel surface. In addition to this, the camera uses a laser beam which can be opened up to 50 degrees. In order to avoid cross-talk between two consecutive cameras, they must be installed in a staggered pattern. To design the most efficient assembly for these cameras, it is necessary to keep these requisites in mind and to have a sound knowledge of the tunnel sections. As each tunnel has its particularities, the assembly of the cameras must be customized for every particular survey. Thus, a software has been developed to place the cameras in such way that the inspections can be performed in the shortest time possible.

The *Tunnelings* devices can be installed in different kinds of vehicles, as shown below:



Figure 7 – The *Tunnelings* system installed on truck with six sensors and on a van with four sensors

In roads or road networks that include many tunnels, it is usual to continuously measure several tunnels, keeping the tracing whenever it is possible. Maneuverability and speed during the inspection prevent unnecessary disruptions to traffic.

7 ANALYSIS AND PRESENTATION OF RESULTS

Once the data are registered, the results are analysed. International standards regarding road tunnels are focused on the inspection of facilities. From the structural point of view of analysis, tunnels are considered as any other type of infrastructure. Thus, the deterioration analysis will depend on the type of support; then, the severity of the different types of deterioration will be classified based on previous experiences in similar tunnels and in similar geotechnical conditions.

In order to analyze the results, it is very useful to view the whole tunnel in a single composition, and to use automatic or assisted tools to detect defects according to the images and to the 3D reconstruction.

To this end, a newly developed software enables the user to arrange the images of the lining in a wider view. It makes it possible to select the analysed defects and to overlap them. The user can move forward along the tunnel, or reach a given milestone or kilometre point. Thanks to a listing, it is possible to locate and display the defects and the area where they are located, as shown in Figure 8. The software offers tools to measure cracks and 3D profiles of the lining; besides, other incidents can also be noted.

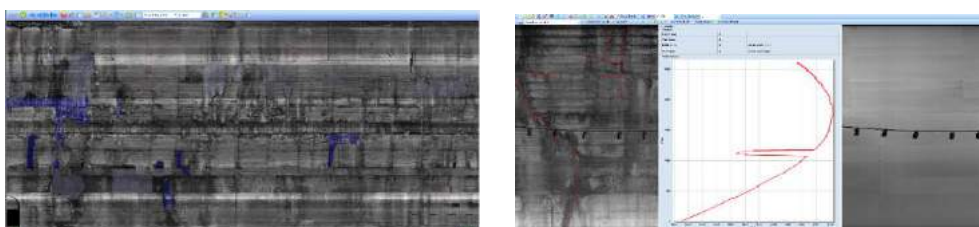


Figure 8 – General Tunnel’s image view and detail view

In order to improve the user experience, the 3D interactive display is available (Figure 10): the software can combine the images and reconstruct the tunnel in a 3D point cloud, which allows the users to move along the tunnels to perform an on-site inspection of the full lining from their own computer.

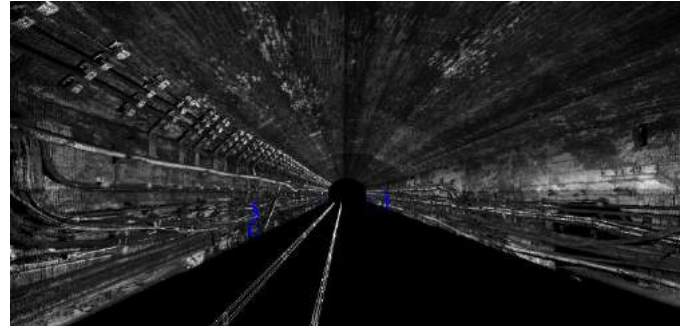


Figure 10 - Tunnel point cloud reconstruction

It is also possible to generate cartography of the full tunnel, obtaining a quick view of the lining of the whole tunnel and other areas with defects (see Figure 11).

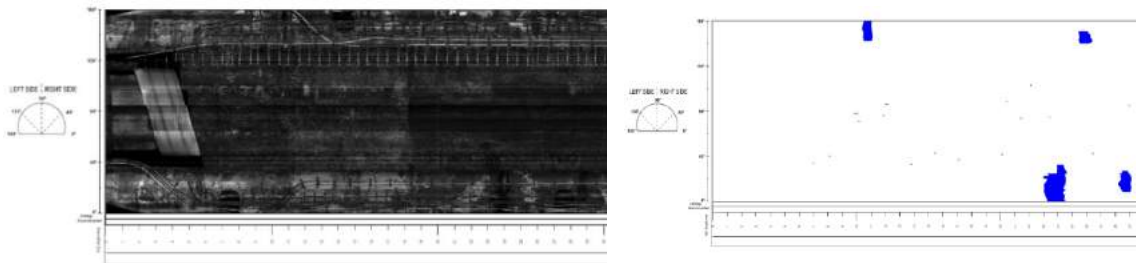


Figure 11 - Cartography: a) Intensity images b) Defects mapping

8 INFORMATION ANALYSIS OF THE LINING SURFACE USING 3D CAMERAS

Even when no regulations are in place, the following defects are considered the most important either in road or railway tunnels: (a) leaks, spalling, lining face loss and cracking on the tunnel surface, and (b) concrete delamination, steel and reinforcement corrosion, voids and debonding in the interior lining –eventually detected during visual inspections.

Although it is not possible to conduct inspections using NDT tests to detect delamination, corrosion or debonding in the lining, performing an automated detailed inspection of the surface allows for the detection of areas where later radar-type tests or impact-echo tests would be most appropriate in order to detect possible debonding or to measure the speed of corrosion of the reinforcement.

The information of the sensors provided by *Tunnelings* makes it possible to develop image analysis algorithms that automatically detect defects on the surface of the lining. The detection algorithms must first consider all the elements located in the tunnel (electrical installations, fire-fighting systems, ventilation...) in order to distinguish them from the supports and to process them separately at inspection level. Once extracted the information of the supports, the software must process the tunnels according to their type.

Considering the type of lining used in the tunnel support, the shape of the tunnel section –due to their construction method–, and the type of defects they might present, *Tunnelings* detects and assesses breaks in the supports (*chipping*), lining face losses cracking indicative of opening and extension, and damp areas. In addition, the clearance and any eventual significant deformations are assessed and compared to the theoretical sections of the project.

Cracking Evaluation

The cracks are detected through *Computer vision*. The area of interest is processed as a 3D point cloud (Figure 12). The 3D point cloud is divided in perpendicular scan-lines along the cracking. Each scan-line is separately evaluated to obtain the width and depth of the crack at that point, and these values are combined to obtain a value representative of the whole crack. Then the cracks are classified according to their extent and severity.

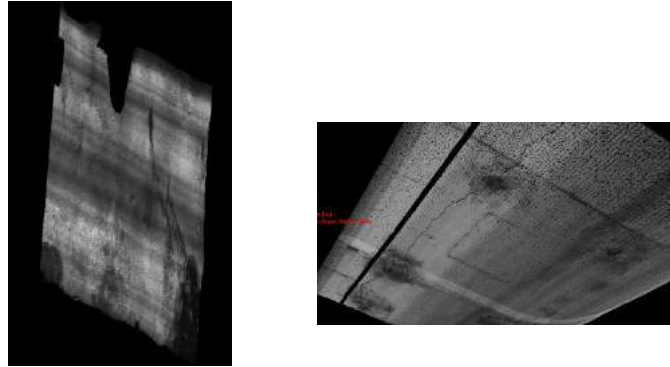


Figure 12: 3D Reconstruction of cracks on a shotcrete lining and a in-situ concrete support

Face loss evaluation

The range image provided by the 3D camera is used to assess the face loss in the different materials that we usually find in the lining of road tunnels:

1. Tunnels with concrete lining or mass concrete lining with transverse joints,
2. Tunnels with shotcrete lining,
3. Tunnels constructed with the *cut and cover* technique using slabs and diaphragm walls,
4. Concrete segment slabs and others.

The main step to detect any face loss or deformation that could pose any potential risk for the lining is to separate all facilities and elements that appear on the image and, using a segmentation method as shown in Figure 13, selecting all areas that present raveling, falling or deformation, and then assessing their extent and severity.

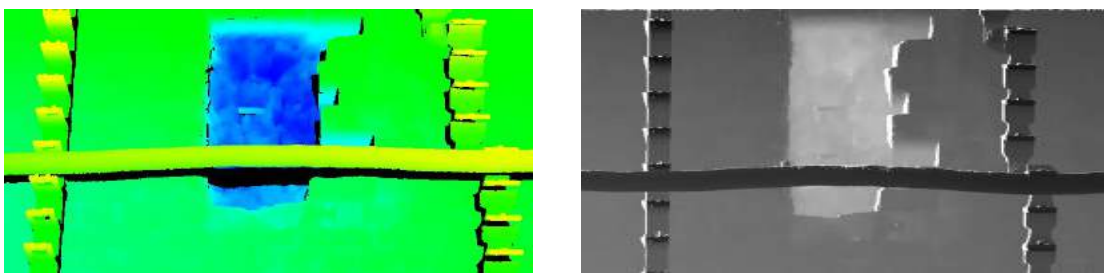


Figure 13 - Lining Face Loss example: a) Point cloud; b) Depth image

Joints evaluation

In the case of segment tunnels, precast tunnels or in situ concrete tunnels, the system allows for a very precise definition that makes it possible to know the location of its elements and to evaluate the opening and stepping of the joints between them (Figure 14). Thus, it would be possible to check in subsequent inspections that no movements have occurred and to provide an initial reference for the new construction tunnels.

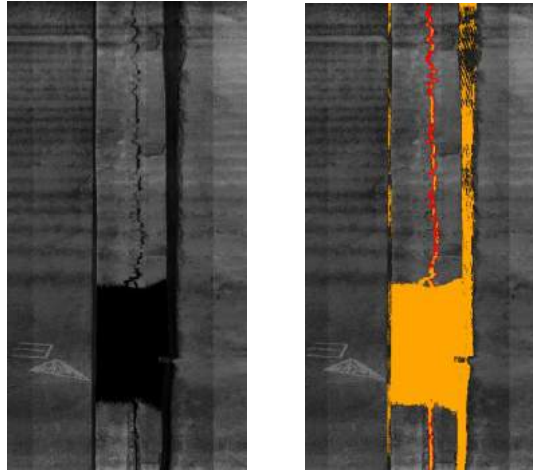


Figure 14 – Condition analysis between module joints in in-situ concrete tunnel

In road tunnels, it is frequent that the structural support of the tunnel is covered by waterproof sheets or decorative elements that improve the lightning and the cleaning works. In these cases, the visual inspection is limited as it is not possible to directly access the support.

In case the support remains hidden, only eventual deformation or breaks of these decorative/waterproof covering will allow us to know that there is deterioration. *Tunnelings* offers very detailed information on the bending and the relative deformations in millimetre sections, so it offers information about the eventual deformation of coverings (for instance, due to material falling from the keystone) and the condition of the waterproof sheets.

9 INFORMATION MANAGEMENT (PERFORMANCE INDICATORS) WITHIN A INFRASTRUCTURE MANAGEMENT SYSTEM

Condition assessment

With an inspection system such as the one described in this document, we have an objective analysis that will not depend on the judgment of the person in charge of the assessment, as is the case of visual inspections. This analysis will offer the same results in successive inspections of an infrastructure in the same state. Besides, having a quantitative analysis of the infrastructure condition will allow us to obtain statistics for every type of assessed defect and to define status indicators that will facilitate the assessment of the current state of the asset and its evolution over time.

The statistics are very useful to obtain a quick overview of the general state of the asset, to identify areas in worse condition and to determine which areas require a closer visual inspection by specialist technicians. The statistical revision of all the data throughout the tunnels enables the user to extract, for example, those areas where the presence of cracking, chipping, repaired areas and dampness is more relevant.

Some representative graphs of the tunnel lining condition are depicted in Figure 18. Moreover, the software makes it possible to acquire a greater level of detail with respect to the analysis, and to carry out multiple combinations and queries about pathologies and surveyed sections. The results are presented by adding the length or area, depending on the incident threshold in each meter of the tunnel,

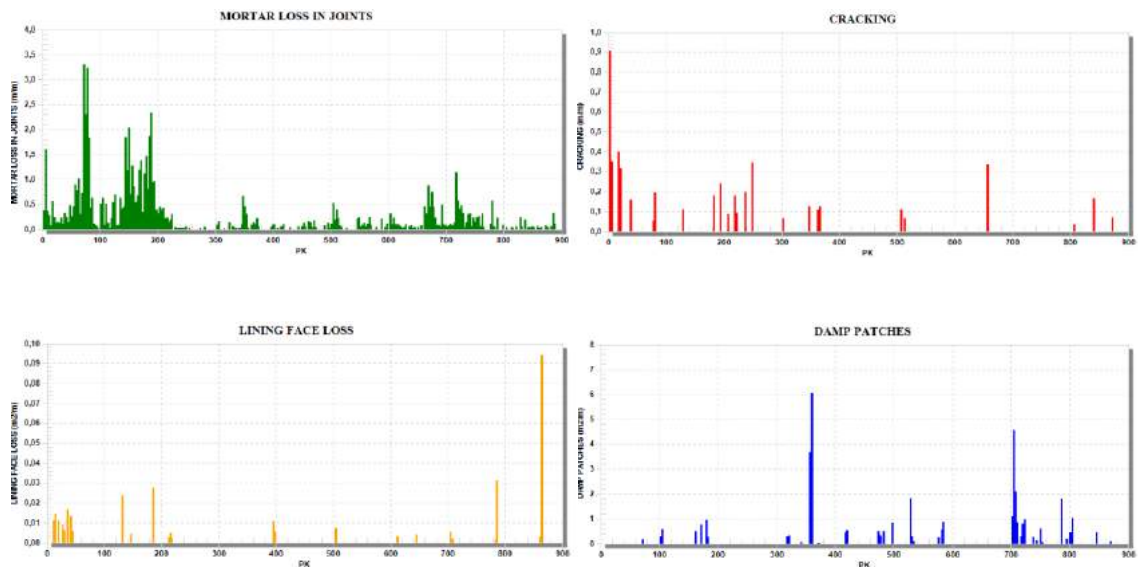


Figure 18 - Statistics

As for the indicators, these will allow us to get a general overview of the condition of the assets, to compare the state of the different assets within our network, and to prioritize the maintenance, either according to the existing risks, or to the profitability of the interventions, as specific interventions will allow for a significant improvement of the overall rate of the asset.

The implementation of tunnel condition indicators makes it possible to integrate the results of the automatic inspections with management systems that use detailed visual inspection performed by specialist technicians. With this method, visual inspection can be focused on areas identified as highly problematic, while the network manager is able to keep working with the assessment systems used until now.

For each of the assessed assets, a value or “item score” is obtained. This value, obtained by combining the extent of the defect and its mean severity, will indicate the state of the asset respect the element considered.

Extent means the proportion of the assessed area presenting this defect. In order to calculate the severity, it is necessary to define thresholds respect the measured parameters, resulting in a series of categories for each of the assessed defects. For example, the mean width can be used for the assessment of the cracking, as representative characteristic of the defect severity. In other cases, such as face loss, the most representative parameter will be the mean depth and the area affected.

Using the extent and the severity of the different parameters we can obtain an item score for each of the assessed elements. In many cases, knowing or even predicting the evolution of the state of the tunnel is as important as finding out the current condition of the asset. This can be seen in the statistics obtained in different areas, by comparing the cumulative values obtained in different inspections or, more simply, by comparing the overall rate.

In addition to this, 2D and 3D comparisons may be performed with *Tunnelings* as seen in Figure 19, that displays a comparison between the extent of humidities in different periods of the year.

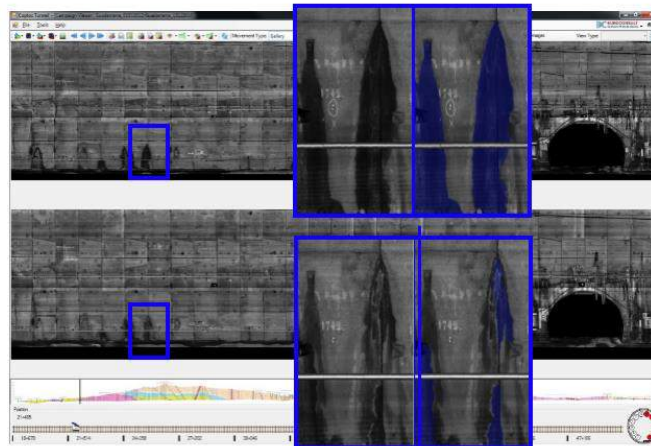


Figure 19 - Comparison of two inspections in different periods of the year to see the evolutions of humidity

With all this information obtained thanks to the automated inspection, and after the analysis of defects and indicators (score), this could be the basis to schedule the instrumentation of those deteriorations which have shown some progress or that could pose a risk for the integrity of the tunnel. In some cases, the said instrumentation will need to allow for the monitoring of deformations, cracks or other structural parameters in order to keep continuous track or real-time track, as shown in the following image of the instrumentation of a joint monitored by means of a fibre optic sensor in order to check the deformations both statically and dynamically.

Integration of the data obtained by all the sensors into BIM

The use of a BIM system (Building Information Modeling system) allows us to have a virtual 3D model of the infrastructure (in this case, of the inspected tunnels) with spatial location of the different elements, including geographical information (e.g. absolute coordinates). It also makes it possible to determine the properties of the different components. Therefore, these systems offer a unique environment where it is possible to share all the infrastructure data and to optimize their management, incorporating infrastructure inspection data and indicators and facilitating the analysis of repair and reconstruction costs.

The BIM systems make the collaboration between tunnel designers and constructors easier. These systems are currently revolutionizing the management of the life-cycle of the infrastructures and optimizing the maintenance works. Despite the BIM have a strong presence in tunnel conferences and published works, the incorporation of 3D information obtained through dynamic inspection systems is not widely spread for the management of huge infrastructures.

Tunnelings offers the possibility to integrate the inspection in an existing BIM system, as well as to create a new virtual model of the infrastructure.

The data provided by the 3D laser cameras, LIDAR and visible or infrared cameras are processed to obtain a 3D reconstruction with integration of inertial data in a fully automated manner, without any user supervision, so it very convenient for the generation of a virtual reconstruction and its integration into a BIM platform. Once the data are stored into a BIM platform, the information can be retrieved by means of different software tools.

Conclusions

The management and maintenance of the tunnels currently in service require the application of techniques of inspection of linings and facilities, using high performance systems. This requires having available systems capable of reducing the risk of traffic interruptions that may occur, reducing the disturbances to road users and the potential incidents in the tunnel structure.

Tunnelings is a system to inspect the condition of the lining of the tunnels (cracks, deteriorations and dampness) that uses high resolution images from the surface while it is illuminated with laser emitters.

The system uses high speed and high resolution digital cameras together with high power lasers aligned in one transversal plane. This configuration offers a large variety of benefits and, together with other conventional techniques of image analysis, its operation is much faster than regular and traditional scanners. This provides the optimal configuration to create a 3D image of the characteristics of the crack and deterioration of the inspected area. The inspection vehicle can also include Lidar-type sensors so as to facilitate the 3D reconstruction of the section and the plan for their incorporation to the BIMs.

In some cases, the inspection of the tunnel supports can be performed together with the pavement inspection (evenness, cracking, macrotexture...). Finally, these systems of lining inspection can also be used for the follow-up and inventory of different elements: firefighting, electricity, anchoring of ventilation systems...

With this method it is possible to obtain results no subjective in relation with the lining surface affected by defects and the severity of those defects. It is possible also to obtain a combined quality indicator in relation about the general structural behavior of the tunnel. Finally, with this value (the combined performance indicator) we can manage the needs to maintain the tunnel in good or propose repairs or permanent monitoring and instrumentation in order to keep track of the evolution of the detected defects.

The software allows the user to combine additional information, such as geotechnical cuttings, parameters of tunnel's progressing during its construction, etc. and also allows the use to overlap the image with the analysis of observed impacts (joint openings, splits and deformations, etc.).

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Pedestrian Safety and Road Crossing Behavior: A Case Study of Abu Dhabi

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Abstract

Reducing the fatalities resulted in traffic crashes, especially pedestrian-related crashes is a strategic target of all road safety authorities around the world. This paper aims to investigate the contributing factors affecting the severity of pedestrian-vehicle collisions in Emirate of Abu-Dhabi (AD). About 20% of severe crashes in AD are pedestrian-related crashes that led to about 24.5% of total fatalities and 12% of total crashes' injuries. A logistic regression model was developed to examine the variables that may affect the severity of pedestrian crashes. About 3,770 pedestrian from year 2008 to 2015 are employed in the analysis. The majority of these crashes were occurred when the pedestrians trying to cross road (about 92.5%).

The results indicated twelve variables associated with the fatality of pedestrian crashes. these variables include young pedestrian (less than 18 years old), older pedestrians (more than 60 years old), pedestrians crossing at non designated crosswalk locations, speeding behavior of the drivers, speed limit of the road, number of lanes, road type, crash locations, pedestrians location, commercial area, road surface condition and darkness. The estimated odds ratios showed that the probability to be involved in a fatal pedestrian crashes increases with increasing speed limit of the road, number of lanes, lack of designated crosswalk, crossing at mid-block section, crossing rural roads and dark locations. In addition, young pedestrians and older pedestrians are more likely to be involved in fatal-crashes than injury-crashes. Finally, preventative countermeasures for increasing road safety of pedestrians are provided.

INTRODUCTION

The pedestrians are always considered as the vulnerable road users due to their lack of protection. Many prior studies proved that the human error is the major cause of traffic crashes (Ren, 2013). In Abu Dhabi Emirate (AD), the capital of the United Arab Emirates, pedestrian crashes and related casualties represent a significant contribution in total number of severe crashes (i.e., any crash with at least one injury or fatality). Table 1 shows the percentage of pedestrian crashes and resulted casualties from the total severe crashes occurred from year 2008 to 2015. The table shows that the pedestrian-related crashes represent 20.2% of total severe crashes, about 24.5% of total fatalities and 17.1% of serious injuries. However, the slight injury represents 7.4% only of all pedestrian casualties. These results reflect the high severity of pedestrian crashes.

Table 1: Percentage of pedestrian-related crashes (2008-2015)

Item	Number of crashes	Number of casualties			
		Fatalities	Serious injury	Medium injury	slight injury
All severe crashes	18581	2553	2962	13219	12812
Pedestrian Crashes only	3762	625	507	2002	949
Percentage	20.2%	24.5%	17.1%	15.1%	7.4%

Table 2 shows the distribution of the pedestrian-related crashes that have been reported by AD traffic police between 2008 and 2015. The table shows that 15.4% of the pedestrian casualties are fatalities. The majority of the pedestrian crashes occurred in urban roads and about 80.2% occurred during the pedestrian crossing at undesignated crosswalk locations. In addition, 42.9% of pedestrian crashes occurred at roads with speed limit 60 kph that represent the majority of road network especially at city center area.

It is worth mentioning that, the Emirate of Abu Dhabi has a unique profile of population where more than two hundred different nationalities are living there. The Emiratis comprise about 18% of the total population of about three million. The majority of the population falls under the working age group (between 18 and 45 years old), which in-turn leads to challenges in traffic mobility and other road safety concerns. As a result, the majority of the pedestrian's casualties (about 60%) fall in the age group from 30 to 45 years old.

Table 2: Pedestrian crashes distribution

	Category	Frequency	Percentage
Pedestrian behavior	During crossing roads	3,462	92.5%
	crossing at intersection	266	6.5%
	crossing at mid-block crosswalk	239	5.8%
	crossing at mid-block not at crosswalk	3283	80.2%
Injury-severity level	Slight	951	23.2%
	Medium	2007	49.0%
	Serious	509	12.4%
	fatal	629	15.4%
	Total	4,096	100%
Age group	< 18	763	18.6%
	18-30	1215	29.6%
	31-45	1252	30.5%
	46-60	680	16.5%
	> 60	199	4.8%
Gender	Male	3146	83.5%
	female	622	16.5%
Speed limit of the road	< 60	986	26.2%
	60	1616	42.9%
	80	419	11.1%
	100	526	14.0%
	120	223	5.9%

Such unique composition of population put challenges to traffic police for improving road safety. Several actions including engineering constructions especially for pedestrian, awareness and education campaigns and automated speed enforcement devices insulations in order to reduce the number of injuries and fatalities resulted in traffic crashes were taken during last five years. As a result, severe crashes reduced from 2,537 in year 2010 to 1,803 in year 2015 that led to a reduction in the fatalities from 376 to 245 (34.8% reduction) during the same period. The statistics shown in Table 3 indicates that a significant improvement in pedestrian crashes and related fatalities has been occurred during the last six years. However, the fatality rates of pedestrian crashes per 100,000 inhabitants and 10,000 registered vehicles remain the same during the last three years. These findings can be justified due to a pedestrian initiative for reducing pedestrian crashes that was conducted between 2010 and 2013 including intensive constructions of crosswalk facilities in AD during this period.

Table 3: Statistics of the total and pedestrian-related crashes in AD

Item	2008	2009	2010	2011	2012	2013	2014	2015
Total number of severe crashes	2,954	3,086	2,537	2,283	2,056	2,071	1861	1803
Total number of fatalities	376	409	376	334	271	288	267	245
Number of pedestrian crashes	625	644	531	486	401	388	334	324
Number of severe injuries in pedestrian crashes	76	93	76	63	77	58	26	41
Number of fatalities in pedestrian crashes	110	118	101	81	70	48	54	48
Pedestrian fatality rate /100,000 inhabitants	3.9	3.8	3.2	2.6	2.1	1.5	1.5	1.5
Pedestrian fatality rate / 10,000 registered vehicles	2.0	1.7	1.4	1.0	0.8	0.5	0.6	0.5
% of pedestrian crashes	21%	21%	21%	21%	20%	19%	18%	18%
% of fatalities in pedestrian crashes	29%	29%	27%	24%	26%	17%	20%	20%

To keep the improvements in pedestrian safety, more understanding about the factors that affect the severity of pedestrian crashes while they are crossing the roads is needed. Two approaches are usually used to enhance the safety of pedestrians: (1) reducing the number of vehicle-pedestrian crashes and (2) reducing the severity level of the injury when pedestrians are exposed to vehicle collisions. Fatality rates especially fatality rate per 100,000 inhabitants, are consider as main performance indicators to monitor the improvement in road safety and for comparison reasons over time and between different countries. Accordingly, this paper focuses on investigating the contributing factors that affect the severity of vehicle-pedestrian collisions.

LITERATURE REVIEW

There are many researchers have attempted to investigate the factors affecting the occurrence and severity of pedestrian crashes. Pour-Rouholamin and Zhou (2016) used ordered-response models to test the contributing factors of pedestrian-vehicle collision occurred in Illinois between 2010 and 2013. It was found that many variables are associated with severe injuries: older pedestrians (more than 65 years old), pedestrians not wearing contrasting clothing, younger drivers (16–24), drunk drivers, time of day (20:00 to 05:00), divided highways, multilane highways, darkness, and heavy vehicles. In addition, crossing the street at crosswalks, older drivers (more than 65 years old), urban areas, and presence of traffic control devices (signal and sign) are associated with decreased probability of severe injuries.

Jimenez, et al. (2016) applied logistic regression modeling to quantify the association between pedestrian and driver related factors by using crash data between 1993 and 2011 in Spain. The study suggested preventive measures for subgroups of high-risk road users including young and male pedestrians, pedestrians with psychophysical conditions or health problems, the

youngest and the oldest drivers, and drivers with markers of high-risk behaviors (alcohol use, nonuse of safety devices, and driving without a valid license). Abdul-Aziz et al., (2013) used log-likelihood ratio test associated variables of pedestrian severity levels in New York City. This study showed that, among many variables, road characteristics (e.g., number of lanes, grade, light condition, road surface, etc.), traffic attributes (e.g., presence of signal control, type of vehicle, etc.), and land use (e.g., parking facilities, commercial and industrial land use, etc.) are found to be statistically significant in the pedestrian severity.

Regarding pedestrians' behavior, Preusser et al. (2002) and Oxley et al. (2006) stated that more than half of the pedestrian deaths occur due to illegal human behavior in UAS, such as violating traffic lights, impairment by alcohol or drugs, misjudgment, fatigue and poor pedestrian visibility. Spainhour et al. (2006) reviewed about 353 pedestrian fatalities and found that alcohol use by pedestrian is the primary factor in 45% of all cases. The results also showed that the use of cell phones and other distraction could increase the severity and the risk of the pedestrian crash.

In terms of pedestrian characteristics, two demographic factors (i.e., age and gender) have been addressed in prior researches (for example; Rosén and Sander 2009; Kim et al. 2008; 2010). Holland and Hill (2007) showed age and gender differences in the likelihood of adult pedestrian crashes. It was found that older people do take fewer risks, being less likely to intend to cross in risky situation. In addition, women are less likely illegally cross roads than what men do. Tournier et al., (2016) provided a reviews current knowledge of older-adult problems with the main components of pedestrian activities. It showed that visual impairment, physical frailty, and attention deficits have a major negative impact on older pedestrians' safety and mobility, whereas the roles of self-evaluation and self-regulation are still poorly understood. Accordingly, all these elements must be taken into consideration, not only in developing effective safety interventions targeting older pedestrians, but also in designing roads and cars. However, Fredriksson et al., (2010) showed that all factors are equally influences on pedestrian crash severity in terms of gender and there is no gender difference in fatality risk.

Other researchers investigated the impact of traffic volume of both vehicles and pedestrians on the occurrence pedestrian crashes (i.e., Elvik (2009), Megan et al. (2009) and Zegeer et al. (2005)). The developed models in these studies showed that pedestrian and vehicle traffic volumes have a significant contribution with positive sign which means the pedestrian-related crashes increase with increasing the traffic volumes of pedestrian and for both locations of marked (designated) and unmarked crosswalks.

DATA DESCRIPTION

The employed data in this study were extracted from the crashes reports of AD traffic police from year 2008 to 2015. The crashes database includes different sets of information as summarized in Table 4.

Table 4: Data reported by AD traffic police in cases of pedestrian crashes

Data set	Collected information
Pedestrian and casualties Characteristics	Age, gender, nationality, and injury severity level (Four levels: Slight, Medium, Severe, and Fatal) *
Pedestrian behavior	Crossing location related to the available crossing facility (crossing at zebra crossing area, not at the zebra area, facility not available)
Road Characteristics	Accident location (intersection/mid-block/out of the road/median), exist of crossing facility, speed limit, number of lanes, road type and class, surrounding land use, and area category
Environmental and general Information	Accident location, coordination, weather condition, lighting, road surface condition, vehicle type, accident description, and accident causes, driving license data, etc.

* The level of injury is determined from the medical reports of the accidents

REGRESSION MODEL DEVELOPMENT

As indicated earlier, the main objective of this study was to identify the factors that might affect the severity of pedestrian-vehicle collisions in AD. The response variable (crash severity) is a binary variable with two levels: “1” if the crash resulted in a pedestrian’s fatality or more and “0” if the crash resulted in at least one pedestrian’s injury. The binary logistic regression is the best modeling approach to deal with this nature of outcomes (i.e., whether it was a fatal or non-fatal pedestrian’s crash).

The logistic regression model can be written in the following form:

$$E(Y/X) = \pi(X) = \frac{e^{B_0+B_1x}}{1+e^{B_0+B_1x}} \dots \dots \dots (1)$$

Where the transformation of the $\pi(x)$ logistic function is known as the logit transformation:

$$g(x) = \ln \left[\frac{\pi(X)}{1-\pi(X)} \right] \dots \dots \dots (2)$$

The logistic regression model estimated in this study can be expressed as follows:

$$P(\text{fatal crash}) = \pi(X) = \frac{e^{g(x)}}{1+e^{g(x)}} \dots \dots \dots (3)$$

Where, $g(x)$ is the function of the examined explanatory (independent) variables that shown in

The examined variables were classified into three groups: (I) pedestrian characteristics including, age and gender, (II) pedestrian behavior including, crossing location selection, pedestrian position, crash cause and (III) environmental characteristics including, speed limit,

road width in terms of the number of lanes, road type, light condition, road surface condition, existence of formal crosswalk, weather condition, day-time, week-day and the surrounding land use type. Several trials were conducted to select the most appropriate variables to estimate the mode and integration in some categories were done in these trails. Table 5 shows the description of the selected explanatory variables that have been used in the estimation of the logistic model.

In this study logistic regression was estimated using the SPSS package.

Table 5: description of the explanatory variables used in the logistic regression model

Variable	Code value	Frequency	Percentage	Mean	Std. dev.
Crash severity	Fatal = 1 , Injury = 0	612	16.2%	0.162	0.369
(I) pedestrian characteristics					
Gender	Male = 1, Female = 0	3146	83.5%	0.835	0.371
Age	Young pedestrian (<18) = 1 , Otherwise = 0	732	19.4%	0.194	0.396
	Older pedestrian (> 60) = 1 , Otherwise = 0	185	4.9%	0.05	0.216
(II) pedestrian/driver behavior					
Cross at crosswalk	Cross at crosswalk = 1, Otherwise = 0	420	11.1%	0.111	0.315
location doesn't have crosswalk	Not crosswalk exists = 1, Otherwise = 0	2342	62.1%	0.620	0.185
Pedestrian position	Crossing the road = 1, Otherwise = 0	3510	93.1%	0.931	0.253
Speeding driving	Speeding = 1, Otherwise = 0	350	9.3%	0.093	0.290
giveaway to pedestrian	give away = 1, Otherwise = 0	2023	53.7%	0.537	0.499
(III) environmental characteristics					
Road type	Urban = 1, Rural = 0	1739	46.1%	0.461	0.499
Speed limit	Continues variable	3770	100%	65.3	24.8
No. of lanes	Continues variable	3770	100%	2.56	0.91
Crash location	Mid-block section= 1, Otherwise = 0	3220	85.4%	0.854	0.353
Light	Day time and good lighting at night = 1, Otherwise = 0	3539	93.8%	0.939	0.240
Weather	Clear weather = 1, Otherwise = 0	3639	96.5%	0.979	0.145
Road surface condition	Dry = 1, Otherwise = 0	3488	92.5%	0.925	0.263
Surrounding land use	Commercial area = 1, Otherwise = 0	1405	37.3%	0.373	0.484
	School and government area =1, Otherwise = 0	615	16.3%	0.163	0.370
Weekday	Weekend=1 , Otherwise = 0	997	26.4%	0.264	0.441
Day time	During daytime = 1, , Otherwise = 0	2632	69.8%	0.698	0.459

Modelling Results and Discussions

Table 6 shows the output of the estimate logistic regression model. The Odds ratio is directly related to the probability of having a more fatality crash. The variable with positive Odds ratio denotes the increasing probability of getting a fatal pedestrian/s in a certain pedestrian-vehicle collision. As shown in Table 6, among the 20 examined variables, twelve variables significantly affect the fatality of the pedestrian-related crashes at a significant level of 95%. These variables varied between pedestrian characteristics, pedestrian and driver behavior, crash location characteristics and road parameters. Among these twelve variables, five variables refer to the drivers' characteristic and behavior; young age, older age, crossing at location that has not designated crosswalk, position of the pedestrian and speeding of the driver. The road variables include number of lanes, speed limit of the road and road type. Finally, the environmental and crash location variables include intersection-related location, commercial area; light condition and road surface condition are significantly affect pedestrian severity.

Table 6: Result of the logistic regression model estimation

Variables	Estimate	S.E.	Wald	Sig.	Odds ratio	95% C.I.for EXP(B) (odds ratio)	
						Lower	Upper
Constant	-6.708	.580	133.911	.000	.001		
Gender	.111	.156	.509	.476	1.118	.823	1.517
Young age < 18 years old	.356	.145	6.038	.014*	1.427	1.075	1.895
Older age > 60 years old	.769	.217	12.602	.000*	2.158	1.411	3.300
No crosswalk available	.385	.138	7.760	.005*	1.470	1.121	1.928
Crossing at crosswalk	.349	.210	2.768	.096	1.417	.940	2.136
Pedestrian position	-.723	.216	11.233	.001*	.485	.318	.741
Crash location	.337	.160	4.415	.036*	1.400	1.023	1.917
Giveaway to pedestrian	-.198	.113	3.051	.081	.820	.657	1.024
Speeding drive	.906	.181	25.000	.000*	2.474	1.735	3.529
Commercial area	-.358	.125	8.273	.004*	.699	.547	.892
School and Governmental zones	-.243	.159	2.318	.128	.785	.574	1.072
Light condition	-.649	.182	12.679	.000*	.523	.366	.747
Number of lanes	.280	.066	17.998	.000*	1.323	1.163	1.506
Speed limit of the road	2.307	.136	289.517	.000*	10.045	7.701	13.103
Road type	-.919	.157	34.307	.000*	.399	.293	.542
Road surface condition	-.559	.193	8.414	.004*	.572	.392	.834
Weather condition	.560	.354	2.502	.114	1.750	.875	3.502
Daytime	-.161	.110	2.163	.141	.851	.687	1.055
Weekday	-.038	.116	.107	.744	.963	.767	1.208

* Significant variables at level of 95%

The odds ratio presented measure the amount by which the crash severity increases. Taking an example of the explanatory variable, no crosswalk available, which has an odds ratio of 1.470, it can be stated that the probability of fatal pedestrian's crash is about 1.47 times higher in cases of the pedestrians cross at locations that haven't designated crosswalk, assuming that rest of the factors remains the same. The likelihood to be involved in fatal pedestrian crashes at not designated crosswalk locations, rural roads, darkness and mid-block locations are significantly high compared to others. In addition, pedestrian more likely to be killed on roads with higher speed limit ≥ 60 kph) by 10 times more than in case if the crash occurred on roads with speed limit less than 60 kph. Furthermore, the fatality increases with increasing number of lanes.

Regarding pedestrians' characteristics, the age showed a significant variable which young (less than 18 years old) and older (greater than 60 years old) are more likely to be involved in a fatal than injury in the pedestrian-related crashes. Among the drivers' behavior variables, the speeding is the most effective variable since the speeding behavior increases the probability to involve in fatal-crash rather than injury-crash. The speeding behavior of vehicle's drivers increases the probability to involve in fatal pedestrian crashes rather than injury-crash. The results also indicate that if the vehicle hits the pedestrian when he/she stopes at roadside, the probability to be a fatal crash is higher than in case of the vehicle hit the pedestrian during cross the road. That can be justified as this type of crashes usually occur when the driver lost control of the vehicle may be due to high speed or Preoccupation with something like a mobile use while driving and then run out of the road and hit a pedestrian. In this scenario, driver hit the pedestrian in high speed that usually lead to his/her death.

Countermeasures for Improve Pedestrian Safety

In light of the model findings, preventive strategies could be developed in order to reduce the fatality of the pedestrian-vehicle collisions in AD. Table 7 summarizes some of possible preventive countermeasures for each of the factors that have been proven as significant variables in the regression model. It seems the proposals not new or unfamiliar proposals but most of them have been proposed in prior studies (for example, lee and abdel-Aty, 2005, World Health Organization, 2013b). This table indicates that improving the pedestrian behavior by the three main elements of road safety; engineering, education and enforcement should be conducted in parallel to achieve the target of AD police for deducing the fatality rates resulted in pedestrian crashes. Integrated efforts with other authorities that have responsibility in road safety such as DOT and municipalities should also be take into consideration during implementation of the pedestrian safety strategies.

Table 7: Proposed countermeasures for pedestrian safety risk factors

Risk factor identifies in the model	Possible preventive countermeasures
Young age < 18 years old	<ul style="list-style-type: none"> - Emphasize awareness campaign for road safety at schools and colleges - Improving road-crossing facilities at school and education intuitions area
Older age > 60 years old	<ul style="list-style-type: none"> - Develop safe walking paths for children in high-risk locations - design the road crossing facilities as friendly as possible for older and handicapati peoples.
No crosswalk available	<ul style="list-style-type: none"> - Construct designated crosswalks including crossover at high crossing pedestrian demand locations.
Pedestrian position & Crash location	<ul style="list-style-type: none"> - Construct guardrails at roads median to prevent illegal crossing at high-risk locations. - Emphasize road safety education for both drivers and pedestrian regarding taking care and be aware always.
Speeding driving	<ul style="list-style-type: none"> - Emphasize enforcement for illegal road crossing - Emphasize enforcement for speeding behavior for drivers. - Emphasize awareness campaign for speeding behavior at driving training schools and by different ways.
Commercial area	<ul style="list-style-type: none"> - Improving road-crossing facilities at commercial areas
Light condition	<ul style="list-style-type: none"> - Installing light poles at high-risk locations. - Improving visibility at hazard locations.
Number of lanes	<ul style="list-style-type: none"> - Prevent at-grade crossing facilities and construct pedestrian crossovers at wider roads (4 lanes and more), roads with high speed limit (greater than or equal 100 kph and at rural roads.
Speed limit of the road	
Road type	
Road surface condition	<ul style="list-style-type: none"> - Emphasize awareness campaign regrading prohibition to crossroads during wet conditions and to be aware in rainy weather

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Characteristics of Severe Crashes during the Holy Month of Ramadan

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ABSTRACT

Ramadan is the holy month for Islamic World. In this month more than 1.6 billion Muslims around the world fast during day time. Accordingly, daily life routines and activities of people change markedly. This paper aims to explore the changes in the traffic crash patterns during Ramadan compared to the rest months of the year. Severe crashes data during six years at Abu Dhabi Emirate are employed. The statistics shows that the percentage of crashes during Ramadan remains between 7 and 8% of the total yearly crashes. Despite the severe crashes decreased by 29.2% during last six years, the severe crashes during Ramadan decreased by 38.6%. Logistic regression model was developed to examine the contributing factors affecting the crashes occurrence during Ramadan compared to other months. The model results showed that five variables significantly affect the crash occurrence in Ramadan; day time, week days, crashes caused by red light crossing and alcohol usage and feeling sleep and fatigue. Among these variables, crashes caused due to sleeping and fatigue of drivers has higher probability to occur during Ramadan 2.4 times more than other months. In addition, crashes due to alcohol usage decreased by 4.23 times. Furthermore, the results indicated that young drivers are more committed to traffic rules and drive more safely during Ramadan. These findings reflected the impacts of fasting and change of daily activities on the road safety in AD.

INTRODUCTION

There are more than 50 Muslim majority countries and over 1.6 billion Muslims around the world which represents about 23.4% of the world population. The holy month of Ramadan is considered as a special month for Muslims world where many religious activities take place, mainly the fasting. Ramadan is the 9th month of the Hijri (Islamic / Lunar) calendar. Ramadan month doesn't start at the same time every year because the lunar year is 11 days shorter than the Gregorian (our conventional) calendar. Ramadan starts and ends at the sighting of the new moon, so sometimes its exact start and end date are a little unpredictable and it may differ from country to country. Ramadan's occurrence varies each year and may in some years falls during very hot summer months, so it is usually a challenging time to remain safe on the road during the summer months.

Emirate of Abu Dhabi, the Capital of UAE, is a Muslim Country. However, there are more than two hundred different nationalities are living there. In Ramadan, working hours are changing for all the people in UAE with all respect from non-Muslims to the religious environment of this month. During the last six years a significant improvement in road safety has been gain in AD. Table (1) shows the statistical information of road safety indicators in AD. It shows that the frequency of severe crashes (i.e., crashes that have casualties; hereafter said traffic crash). In general, all indicators show a significant improvement in road safety during this period by different levels. The crashes reduced by about 29.2% in year 2015 compared to 2010. However, it was reduced by 38.6% for crashes that have been occurred during Ramadan. That means the improvement more significant in the month of Ramadan regarding the pedestrian-related crashes; the number of pedestrian fatalities decreased by 52.5% but not changes has been occurred during Ramadan.

TABLE 1: Statistics of Crashes during Ramadan in AD

Abu Dhabi Emirate	2010	2011	2012	2013	2014	2015	% of change
Total number of crashes	2,537	2,283	2,056	2,071	1,864	1,797	-29.2%
Severe crashes in Ramadan	215	176	149	166	157	132	-38.6%
% of crashes in Ramadan	8%	8%	7%	8%	8%	7%	-
Total number fatalities	376	334	271	289	267	245	-34.8%
Number of fatalities in Ramadan	28	22	20	30	25	19	-32.1%
% of fatalities in Ramadan	7%	7%	7%	10%	9%	8%	-
Total number of serious injuries	400	390	364	366	240	293	-26.8%
Number serious injuries in	41	32	26	43	17	29	-29.3%
% of serious injuries in Ramadan	10%	8%	7%	12%	7%	10%	-
Total number of pedestrians' fatalities	101	81	70	48	54	48	-52.5%
Number of pedestrians' fatalities in Ramadan	6	4	5	7	5	6	0.0%
% of pedestrians' fatalities in Ramadan	6%	5%	7%	15%	9%	13%	-

In general, the changes of people daily life style are markedly altered during Ramadan. The commitment to eat and drink only during the night leads to a definite change in the rhythm of life; sleep duration becomes shorter, eating schedule, and the alternation of rest and activity are especially affected. Understanding how such changes affect drivers' behavior and as a result affect traffic crashes pattern is a key for improving traffic safety performance during the month of Ramadan. Accordingly, this paper tries to investigate the factors that may affect the frequency of traffic crashes, crash type and traffic crashes patterns during Ramadan.

LITRITURE REVIEW

Monthly and seasonal variations of traffic accidents at different places have gained a significant attention from the researches. Many studies tackled the impact of special events on the traffic accident patterns and road safety aspects, such as during Christmas and holiday season, in western countries. However, very few studies addressed the changes in the traffic accident pattern during the month of Ramadan despite the relatively big number of Muslim countries and Muslim population.

A study conducted by Yildirim et al. (1) used observational data to examine how Ramadan was associated to observable driving behaviors (i.e., speeding, horn honking, and using seat belts) as compared to non-Ramadan. The observational survey was conducted during and after Ramadan in different times of the day in the same region of the city of Ankara. The results demonstrated that mean speed was lower, honked horns were higher, and seat belt use was lower in Ramadan as compared to non-Ramadan, though each negative driving behavior was prevalent in both periods.

Hossein Ebrahimi et al. (2) found that the significant reduction in the incidence of traffic accidents in August (2011) may be due to the coincidence of August with Ramadan, which had led to a significant decrease in out of city journeys.

M.N. Tahir et al. (3) investigated trends in road traffic crashes (RTCs) managed by an emergency service, in 2011 in Punjab, Pakistan. Results of the study showed that ambulance service experienced a higher burden of RTC emergencies in the month of Ramadan 2011 compared with the preceding months of the year. This increase was mostly concentrated among younger ages, especially those driving motorcycles. Furthermore, the majority of RTCs were caused by speeding during the peak rush hours before Iftar (breakfast).

Shawky M. et al. (4) investigated the change in the traffic accident patterns and road safety concerns during the holy month of Ramadan in Abu Dhabi, UAE during the period (2008-2012). His comparison study is carried out using statistical t-test to examine the difference between Ramadan and other months with respect to traffic accident patterns (over months, weekdays, and daytime), accidents' severity, causalities, types, and causes. In addition, a comparison of the main traffic safety indicators is presented. The comparisons have been further extended to the drivers' behavior in terms of their traffic violation, attitude of speed limit, crossing during red light, etc. The findings of this study indicated that there are significant differences between the traffic accident fluctuation during the month of Ramadan compared to other months across days of the week and hours of the day. No evidences of that the traffic accident number, severity, causes, and

types have been significantly changed during the month of Ramadan. However, that study didn't provide statistical model to investigate the contributing factors affecting the occurrence of crashes during Ramadan.

Al-Houqani et al. (5) showed that driving during Ramadan was a predictor of sleep related collisions which was prevalent during this particular period. Monirah Alnasser et al. (6) conducted a retrospective study, which included all pediatric trauma cases related to Motor Vehicle crashes, from 2001 to 2009, which were registered in King Abdulaziz Medical City Trauma Registry. Trauma patterns were divided into two groups according to the date of occurrence: victims in Ramadan versus victims in non-Ramadan. They compared trauma in the pediatric age group in Ramadan with non-Ramadan months in terms of frequency, patterns, and severity. The results showed that there were no significant differences between trauma cases related to Motor Vehicle crashes in Ramadan and non-Ramadan months, except for the higher percentage of vascular and neurological injuries in Ramadan.

Another study conducted in Karachi, which analyzed frequency of injuries in Ramadan in 2009 and 2010, found that traffic behavior changed in the approach to Iftar (the time of breakfast) and speeding was observed, increasing the chances and severity of crashes (7).

Another study conducted by Yildirim et al. (8) used observational data to investigate the risky driving behavior during Ramadan. The observational survey was conducted in any three days during and after Ramadan in different times of the day (i.e. morning, afternoon and evening) for driver violations (i.e., speeding, mild aggressive driving, and not using a seat belt). The results demonstrated that more frequent aggressive driving, less seat belt use and less speeding were observed during Ramadan than before it. These results were discussed in terms of moral, spiritual and self-regulatory explanations, as well as religion's distal influence on traffic behavior.

Another study conducted on 2007 by Tolon et al. (9) examining traffic accidents in Turkey between 1984 and 2004 found that the number of accidents was not higher in Ramadan compared to other months. Muhammad et al. (10) observed that the rate of road traffic accidents was lower during the month of Ramadan than other months in Jordan. They believed that delaying the daily time of work and school beside the religious and spiritual atmosphere that embraces people during the holy month of Ramadan is the main reasons of these results.

Variety of factors including alteration in traffic pattern or observation of traffic rules in Ramadan may influence the frequency of car accidents. Rezaei N. et al. (11) conducted study to compare car accidents in Ramadan with other months during a 4-year period (1997-2000). They extracted frequency of car accidents from related files of Police Office in Tehran and stratified according to the months. Then frequency of car accidents was estimated in Ramadan and compare with other months. Their study concluded that car accidents in Ramadan did not significantly differ from other months. They also found that changes in the traffic pattern and people's behavior to observe the regulations and the rights of others are among the factors which increase or decrease accidents during Ramadan compared to the other months.

Rachida et al. (12) found that diurnal fasting induces an increase in subjective and objective daytime sleepiness associated with changes in diurnal rectal temperature. Also, Kadri et al. (13) noted that significantly increased irritability during the month of

Ramadan; this is noticed more among smokers and especially related to the decrease in the sleeping hours.

DATA COLLECTION AND METHODOLOGY

As mentioned before, the month of Ramadan doesn't start at the same date every year due to the differences between the lunar year and solar year. Ramadan starts and ends at the sighting of the new moon, so sometimes its exact start and end date are a little unpredictable. Also, the start and end dates of the month of Ramadan may differ from country to another at the same year around one-day shift. The start and end dates of the month of Ramadan during study period in UAE are shown in Table 2.

TABLE 2: Dates of Ramadan months during the last six years in UAE

Year	Start Day of Ramadan	End Day of Ramadan
2010	11 th August	9 th September
2011	1 st August	29 th August
2012	20 th July	18 th August
2013	10 th July	7 th August
2014	29 th June	28 th July
2015	18 th June	17 th July

The employed data in this study was extracted from the traffic crashes database of Abu Dhabi Traffic Police for six years from 2010 to 2015. Severe crash (i.e. any crash involving at least one injury) data are used in this analysis due to the limitation of the available property damage only crash data. The database includes different sets of data groups; basic crash information, at-fault drivers and vehicle's characteristics, casualties' information, environmental and surrounds condition, traffic violations, etc. Full data of about 12,611 crashes has been extracted and utilized in this study. It is worth mentioning that the driver's community in AD consists of more than one hundred different nationalities. Male drivers represent 85% of total number of licensed drivers and 92% has age less than 45 years.

To achieve the objectives of this study, detailed descriptive statistical analysis will be firstly provided. After that an investigation of the contributing factor that affects Ramadan related crashes (i.e. crashes occurred during Ramadan) will be conducted by applying logistic regression modeling approach.

STATISTICAL DESCRIPTIVE ANALYSIS

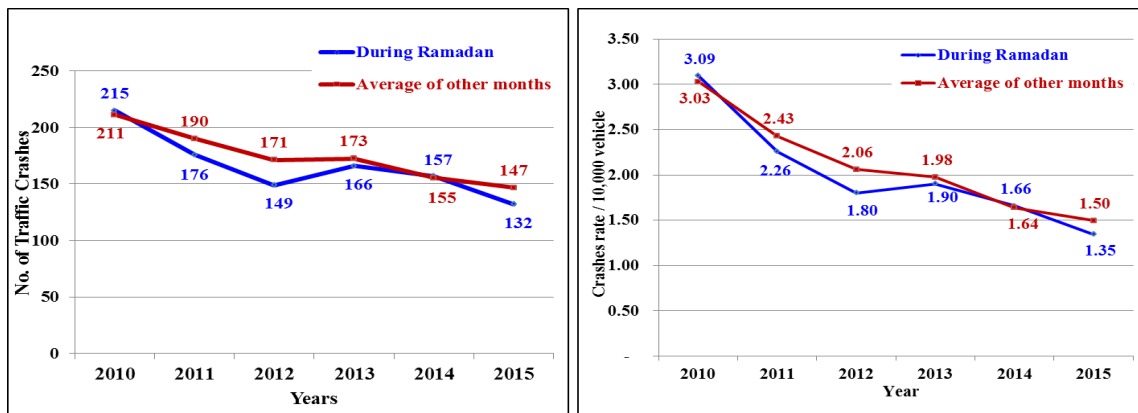
Comparisons between the month of Ramadan and the rest of months of the year was conducted in terms of the frequency of traffic crashes, crash severity, weekdays, daytime, crash types, crash causes and traffic violations as follows:

Frequency of Crashes Cross Years

Figure (1-a) presents the number of severe crashes that occurred during Ramadan and the average monthly of rest months. In general, a significant decrease (about 29.2%) in the number of severe crashes was occurred during the month of Ramadan in the last six years. In addition, a decrease in the average monthly number of crashes for the same period, where the number of crashes decreased in the month of Ramadan, during the

study period by 38.6%, while the monthly average of crashes declined across the emirate by 30%.

The crashes fatalities in Ramadan decreased during the study period by 22.7%, while the monthly average of the fatality incidents at the emirate level significantly decreased by 37% as shown in Figure (1-b) which presents the crashes rates per 10,000 vehicles. These two figures indicate that serious traffic crashes during Ramadan are around the average value of the rest of months. Furthermore, Figure 1 shows a gradually improvement in number and rate of traffic crashes during the last six years in AD. The t-test analysis proved that no significant difference for serious accident number between Ramadan and other months of the year at significant level of 5%.



a) Crashes Frequency

b) Crashes rate per 10,000 vehicles

FIGURE 1: Traffic crashes during Ramadan and the rest months

Crashes Frequency Cross Weekdays

The analysis of traffic crashes number across the days of the week showed that one of the weekend days (Friday and Saturday in UAE), which is Saturday, have the highest number of accidents and Thursdays (The last day of Weekdays in UAE) also have the highest number of crashes at each year individually. Also, during Ramadan the highest number of traffic crashes occurred on same days mentioned above during the last six years. The lowest number of crashes occurred at the weekdays like the rest months of the year. Figure 2 shows the percentage distribution of traffic accident across the days of the week during Ramadan and other the months over the last five years. The aggregated data of the six years showed that Saturday and Thursday gained the highest crashes number during the month of Ramadan.

Crashes Frequency Cross Daytime

Figure 3 shows the distribution of traffic crashes during Ramadan and non-Ramadan across the hours of the day. It shows a significant difference between Ramadan and the rest months of the year. The morning peak hour of accidents shifted from 7:00 to 8:00am and a clear peak hour at noon 12:00 has been occurred. These changes can be explained due to change of working hours during Ramadan. In addition, other peak hours of accidents appeared during Ramadan; at 16:00 pm (before the breakfast time), during the period after the Tarawih prayers, which extends from 20:00 pm until 12:00 at midnight, and at 2:00am (the time before starting fast, Suhoor). This clearly proves how the

changes in the routine of the daily life in Ramadan affect the traffic crashes fluctuation over the hours of the day. The highest number of incidents occurred on Ramadan was during the time from 12:00 until 13:00 in the afternoon followed by a time of 02:00 in the morning until 03:00 in the morning.

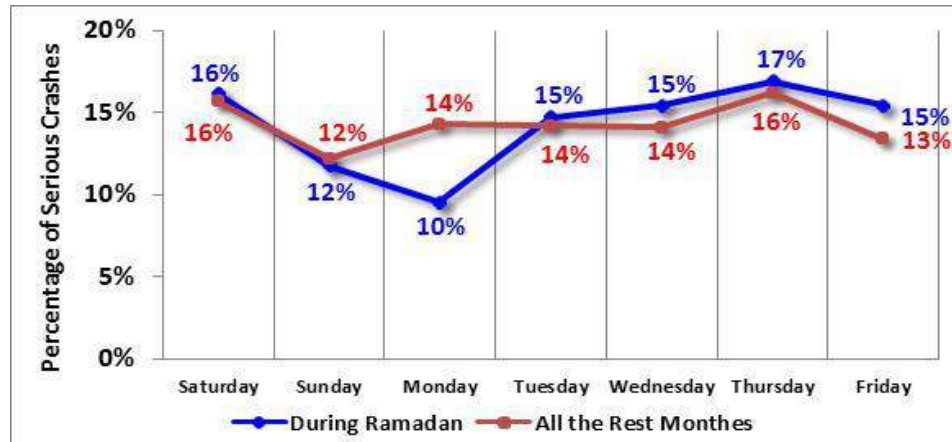


FIGURE 2: Percentage distribution of traffic crashes across the weekdays

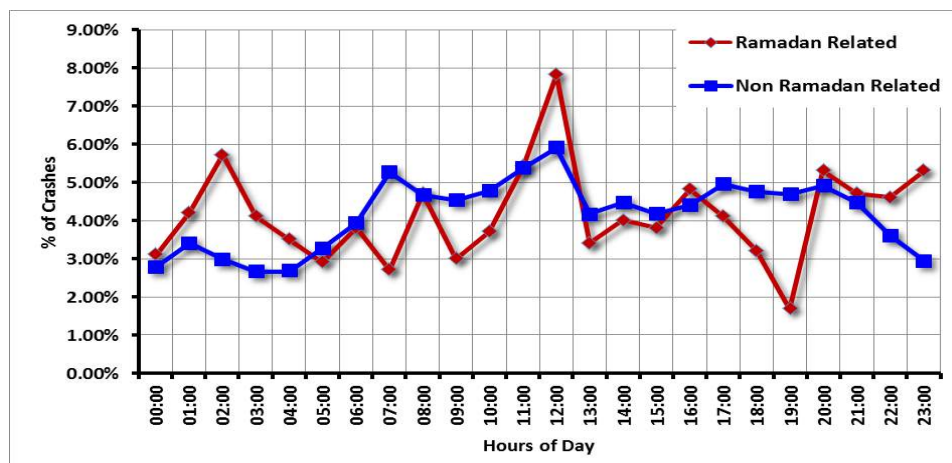


FIGURE 3: Percentage distributions of traffic crashes across hours of the day

Severity of Crashes

Figure 4 shows the distribution for number of traffic crashes fatalities during Ramadan and non-Ramadan across the study period. It shows a non-significant difference between Ramadan and the rest months of the year except the years 2011 and 2013. In generally, the number of crash's fatality in the month of Ramadan during the study period decreased by 32%, and the average monthly crash's fatality decreased in the Emirate by the same rate. In addition, Figure 5 shows the distribution for Pedestrian Crash's Fatalities & Serious Injuries during Ramadan and non-Ramadan across the study period. It shows that serious injuries and fatalities during the month of Ramadan due to pedestrian crashes are less than the monthly average during the years from 2010 to 2012, while those injuries and fatalities increased than monthly average during the years 2013 to 2015. Serious injuries and fatalities increased during the month of Ramadan as a result of pedestrian crashes increased than the monthly average during the year 2015 by 28.6%.



FIGURE 4: Number of Fatalities

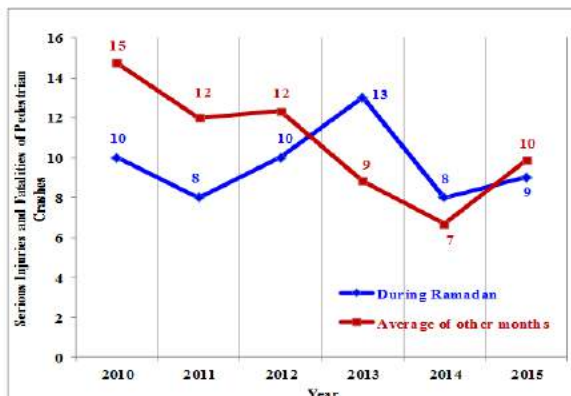


FIGURE 5: Pedestrian Crash's Fatalities & Serious Injuries

Crash Types and Causes

Table 3 shows the percentage of the highest five types of traffic crashes in Abu Dhabi Emirate during the month of Ramadan and the other months of the year. It shows non-significant differences in the percentage share values of the crash types over the six years that occurred during Ramadan and during other months for all crash types except a slight increment in turn-over accidents during Ramadan.

TABLE 3: Crash Types Distribution

Year	Turn-over		Pedestrian		Side collision		Rear end Collision		Perpendicular Collision		Others	
	No.	%	No.	%	No.	%	No.	%	No.	%	N	%
Ramadan	185	19%	182	18%	141	14%	160	16%	155	16%	171	17%
The rest months	1896	16%	2,282	20%	1,622	14%	1835	16%	1,728	15%	2253	19%

Table 4 shows the percentage of the highest five causes of traffic crashes in Abu Dhabi Emirate during Ramadan and the rest of months of the year. In general, non-significant differences exist between Ramadan and other months in terms of the causes of traffic crashes except a slight increment for those due to carelessness and sudden lane changing.

TABLE 4: Crash Causes Distribution

Year	Carelessness		Sudden lane change		Over-speeding		Tailgating		Red light Crossing		others	
	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Ramadan	120	12%	199	20%	116	12%	98	10%	101	10%	372	37%
The rest months	1048	9%	2024	17%	1579	14%	1237	11%	1019	9%	4709	41%

Figure 6 shows the distribution of sleeping & fatigue crashes during Ramadan and non-Ramadan across the study period. It shows a significant difference between Ramadan and the rest months of the year except the years 2011 and 2015.

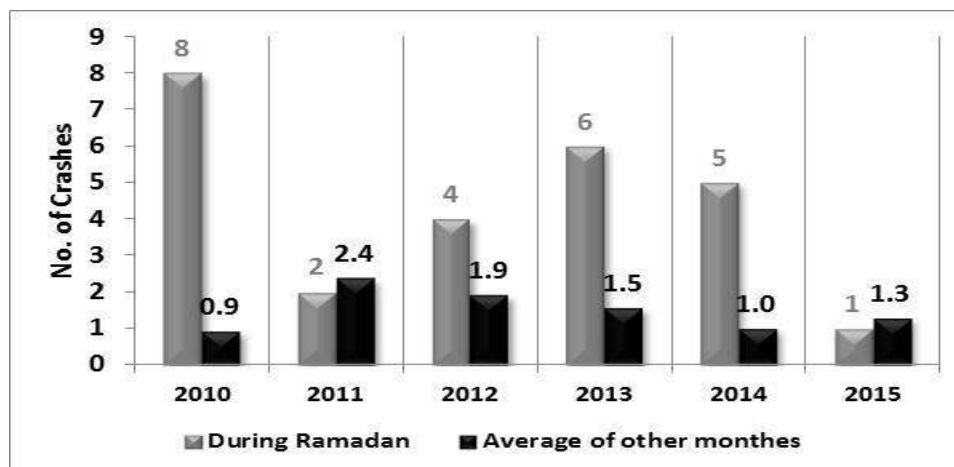


FIGURE 6: Number of crashes due to Sleeping & Fatigue of drivers

Traffic Rule Violations Records

Number of traffic violations can be used as an indicator of drivers' behavior changes. The database includes two types of violation tickets; in presence (by face to face tickets) and in absent (by automated enforcement devices). Table 5 shows that the total number of traffic violations increased in the month of Ramadan during the years 2010 to 2015 by about 109%. The total number of in presence traffic violations increased in the month of Ramadan during the years 2010 to 2013 by about 67%, and then decreased during the years 2013 to 2015 by about 41%. It was observed that the total number of absent traffic violations during the month of Ramadan increased than the monthly average during the year 2015 by 42%.

TABLE 5: Traffic Rule Violations Records In Ramadan

Violation Type	Years					
	2010	2011	2012	2013	2014	2015
Presence	33,572	37,564	36,587	56,060	24,799	32,921
Absent	137,436	251,233	259,342	259,139	170,144	325,080
Total	171,008	288,797	295,929	315,199	194,943	358,001
Changes %	-24.6%	68.9%	2.5%	6.5%	-38.2%	83.6%

Over-Speeding traffic violations represented about 84% of the total number of traffic violations during the month of Ramadan over study period, while it represented about 82% of the total number of traffic violations during other months.

Figure 7 shows the distribution of Over-Speeding traffic violations during Ramadan and non-Ramadan across the study period. It shows reduction between Ramadan and the rest months of the year especially a significant reduction on the last two years.

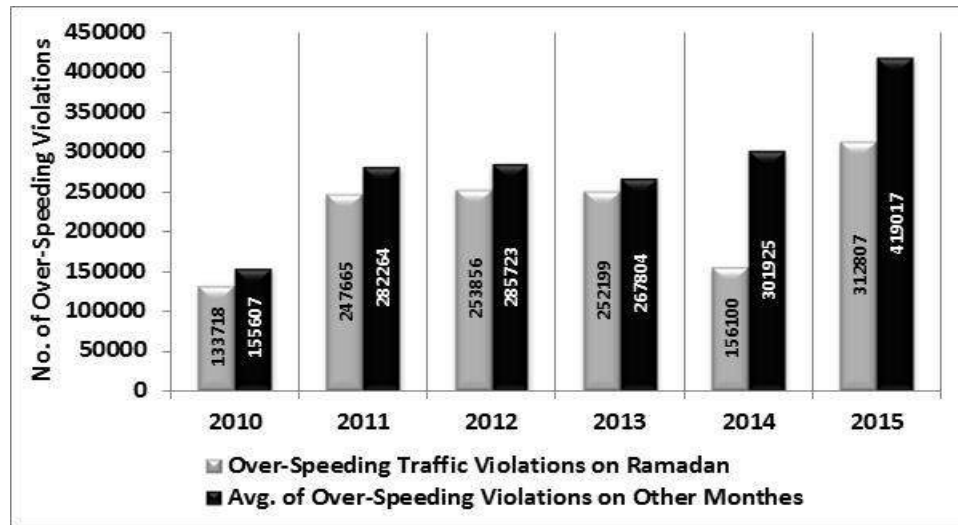


FIGURE 7: Main Traffic Violations, Abu Dhabi (2010-2015)

Figure 8 shows a significant increasing for red light violations during Ramadan. Other violation types showed a significant reduction in driving recklessly violations, drinking alcohol violation during Ramadan, despite its limited use in AD. Also, a reduction in seat belt, tailgating, and using mobile during driving violations was observed during Ramadan. These results cannot be explained based on an improvement in driver’s behavior during Ramadan that related to such violations because it also depends on the intensity of Abu Dhabi Police enforcement campaigns during the month of Ramadan.

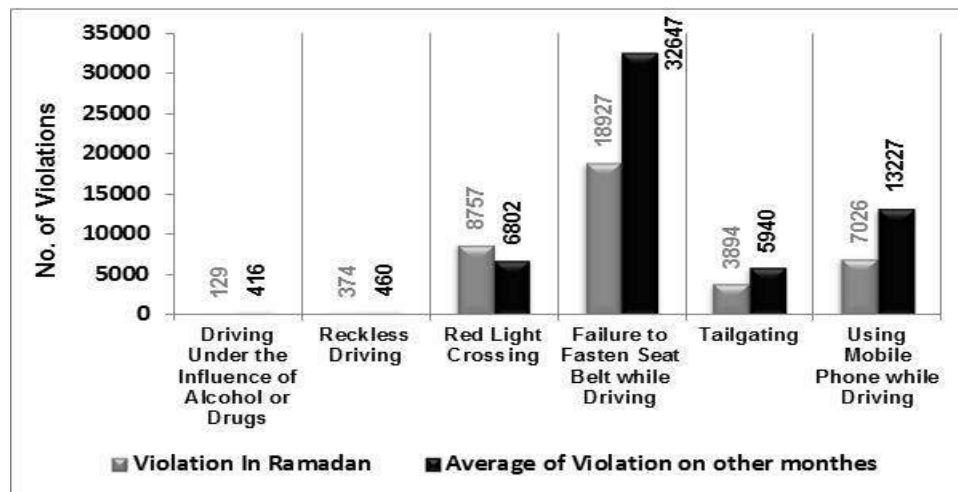


FIGURE 8: Main Traffic Violations, Abu Dhabi (2010-2015)

LOGISTIC REGRESSION MODEL

In order to define the factors affecting the occurrence of crashes during Ramadan combined to other months, logistic regression analysis applied. Binary logistic model is a good approach to deal with binary outcomes (1= Ramadan related crashes, 0= Crashes on other days of the year). Also, as one of the aims of the study was to develop models to

predict the Ramadan related crashes, logistic regression was identified as the most suitable approach to identify the important factors.

In case of binary logistic regression model, the response variable, y takes the form of either of the two binary values (0 or 1). For k explanatory variables and $i=1, 2, 3, \dots, n$ individuals, the model takes the form as follows

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

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

α = Intercept parameter,

β = Vector of slope parameters,

X_i = Vector of explanatory variables.

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

$$OR = \frac{\pi(1)/[1 - \pi(1)]}{\pi(0)/[1 - \pi(0)]} \dots \dots \dots (2)$$

Where,

OR = Odd Ratio

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

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

The model was estimated by using SPSS software package. The total number of traffic crashes involved in the model estimation is about 12,611 crashes.

Explanatory Variables

Many variables were tested includes vehicle, at-fault driver, roadway, and environment characteristics. At that stage it considered the fact that the quality of the modeling could be expected to increase to a certain level once the number of variables increases. Secondly, selection of the variables was carried out depending on the assumption that a particular variable would affect the Ramadan’s crashes. The descriptions of 17 explanatory variables that are considered for the modeling are provided along with their statistics in Table 5. All the explanatory variables are binary. Binary variables take the form of either 0 or 1; for example, if a crash occurs on WEEKEND, the variable WEEKEND has been assigned “1” as its value; otherwise “0” is assigned to this variable. One binary logistic regression model was developed by considering crash been during Ramadan as the response variable and the description of the model which is binary in nature (RAMADAN_RELATED = 1 if the occurred on Ramadan, =0 otherwise).

TABLE 5: Description of explanatory variables considered for severity modeling

Variable	Description
ROAD_SPEED	= 1 if crash occurred on road speed ≥ 100 km/h, =0 otherwise
ROAD_TYPE	= 1 if crash occurred on Urban road, =0 Rural road
VEHICLE_TYPE	= 1 if the vehicle involved in the crash was light vehicle, = 0 otherwise
DAYLIGHT	= 1 if the crash occurred during daylight, =0 after sun set
WEEKEND	=1If the crashes occur during weekend (Friday & Saturday), = 0 otherwise
GENDER_AT_FAULT_DRIVER	= 1 if at-fault driver is male, = 0 otherwise
DRIVING_EXPERIENCE	= 1 if at-fault driver has experience between (1-5) years , = 0 otherwise
LICST	= 1 If at-fault driver got his license on Abu Dhabi, = 0 otherwise
OLD_DRIVER	= 1 if the driver is over 60 years, = 0 otherwise
YOUNG_DRIVER	= 1 if at-fault driver is between 18 and 24 years, = 0 otherwise
RED_LIGHT_CROSSING	= 1 if the crash occurred due to Red Light Crossing, = 0 otherwise
ALCOHOL	= 1 if at-fault driver is under influence, = 0 otherwise
SPEEDING	= 1 if crash occurred due to driving too fast or exceed posted speed limit, = 0 otherwise
SLEEPING_FATIGUE	= 1 if crash occurred because of at-fault driver sleeping or fatigue, = 0 otherwise
COLLISIONS	= 1 if crash is any type of two vehicle collisions, = 0 otherwise
RUN_OFF_ROAD	=1 if crash type is run off road, = 0 otherwise
PEDESTRIAN_CRASHES	=1 if crash is pedestrian related, = 0 otherwise

MODEL RESULTS AND DISCUSSIONS

Table (6) shows the results of the estimated parameters of the logistic regression model. The results indicate that five variables are significantly changed during Ramadan compared to other months of the year at significant of 95%. These variables are day time, week days, crashes caused by red light crossing and alcohol usage and feeling sleep and fatigue.

The significance of the day time and weekday variable can be justified due to the change in the daytime routine of people and working time. Among these variables, crashes due to sleeping and fatigue have higher probability to be occurred during Ramadan by 2.4 times more than other months. This can be explained due to fasting and limit hours of sleeping during the month or Ramadan. In addition, crashes due to alcohol usage decreased by 4.23 times compared to other months. These findings reflect the religious environments of Ramadan (note: that alcohol usage is total forbidden in Islam) on the people behavior not only the Muslims but also non-Muslims in AD. The significance of the red light crossing crashes implies that drivers during Ramadan may

not in as high concentration as during other months. The probability to be involved in a red light crossing crash in Ramadan is higher by about 1.284 times than other months.

The results also show that four variables are significant affect crashes in Ramadan at a significant level of 90%. These variables are road type, vehicle type, gender of at-fault drivers and young drivers. It indicates that the probability of crashes on rural road is higher during Ramadan and also increases with male drivers and private small cars. However young drivers seem more committed to traffic rules and safe driving Ramadan compared to other months.

TABLE 6: Results of the estimated model

Variable	B	S.E.	Wald	Sig.	Odds Ratios
ROAD_SPEED	.101	.079	1.656	.198	1.107
ROAD_TYPE	-.149	.084	3.164	.075**	.861
VEHICLE_TYPE	.139	.077	3.244	.072**	1.149
DAYLIGHT	-.283	.067	17.561	.000*	.754
WEEKEND	-.187	.082	5.242	.022*	.830
GENDER_AT_FAULT_DRIVER	.188	.111	2.851	.091**	1.207
DRIVING_EXPERIENCE	-.012	.073	.026	.873	.988
LICST	-.028	.073	.145	.703	.973
OLD_DRIVER	-.275	.249	1.224	.269	.759
YOUNG_DRIVER	-.144	.087	2.746	.097**	.866
RED_LIGHT_CROSSING	.250	.122	4.208	.040*	1.284
ALCOHOL	-1.442	.324	19.764	.000*	.236
SPEEDING	-.075	.102	.548	.459	.927
SLEEPING_FATIGUE	.891	.224	15.885	.000*	2.439
COLLISIONS	-.055	.133	.170	.680	.947
RUN_OFF_ROAD	.143	.126	1.283	.257	1.154
PEDESTRIAN_CRASHES	-.042	.159	.071	.790	.958
Constant	-2.439	.190	165.249	.000	.087

* Significant at $\alpha = 0.95$ level

** Significant at $\alpha = 0.90$ level

CONCLUSION

The main objective of this paper is to investigate the traffic crash patterns during the holy month of Ramadan in Abu Dhabi Emirate, and investigate the factors that affect the traffic crashes type and severity during Ramadan. Traffic crashes and violations data from AD Traffic Police database during six years (2010-2015) were employed in this study. The main findings of the data analysis and modelling results can be summarized as follows:

- Severe crashes during Ramadan have been decreased by 395 from 2010 to 2015. However, the total crashes have been reduced by 29.6%.

- Total number of traffic rules violations were reduced during Ramadan compared to other months.
- There are significant changes in traffic crashes patterns across hours of the day been proved, and new peak hours of traffic crashes have been raised during Ramadan.
- The crashes caused by sleeping and fatigue of drivers have higher probability to be occurred during Ramadan by 2.4 times more than other months of the year.
- Crashes caused due to alcohol usage decreased by 4.23 times compared to other months
- The probability of crashes on rural road is higher during Ramadan and also increases with male drivers and private small cars.
- Young drivers seem more committed to traffic rules and do more safe driving during Ramadan compared to other months.

These findings reflected the impacts of fasting, changing in daily activities of the drivers and the religious environments of the holy month of Ramadan on the road safety in AD.

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ABSTRACT

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Development and Deployment of Asset Management Systems for Ashghal

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, polylines and polygons 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: 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 geo-referencing 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 polylines and polygons 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.

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

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



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 detect 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 – to 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.

Structural Capacity

The Falling Weight Deflectometer (FWD) is used to assess 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 protect the traveling public and the testing crews.

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.

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.

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 and 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 not of much 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 in to 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.

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 be 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.

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.

CONCLUSIONS

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.

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PAPER TITLE	Mobile Mapping Applications for Data Collection and Evaluation of the Rural Road Assets		
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KEYWORDS:

Department of rural road, Global Navigation Satellite System (GNSS), Inertial Measurement Unit (IMU), Mean Sea Level, Panorama View Image

ABSTRACT:

Since the Department of Rural Road (DRR) has more than 47,100 km of road network, surveying the whole rural network would take a lot of resource, equipment and labor work. Therefore, Mobile Mapping System (MMS) has been used to achieve this goal. MMS has begun in 2014. Due to its initial stage, MMS can only operate on asphalt and concrete pavements. The MMS has been utilized on over 2,500km of road, from six difference provinces; Sakaew, Chachoengsao, Samutprakan, Nonthaburi, Nakhonnayok and Prachinburi. MMS records 16 frames per second, and is attached on a normal four-wheel vehicle which operates less than 60 kilometers per hour.

The visual data, which has been collected by two Ladybug-3cameras installing on a top of a vehicle, was processed to produce 5,400-time-2,700-pixel image. Asset's position was acquired from the Global Navigation Satellite System (GNSS) and Inertial Motion Unit (IMU) signal, hence enable dimension measurement feature. Furthermore, Mean Sea Level (MSL) was obtained from existed reference mark, which has been established by National Vertical Reference Network, or a new reference mark.

At the last step, MMS survey's data would processed then stored on the Rural Road Management System (RM), which allows users to access the database via the Web application. Furthermore, RM allows additional modification of the data in digital form, which shows high-quality detailed visual data, and can be used for more liberate application.

Mobile Mapping Applications for Data Collection and Evaluation of the Rural Road Assets

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1 RATIONALE

Conventional surveying method, which requires technician to work on the field, is impractical to apply to the whole rural network since it would consume huge labor force and time. Therefore, mobile mapping technology is seen as the best way to achieve this goal. Mobile mapping system (MMS) is installed on a vehicle, which operates on concrete or asphalt pavements. MMS comprised positioning tool and sensors, hence it can collect data rapidly.

In 2014, DRR had collected road asset data for 2,500 kilometer, from difference districts; NakornNayok, Prachinburi, Sra-kaew, Cha-Seongsao, Nontaburi and Samutprakarn, using MMS.

The MMS system uses 360-omni-video-recording-camera system (Ladybug3) to collect data, which will later be processed for later object-position identification by trained spectators. Road-side assets store in RM divided in to 13 categories. A certain quantity and quality could be evaluated or measured on the 360-video. Furthermore, the coordinate system was tied up to the national vertical reference system (Mean Sea Level: MSL). The mean sea level of road surface is useful for road safety management and natural disaster prevention, especially the flood mitigation.

2 SENSORS INSTALLED IN THE MOBILE MAPPING SYSTEM (MMS)

MMS data collection composed of vehicle, Ladybug3, Computer, GNSS and Inclinator.

Vehicle: The vehicle is a compact passenger car. A rack is installed on the top of the vehicle for fixing a dual antenna. Dual antenna is designed for azimuth determination which is more accurate than 0.01 degree (RMS).



Figure 1. MMS vehicle installed with sensors

LadyBug3: Two panoramic 360 video systems (Ladybug3) are installed on the roof. The first 360 video camera records picture of all assets surrounding. The second 360 video camera records images of the road surface, which shows the texture of the road for later evaluation. Both 360 video camera have resolution up to 5,400 by 2,700 per frame with maximum frame rate of 16 fps.



Figure 2. The panoramic 360 video camera system

Computer: Computer is used for storing information and synchronizing all sensors. According to the DRR standard, the storage needs to accommodate more than 2 Terabyte, which is sufficient for a day of MMS survey or 140 kilometer of survey.



Figure 3. Computer and storage inside MMS car

GNSS and Inclinometer: Both GNSS and IMU are used to record location. Global Positioning Navigation System (GNSS) receives location data from satellites. IMU determines location through the calculation of measured rotation and measured acceleration rate from six directions. GNSS and IMU are used together to increase the accuracy and to eliminate bad signal problems in the urban area and dense-forest area.



Figure 4. GNSS/IMU on top roof of MMS car

Inclinometer: Inclinometer is installed with one of the 360 video camera to optimize vertical visual output, and to determine location of assets.



Figure 5. Inclinometer

3 SURVEYING METHODOLOGY AND RECORDING OF THE PANORAMIC IMAGERY OF ALL ASSETS ON THE RURAL ROADSIDE

1) Once all the equipments are installed, MMS will calibrate itself with its surrounding, in order to determine road assets location. The coordinate of a reference point will be manually measured. Utilizing both reference point location and recorded pictures, location of roadside assets can be.

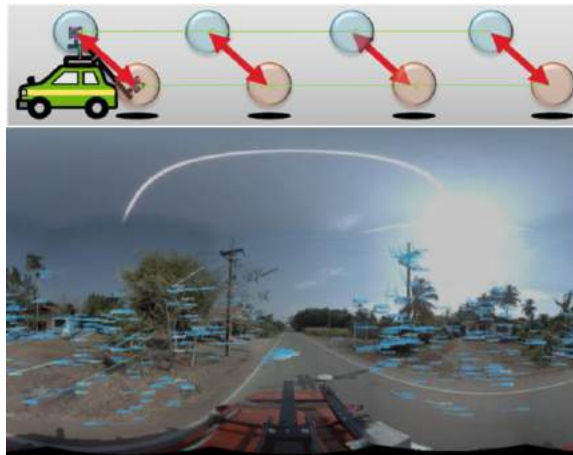


Figure 6. Image Processing within MMS

2) The calibrated data, from comparison of measurement on panoramic camera fused with the measured GNSS/IMU trajectory.



Figure 7. Special apparatus Helico for determination of positioning of the GNSS antenna

3) The optimal speed for panoramic video recording is 60 kilometer per hour. Light condition must be taken care since the system is image-based. Too dark or too bright image will prohibited fine detail of the assets and later coordinate measurement.

4) During MMS survey, positions will be recorded and synchronized with the panoramic video. The post-processing software will integrated both dataset.

5) GNSS base station must be established before the survey. The distance between MMS car and its base station has to be kept within 50 kilometer. This system results in 30-centimeter horizontal accuracy and 1-meter vertical accuracy.

6) The post-processing includes an image processing and post-processed GNSS Trajectory. The resulted video could be used to identify road asset objects and to measure their coordinate based on 3-D stereo measurement principle

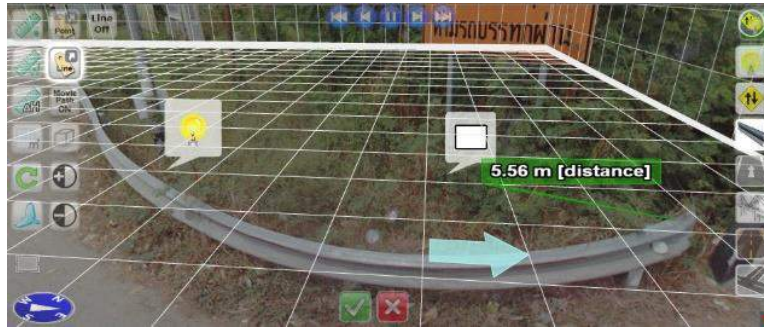


Figure 8. Application for measurement of object coordinate

7) Within the period of DRR MMS pilot study, 2,500 kilometers of rural road network have been registered in the system. The post-process data are prepared and put in to a web-based application named Rural Road Management System (RM).

RM is a software system which combines road inventory and asset management system. RM is a web-based application and allows users to easily access by the internet.

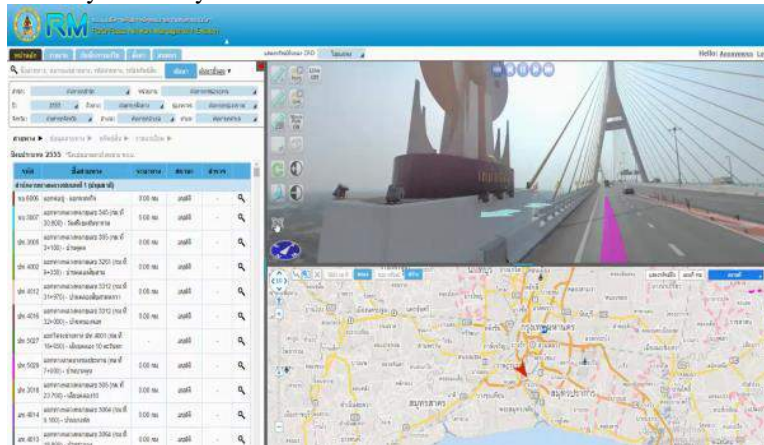


Figure 9. Web-based application RM

4 GNSS SURVEYING FOR HORIZONTAL POSITIONING AND VERTICAL POSITIONING (MEAN SEAL LEVEL: MSL) OF THE ROAD SURFACE

This project measures horizontal position in form of MSL. The satellite surveying resulted from high-end GNSS receiver, Novatel ProPak-6 with a geodetic-grade satellite GPS-702-GG antenna. When antenna must be put on a survey mark, a survey bipod or tripod with 2-meter-fixed height would be used. This 2-meter fixed height antenna will ensure the quality of MSL of the road surface.



Figure 10. Setup GNSS Base-Station

Referring to the local reference survey mark: MMS refers to a local survey benchmark, which provides horizontal and vertical reference; Mean Sea Level (MSL). The nation reference survey mark has been established then maintained by the Royal Thai Survey Department (RTSD). Furthermore, DRR has developed their own survey benchmarks for road construction and maintenance purposes.



Figure 11. Survey measurement on the DRR survey mark

MMS survey over a new establish survey mark: In case reference survey marks do not exist, new temporary survey marks must be established. These new benchmark will be measured by Precise Point Positioning (PPP) method. PPP is a GNSS survey method by receiving data over 24 hour and the data will be computed with precise orbit and clock modelled by an international GNSS service.



Figure 12. measurement over the new survey mark

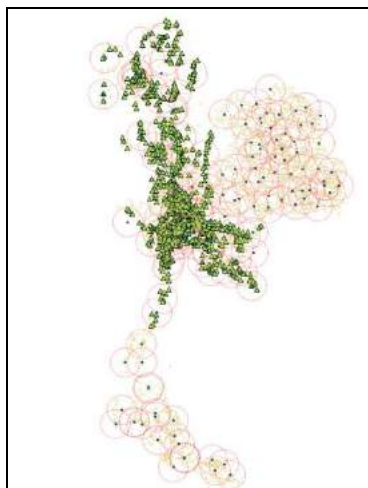


Figure 13. Distribution of DRR survey benchmark in 2016

5 GNSS / IMU PROCESSING

PPP Processing The signal from PPP measurement takes around seven days to ten days to calibrate. The delay is a result from calibration of the Precise Orbit and Clock Correction by International GNSS Service. The processing model, which is called Post-Processed Precise Point Position (PP-PPP), applies precise orbit and clock corrections. These precise orbital and clock parameter can be obtained from the International GNSS Service (IGS). The processing software for GNSS/IMU data is Novatel Inertial Explorer version 8.60

6 CONCLUSIONS

To conclude, by applying mobile mapping system the Department of Rural Road could collect panoramic images of the DRR assets. All panoramic images and trajectory data are processed and imported to the Rural Road Management System (RM). All asset objects can be identified, measured, count and inventoried using the RM.

Extending from pilot project, DRR is now conducting MMS survey for asset management in 47,000 kilometer of its networks. DRR is expecting to complete inventory of DRR assets by 2016.

7 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the support and contributions in preparing this report. This report is an outcome of a collaborative effort between the Department of Rural Road (DRR) and Chulalongkorn University Infrastructure Management Research Unit.

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PAPER TITLE	Road Pavement Design for the Pacific Region – Desk Research on the Use of Locally Available Materials		
TRACK			
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KEYWORDS:

Road Pavement Design Pacific PRIF

ABSTRACT:

The abstract should be written in English, readily understandable to most readers and may contain up to maximum of 500 words. Authors are invited to use Times New Roman Size 10 and the full type width of the page (single column).

Road Pavement Design for the Pacific Region. The study investigates use of locally-available pavement materials to improve pavement quality, resilience and sustainability and to reduce the overall cost of road construction and maintenance. The report by the Pacific Region Infrastructure Facility (PRIF) examines which locally available materials can be used to construct resilient, low volume, roads (both unsealed and sealed). It also sets out factors to ensure success, including ongoing training and support, coupled with use of appropriately-scaled and resourced pavement investigation, design, material supply and processing; sound road construction practices; and knowledge of local materials. However, while the use of more robust, cost-effective and stable materials is important, proper asset maintenance is also needed to minimize whole-of-life costs and ensure sustainability. Ongoing performance reviews will enable local engineers to reasonably evaluate material and design options, develop and implement ongoing country-based training, and support pavement design and maintenance.

2nd IRF Asia Regional Congress & Exhibition

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1. INTRODUCTION

‘Road Pavement Design for the Pacific Region - Desk Research on the Use of Locally Available Materials’.

This report, published in January 2016 by the Pacific Region Infrastructure Facility (PRIF), represents a collaboration of major development partners in the Pacific region – ADB, World Bank, EIB, EU, Australia, New Zealand and Japan¹.

The study was proposed by the PRIF Transport Sector Working Group (TSWG). It is being managed by the PRIF Coordination Office (PCO) and consists of two phases: a research phase and a trial phase (underway). The study is designed to investigate options to engineer the use of locally available pavement materials to improve pavement quality, resilience and sustainability and to reduce the overall whole-of-life cost of road construction and maintenance. The context is the Pacific Island Countries (PICs) in which PRIF operates².

The published report has been developed during the research phase of the study. It brings together contemporary global research about a variety of pavement design and material options available to PRIF agencies working in the Pacific, including the potential use of pavement construction additives that can be reasonably sourced in the region and are appropriate for application in the Pacific context. The use of in-country waste stream materials (e.g. recycled glass or crumbed tyre rubber) have been considered as an additive to support granular stabilisation. In addition, when comparing the whole-of-life costs of various pavement rehabilitation opportunities, a number of issues have been taken into account, including: logistics and likely costs associated with contractor, plant and material establishment future maintenance needs of the rehabilitated pavement, and what equipment is required to support future rehabilitation and maintenance works.

While these latter points are mentioned in general terms in this report, they will also be developed further during the remainder of the study. This may involve Net Present Value analysis, using likely unit rate/quantity construction cost profiles, agreed discount factors and supporting information to enable in-country engineers to make informed decisions.

The research report is released as a knowledge product for engineers and project managers to refer to, as needed, in conducting design and construction work in the Pacific. Further reports will also be prepared during the remainder of the study.

¹ PRIF Members: Asian Development Bank (ADB), Australian Department of Foreign Affairs and Trade (DFAT), European Union and European Investment Bank (EU/EIB), Japan International Cooperation Agency (JICA), New Zealand Ministry of Foreign Affairs and Trade (NZMFAT), and the World Bank Group including the International Finance Corporation (IFC).

² At present these are: Cook Islands, Federated States of Micronesia (FSM), Fiji, Kiribati, Nauru, Niue, Palau, Republic of the Marshall Islands (RMI), Samoa, Solomon Islands, Tonga, Tuvalu, and Vanuatu, with monitoring for Papua New Guinea (PNG) and Timor-Leste.

2. RESEARCH OBJECTIVES

The objective of this study is to investigate options to engineer the use of locally available pavement materials to improve pavement quality, resilience and sustainability and to reduce the overall whole-of-life cost of road construction and maintenance. The report summarises the desk research phase of the study. The report explores the extent to which locally available pavement materials can be used to construct resilient low volume roads. It also sets out factors to ensure success, which include ongoing training and support, coupled with use of appropriately-scaled and resourced pavement investigation, design, material supply and processing; sound road construction practices; and knowledge of local materials and their applicability and limitations.

The locally available pavement materials used in road pavements in the Pacific region are largely sourced from coralline or igneous rocks. Countries where the central volcanic core remains (e.g. Fiji and Papua New Guinea), have access to inland and coastal igneous rock formations, with associated residual or tropical soil intrusions. Inland coronus

reserves, coral and coral sand from the adjoining reef also contribute to the pavement aggregate options. In such countries, which are typically not low-lying, access to fresh water supply also enables more conventional earthworks engineering processes to be used.

3. STRUCTURE OF REPORT

The report addresses four topic areas in regard to locally available material for pavements:

- geological setting in the PICs
- aggregate materials in use in the Pacific
- specifications and standards in use
- aggregate stabilisation

Each topic area is discussed in separate sections of the report. The material is broad-ranging in parts, whilst remaining ‘on topic’ technically. At the end of each section, a conclusion considers how the findings could be relevant to the current and future use of locally available aggregate materials. A full list of references is included in the report.

4. BACKGROUND

Smaller low-lying island countries e.g. Kiribati and the Republic of the Marshall Islands often retain only the coastal coral reserves and, given concerns about environmental sustainability, probably only the coralline sand (cascajo) deposits inside the reef itself. Here access to fresh water is limited, so construction with salt water is more likely. Experience dating from World War II has shown that pavement construction using coralline materials with salt water can provide some benefits, assisting with self-cementation of the compacted coral aggregate. Investigations of failing airfield pavements sections have shown that the failure can be confined to the more recent overlying asphalt bound surfacing, and not the underlying, more resilient coral-based pavement layers.

A description of how geography and geology affects pavement material supply for each of the countries is summarised here in Table 1.

Table 1. Expected Local Aggregate Options in Pacific Islands

Country	Coral Aggregate	Coronus Aggregate	Volcanic Aggregate
Cook Islands	✓	✓	✓
FSM	✓	✓	✓
Fiji Islands	✓	✓	✓
Kiribati	✓	✗	✗
Nauru	✓	✓	✗
Niue	✓	✓	✓
Palau	✓	✗	✓
PNG	✓	✓	✓
RMI	✓	✗	✗
Samoa	✓	✓	✓
Solomon Islands	✓	✓	✓
Timor-Leste	✓	✗	✓
Tonga	✓	✓	✓
Tuvalu	✓	✗	✗
Vanuatu	✓	✓	✓

Source: Brij, V.L., & Fortune, K. (Eds). (2000).

The pavement design and construction principles used around the Pacific study area vary, and appear from anecdotal evidence to be largely based on recognised, published empirical methods including some local country-based variations. To be successful, whichever design and construction method is used, the pavement designer needs access to relevant and accurate information about future traffic loading, in-situ foundation conditions and material characterisations, and environmental constraints.

Questions about pavement project reliability should be raised first with the pavement owner, because reliability, cost and the importance of the project in the wider context (e.g. small local road versus main highway) sometimes need to be traded off to achieve the best for project outcomes.

5. DESIGN AND CONSTRUCTION PRINCIPLES

Pavement aggregate stabilisation is a proven technique around the Pacific region. The improvements in pavement layer stability, strength and load bearing capacity can be utilised from the subgrade up, throughout the pavement structure.

On low-lying coral atolls, cement, foamed or emulsified bitumen or cement/polymer stabilization would appear better suited to the sand grained, unprocessed and compacted coral reef aggregate materials. Self-cementation in constructed coral aggregate roads is a recognised, beneficial outcome, with proven and tangible pavement performance benefits in low volume sealed and unsealed roads.

For road aggregates prepared from volcanic source rock, or weathered inland coronus deposits, lime and cement stabilisation would help to mitigate the adverse effects of high plasticity, water sensitivity, and high natural water contents.

Generally aggregate stabilisation with cement and/or foamed bitumen can be used with both volcanic and coronus materials to successfully mitigate the adverse effects of plasticity and enhance layer strength and load bearing capacity, provided that when a bound or lightly bound layer is prepared that the potential adverse effects of cracking are considered and proactively managed.

Pavement aggregate materials processed using local coralline or volcanic sources and added to using recycled material from other waste streams (e.g. recycled crushed glass, tyres or plastic for example), with or without existing or new polymer or other additives, presents an opportunity. The economics of using waste material need to be carefully considered, taking into consideration the whole-of-life costs of sourcing aggregate, especially the cost of collection, storage processing and transport.

Small island states will probably not generate enough waste glass, used tyres and plastics on their own, except perhaps in one-off project situations. This raises the opportunity perhaps for wider regional collaboration.

6. KEY FINDINGS

All the options described above are technically feasible in the Pacific region. However, investigations in this study indicate that, whilst larger one-off capital projects may justify the establishment of external plant, resources and even materials, what is needed at a country level is appropriately trained and resourced local contractors, staffed by people who understand what can be achieved with local materials, with or without modification. With this knowledge, local contractors may be more credible and competitive when bidding for pavement construction and rehabilitation works across the region, thereby improving local productivity and, ultimately, supporting poverty reduction.

The study focuses on effective use of local materials and resources for improvement of low volume roads (both unsealed and sealed). However, it is stressed that while the use of more robust, cost effective and stable materials is important, proper asset maintenance is also needed to minimize whole-of-life costs and ensure sustainability.

The use of proven modifiers/stabilisers (lime/cement) with existing coral or volcanic source aggregates, and suitable in-country waste stream materials, by means of simple, repeatable production and construction techniques, appears to offer exciting opportunities, provided that the wider economics associated with collection, processing and utilisation of locally available aggregate materials incorporating such materials are understood. This would include understanding of the fixed and/or mobile plant needs to process aggregate materials, and the longer term plant and pavement maintenance expectations.

The ideas presented in the published report require corroboration from the secondary phase in-field trials and performance monitoring to determine the long-term sustainability of the pavements. The resulting information database containing relevant feedback on processes and costs that can be linked to ongoing project performance reviews will enable local engineers to reasonably evaluate material and design options, and to develop and implement ongoing country-based training and support for pavement design and maintenance.

7. APPLICATION OF REPORT FINDINGS

Given the nature of the overall study, it is not expected that all the issues will be applicable to future use of locally available aggregate materials in the Pacific. For example, when discussing pavement design standards (Section 4), applicable engineering test methods that could be used to define pavement material properties may not all be suitable for use in the Pacific region under current conditions. Also, the scale of plant and materials needed to undertake large-scale pavement aggregate stabilisation projects in an established PIC with access to a range of aggregate and recycling

sources may not be applicable to a smaller, atoll-based island state, or for use on low volume roads. Where possible, suitable applications and solutions are identified that are feasible for a range of countries.

7. CONCLUSIONS

The desk-based research undertaken for this study shows that it is possible to use locally available coral and igneous rock-based pavement materials to improve road pavement quality, resilience and sustainability and to reduce the overall whole-of-life cost of low volume road construction and maintenance in the Pacific region.

The science and technology exists now to source, extract, process and use local pavement materials effectively at whatever scale is necessary to best suit the local conditions. The properties of the local pavement materials can be improved by processing and stabilisation using a variety of stabilization agents to suit the needs of unsealed and sealed pavements in the region.

The challenges to achieving the above are real and immediate:

- climate change-induced sea level rise and damage caused by major storm events
- poor regulation and control of overweight commercial and industrial traffic
- insufficient and poorly informed pavement investigation and design
- lack of maintenance of road pavement and drainage infrastructure
- lack of investment in any pavement resurfacing.

All of these contribute to the ‘build-neglect-rebuild’ paradigm evident on roads across the Pacific region. As part of this current study, field trials are underway that will help determine the long-term sustainability and performance of road pavements built and maintained using locally available materials to meet current and future traffic demands. The manner in which the trials are developed and implemented will encourage involvement by and training of local engineers and contractors (as needed), using appropriately-scaled labour-based and equipment supported methods.

8. ACKNOWLEDGEMENTS

The report was researched and drafted by a consultant, William Gray, working under the supervision of a committee of experts drawn from the main PRIF agencies and other regional organisations, including Dr. David Aitchison (University of the South Pacific), Dr. Theuns Henning (Auckland University), Peter Kelly (Australian Department of Foreign Affairs and Trade) and Megan Schlotjes (World Bank).

9. REFERENCES

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The published report includes a full list of references.

Investigating the Characteristics of Drivers with Multiple Crashes

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ABSTRACT

Identifying the drivers with higher traffic crashes involvement frequency (i.e. high-risk drivers) is the main concern of all road safety related entities. Driver's behavior factors are considered as a leading cause of traffic crashes. Driver's behavior can be measure based on the historical records of their violations and crashes. This paper aims to investigate the characteristics of drivers who are frequently involved in severe crashes and to define the parameters that can be used to recognize these drivers. About 20 types of unsafe traffic violations are intensively investigated. The interrelationships between the at-fault drivers' involved in traffic crashes and their demographic characteristics, historical traffic violations, historical severe and property damage only (PDO) crashes involvements and historical violations of different types were explored.

Negative Binomial Regression modeling approach is applied to define the associated variables that can be used to predict the driver's severe crashes future involvements. The results show that females, young, local, and less driving experience drivers have higher risk to be involved in future severe crashes. In addition, the following violations can be used as predictors to define drivers with multiple crashes: exceeding speed limit by more than 50 kph, car racing involvement, alcohol use, mobile use, tailgating, entering road suddenly, not using helmet, and overtaking-related violations. The findings can also be used to develop or improve the preventative strategies against high risky drivers.

Key words: *Risky driving behavior, multi-crashes drivers, traffic violations, crash rate estimation. Crash-prone drivers*

INTROUDUCTION

One of the primary missions of the traffic police in most of the world-wide countries is to enforce the drivers with aggressive attitudes and high crash risks. Each driver has a degree of risk to be involved in a crash given some factors related to the environment, human, and road. Many previous studies proved that the majority of the traffic crashes occurred due to human factors (Karscasu and Er, 2011). This conclusion produces a challenge to the transportation planners and decision makers to develop traffic safety strategies due to the un predictable factors of crash occurrence.

In Abu Dhabi (AD), the capital of United Arab Emirates (UAE), the driver's community is a mix of different nationality groups from all over the world where the total number of registered non local (i.e. non-Emirati) driver groups is more than 200. Referring to the local traffic safety statistics in Abu Dhabi regarding the at-fault drivers who were involved in severe crashes (i.e. a crash with at least one injury) during the period between 2010 and 2015, about 88.5% of them are male, 40.3% aged between 18 and 30 years old, and about 35% are from Asian countries. These facts produce many challenges to Abu Dhabi government to control different backgrounds, cultures, and driving experiences of multi-national driver society. It is worth mentioning that Abu Dhabi government has a strategic target to reduce the road fatality rates from the current value of about 7.4 fatalities per 100,000 inhabitants to 3.0 fatalities per 100,000 inhabitants by year 2023. Table 1 shows the main road safety indicators in Abu Dhabi during the period from 2010 to 2015. The numbers indicate that a significant improvement in road safety has been achieved during last six years. The number of fatalities has been reduced by 34.8% and the fatality rate has been reduced by 39.8% at the end of 2015 compared to the same period in 2010.

Table 1: Traffic Safety main indicators in Abu Dhabi

<i>Year</i>	<i>Number of severe crashes</i>	<i>Number of causalities</i>	<i>Number of serious injuries</i>	<i>Number of fatalities</i>	<i>Fatality rate per 100,000 inhabitants</i>	<i>Fatality rate per 10,000 registered vehicles</i>
2010	2,537	4,307	400	376	12.3	5.4
2011	2,283	3,871	390	334	10.7	4.3
2012	2,056	3,498	364	271	8.7	3.3
2013	2,071	3,644	366	289	8.9	3.3
2014	1,861	3,154	240	267	7.6	2.8
2015	1,803	3,025	1478	245	7.4	2.4

One of the promising tools to achieve the above mentioned strategic target is to identify the high-risk drivers based on traffic safety related criteria as to be demonstrated in this study. After that, preventative programs and strategies to enforce and aware such driver group can be developed including some penalties such as license suspension, attending special road safety training courses, warning messages, etc. It is worth mentioning that no clear definition is yet published regarding "high-risk" drivers in Abu Dhabi which leaves a space for this study to set criteria in order to propose such definition.

Table 2 shows statistics of the traffic violations in Abu Dhabi during the period from 2010 to 2015. The table shows that the total number of violations has been increased by 130% in year

2015 compared to year 2010. This increasing trend can be justified due to the increase number of the installed automated speed enforcement devices in the last few years. The number of speed cameras has been increased from 414 in 2010 to 704 in 2015 (about 70.0% increment). In addition, about 198 red light violation cameras have been installed at 77 intersections during the last three years. Also, Table 2 shows that at-site enforcement (i.e., face to face traffic violation tickets) has been increased by 37% in year 2015 compared to year 2010.

Table 2: Traffic violations statistics in Abu Dhabi

<i>Year</i>	<i>Total number of violations</i>	<i>At-site violations</i>	<i>Speed-related violations</i>	<i>% of speed-related violations</i>	<i>Violation rate per registered vehicle</i>
2010	2,477,340	504,559	1,851,907	74.8%	3.5
2011	4,011,310	615,826	3,352,570	83.6%	5.1
2012	4,085,074	641,314	3,399,460	83.2%	4.9
2013	3,832,685	745,159	3,197,320	80.3%	4.4
2014	4,338,701	719,156	3,477,274	80.2%	4.6
2015	5,692,745	688,845	4,921,934	86.5%	6.0
% of change	130%	37%	165%	-	69%

On the light of previous statistical facts, the following question was raised: Was increasing the number of automated speed enforcement cameras on roads in the past years is the right approach to improve road safety levels in Abu Dhabi? In other words, Is there any relationship between the “high-risk” drivers and traffic rule violations record? This paper tries to find an answer to the mentioned question by exploring the “high-risk” drivers in terms of their historical traffic safety records (i.e. violations and crashes) in addition to their demographic characteristics. Thus, this study focuses on predicting the likelihood that a “high-risk” driver will be involved in at-fault recurrent crashes in the future using driver records from the period between 2008 and 2015 (i.e., eight years of data) that have been reported by Abu Dhabi traffic police.

LITERATURE REVIEW

Many researchers have attempted to determine which variables are the best predictors for traffic crashes. The examined variables in prior studies can be classified as: demographic characteristics of drivers, psychosocial (e.g., aggressiveness, impulsiveness), behavioral (e.g., crash involvements history, traffic rules violations). Early study by Peck at al. (1971) concluded that the statistical nature of driver crash frequencies makes it impossible to accurately predict which individuals will or not will be involved in at-fault crashes in the future. However, many later studies found a statistically significant relationship between the number of crashes involvements and the historical number of traffic violations of the driver (Gebers and Peck, 2003; Blascoa, 2003; Chandraratna et al., 2006). The substantial body of these studies focused on establishing estimations of the potential of a driver to be involved in a crash based on prior driving records (e.g., crashes and violations).

Lui and Marchbanks (1990) determined a relationship between previous traffic infractions (i.e. violations) and fatal car crashes by studying the fatal accident reporting system from

1984 to 1986 period. They suggested that the involvement in a fatal crash is not a random event and showed that the mean time between a previous traffic infraction and a fatal crash was the shortest for individuals aged 16 to 25 years, who had a mean recurrent time of 14.2 months. In addition, about 97% of the recurrent times occurred within 60 months of a given traffic infraction, with the highest risk of a fatal crash from three to seven months following the infraction.

Hauer et al. (1991) examined several methods to identify drivers who are most likely to have a crash in the near future using a four-year record for a large sample of Ontario drivers. About 16 different prediction models were developed and compared. It was found that the model that made use of detailed information on age, gender, number of each of 14 types of violations, and number of at-fault crashes and not-at-fault crashes, was the most efficient model in terms of explaining the variance of estimated crash potential. The authors concluded that if the prediction model makes use of the driver's prior crash records, the performance of the prediction model is notably improved.

Chen et al. (1995) applied logistic regression approach to identify the drivers who were most likely to have one or more at-fault crash involvements, based on prior records of at-fault crash involvements drivers. The results indicated that a model that makes use of prior at-fault crash information can identify up to 23% more drivers who will have one or more at-fault crash involvements in the next two years than a model that uses violation records alone. After studying 17 logistic regression models, Gebers (1999) concluded the final developed model could correctly classify crash-involved drivers up to 27.6%. In addition, the model indicated that age, gender, license class, total citations and total crashes are the greatest prediction variables of crash-prone drivers.

Alver et al. (2004) explored the interaction between socio-demographic characteristics of traffic rule violators (four types of traffic violations records were aggregated) and crash history for young drivers (18-29 years old) by applying binary logit models. The results showed that 23.9% of drivers were involved in at least one traffic accident in last three years. The crash rate increases to 38.3% for drivers who received at least one traffic violation ticket in last three years and peaks to 47.4% for those who were fined for seat belt violations.

Chandraratna et al. (2006) studied Kentucky drivers to develop a crash prediction model that can be used to estimate the likelihood of a driver being at fault for a near future crash occurrence by using multiple logistic regression technique. The authors dedicated that the developed model can be used to correctly classify at-fault drivers up to 74.56% with an overall efficiency of 63.34%. The total number of previous at-fault crash involvements, and having previous driver license suspensions and traffic school referrals are strongly associated with a driver being responsible for a subsequent crash. In addition, a driver's likelihood to be at fault in a crash is higher for very young or very old, males, drivers with both speeding and non-speeding citations, and drivers that had a recent crash involvement.

Zhang et al. (2013) analyzed crash severity and violation data of Chinese drivers. The results established the role of traffic violations as one of the major risks threatening road safety. In addition, specific risk factors associated with traffic violations and accident severity was determined. The authors suggested that to reduce traffic crashes and fatality rates, measures such as traffic regulations targeting different vehicle types/driver groups with respect to the

various human, vehicle and environment risk factors are needed. Some studies proved the significant effect of specific drivers' behavior such as speeding and drunk and drug usage on the prediction of the crash-prone driver (Winter and Dodou, 2010; Watson et al., 2015; Kim, 2015). Dissanayake and Lu (2002) and Baker et al. (1992) proved that the seat belt usage and alcohol usage have major effects on the crash severity. In addition, Tseng (2013) showed that both younger and older drivers have relatively higher speeding risk.

Berdoulat et al., (2013), investigated the aggressiveness and impulsiveness in the prediction of risky drivers. It was found that aggressiveness and impeded progress were the best predictors of violations and aggressive violations. The results supported that transgressive driving behaviors are relevant indicators of aggressive driving. The same result was concluded by Bachoo et al. (2013) from a sample of post graduate students in Durban, South Africa.

A new definition of the impulsivity in driving context was suggested by Bıçaksız and Ozkan (2015) during analyzing 288 student self-reported questionnaires in driving behavior, violations and accident involvements. Machado-Leon et al. (2016) investigated crash risk perceptions in an inter-city, two-way road context of 492 drivers by using a Stated Preference ranking survey. The study that all risky driving behaviors showed a significant potential effect on crash risk perceptions, and model's results allowed to differentiate more important from less important unsafe driving behaviors based on their weight on perceived crash risk

DATA PREPARATION AND ANALYSIS METHODOLOGY

Database Preparation and Sample Size Selection

The employed data in this study were extracted from four different datasets of Abu-Dhabi traffic police during the period from 2008 to 2015. These datasets are: 1) traffic violation system, 2) property damage only (PDO) crashes system, and 3) severe crashes system, and 4) licensed drivers' system. Individual drivers' data were integrated in one comprehensive database by using the driver's unique traffic code that is given to each licensed driver at the day when he/she issued a driving license.

The total number of registered driving licenses in year 2008 was 636,907 and increased to 1,234,009 licenses by the end of year 2015. Serial processes of data filtrations were carried out to select the sample of drivers that will be used in the analysis. First, records of drivers who have private driving licenses only were used in the analysis (i.e., excluding the driving licenses of companies, governmental, diplomatic, intuitions, etc.). Second, drivers with zero records of both violations and crashes during the eight years were excluded from the analysis as well as those who do not have fully populated demographic information in the database. Finally, records of drivers who issued their driving licenses after year 2008 were not considered here to assure that all drivers in the data sample have practiced driving during the analysis period.

Accordingly, the data sample was reduced to be 324,644 drivers. The drivers existed in the data sample recorded a total number of 4,116,149 traffic violations; 578,619 property damage only (PDO) crashes (i.e. any crash without any injury or fatality); and 7,676 severe crashes (i.e., any crash with at least one injury or fatality). The database is structured to show the

detailed historical records of each individual driver as well as groups of selected traffic violation types that have direct or indirect impact on road safety, at-fault crash involvements and demographic characteristics. These violations have been clustered into 20 groups of violations by aggregating some types in one group, for instance, the (overtaking-related violations) group which includes four types of violation: overtaking from the right, overtaking in prohibited locations, overtaking from shoulder, dangerous overtaking behavior.

Data Analysis Methodology

To achieve the objectives of this paper, the analysis process was conducted in two stages:

- 1) The first stage aims to develop the relationships between the severe crash rates per drivers in terms of violations types, drivers’ demographic characteristics and the frequency of the violation types and crashes per drivers. In this stage the violations type and drivers’ characteristics that have high severe crash rates can be recognized.
- 2) The second stage provides an estimation model to determine the best predictors/variables that can be used to identify “high-risk” drivers (i.e., the drivers have a highest likelihood to be involved in crashes in the future) using negative binomial regression model.

DATA ANALYSIS

Interrelationship between Crash Rates and Violation Records of Drivers

Unlike traffic crashes, not all traffic violation types are indicative of risky driver’s behavior or are predictors to future crashes such as: “illegal parking, driving with expired license, etc.” Therefore, in the primary analysis of this study, the driver’s violations were classified into two classes. Class (I): speed-related violations which includes six types of violations. Class (II): risky behavioral violations which includes 14 types of unsafe behavior violations such as: tailgating, mobile use when driving, non-seatbelt use, sudden lane changing, etc. Accordingly, drivers were classified into different groups based on their historical records of violation’s type.

Crash rates and percentage of drivers involved in crashes of each group of drivers as shown in Table 3. It shows that drivers of class II have higher rates of severe crashes than those in class I. However, class I drivers are more likely to be involved in property damage only (PDO) crashes than class II drivers. In addition, Table 3 shows that the overall severe crash rates of the studied sample of drivers is 26.1 crashes per 1,000 drivers during the data period.

Table 3: Crash rates and percentage of divers involved in crashes for drivers group during eight years (2008-2015)

<i>Drivers' Group</i>	<i>No. of severe crashes</i>	<i>No. of PDO crashes</i>	<i>No. of drivers</i>	<i>Drivers involved in severe crash</i>	<i>Drivers involved in PDO crash</i>	<i>Severe crash rate per 1000 driver</i>	<i>PDO crash rate per driver</i>	<i>% of drivers involved in severe crash</i>	<i>% of drivers involved in PDO crash</i>
All drivers	16,326	965,341	624,422	14,924	425,806	26.1	1.5	2.4%	68.2%
Class I	7,376	253,604	312,496	6,883	223,603	23.6	0.8	2.2%	71.6%

Figure 1 shows the calculated severe crash rates of drivers who belong to Class I (speeding-related violations group). This figure clearly proves the strong relationship between the speeding behavior of drivers and their crash rates as a representative of their traffic safety. The severe crash rates of the drivers are significantly increasing with higher over-speeding values exceeding the legal posted speed limit on the road. For instance, drivers who exceeded the speed limit up to 30 kph have a crash rate of 23 crashes per 1,000 drivers during the study period, however drivers who exceeded the speed limit by 60 kph have crash rates of 46 crashes per 1,000 drivers (i.e. almost double) and drivers' group of those involved in car racing on roads that have crash rates of 66 crashes per 1,000 drivers (i.e. 2.87 times more than the others).

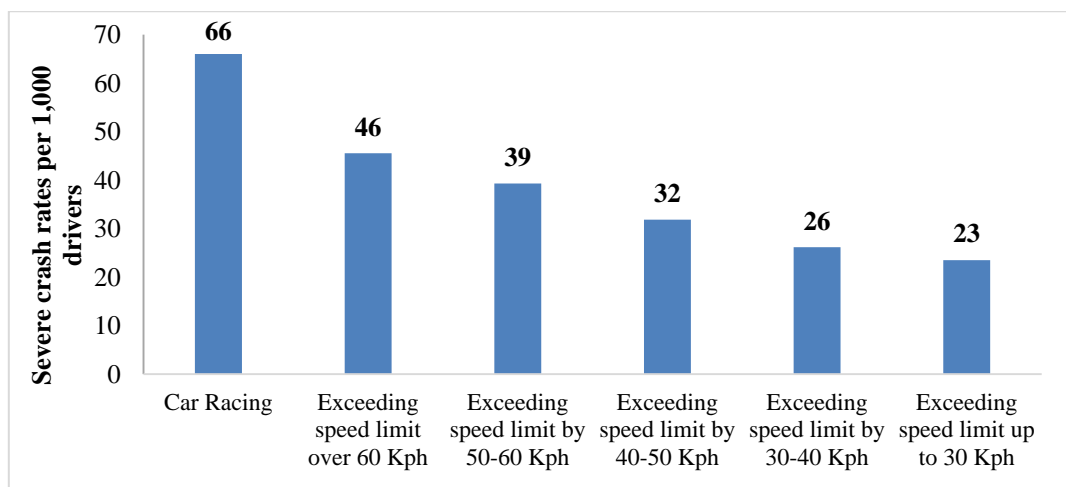


Figure 1: Severe crash rates for Class I drivers group

Figure 2 shows the calculated severe crash rates of drivers who belong to Class II violations group. It shows that alcohol usage drivers have the highest crash rates (i.e. about 4.8 times more than the average rate of all drivers shown in Table 3). On the other hand, crash rates of drivers using mobile phones while driving has a relative lower crash rate. This result is not an accurate representation of its severity level; this is because such violations are difficult to be cited by the traffic police specially on the highways where most of sever crashes occur which leave a relative lower citation records in the violation database that is reflected in lower crash rate.

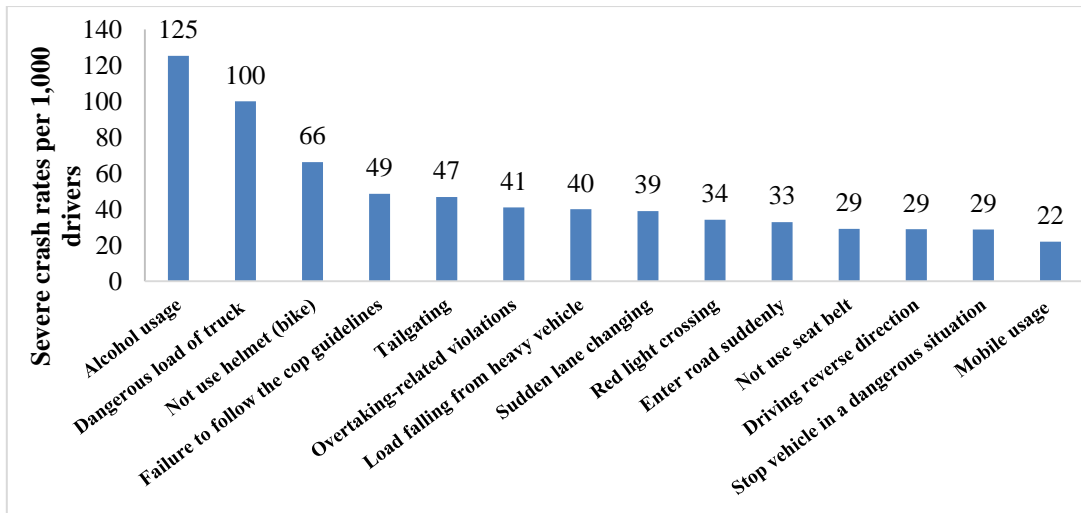
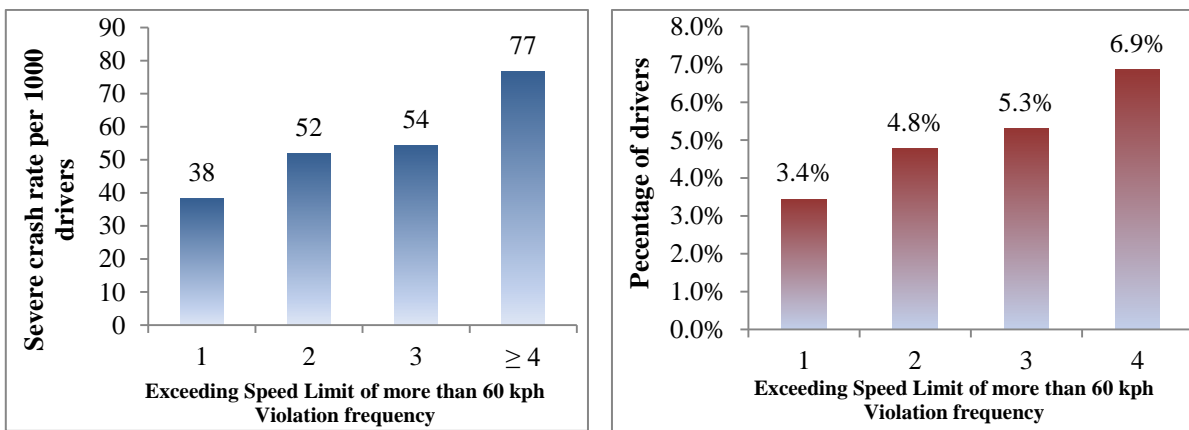


Figure 2: Severe crash rates for Class II drivers group

Frequency of Violations Impact and Crashes per Driver

The frequency of the violation (i.e., number of repeating the same violation type) by the individual drivers was investigated for all type of violations during the study period. Due to the size limitation of this paper, an example of these analyses are presented. Figures 3-a and 3-b show an example of the analysis outcomes for the violation titled “Exceeding the speed limit of more than 60 kph”. These figures show a significant increase of crash rates and percentages of drivers involved in severe crashes by increasing the frequency of the violation numbers. Drivers who get four or more violation tickets of this violation have about double crash rates compared to drivers who get only one ticket.



(a) With severe crash rate

(b) With percentage of drivers

Figure 3: Crash rates and percentage of drivers with respect to the frequency of the violation (Violation example: Exceeding speed limit by more than 60 kph)

Figure 4 shows the crash rates of the driver’s group based on their records number of all types of violations. in general, this figure shows that the crash rate of drivers increases with increasing the total number of traffic violations of individual drivers.

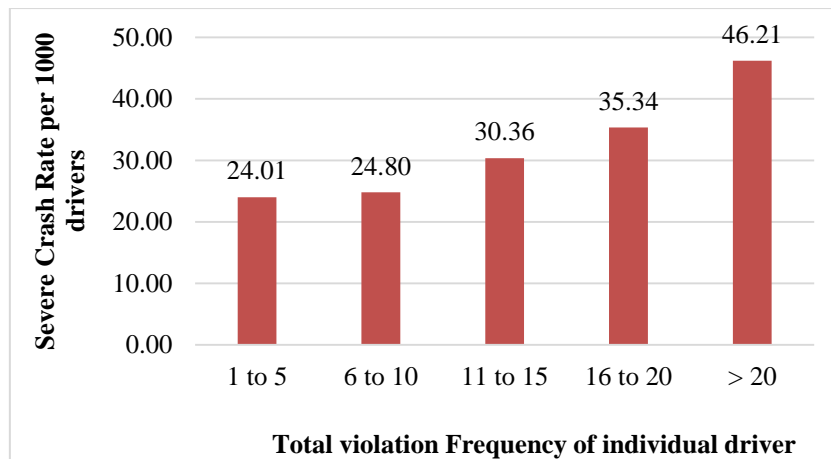


Figure 4: crash rates based on total number of violations of the driver

Figure 5 shows the relationship between the frequency of severe crashes of individual drivers and PDO crashes records. This figure shows that drivers have more than one severe crashes were involved in higher PDO crash rates which means the PDO crashes records of the drivers can be used as indicator for the potential of the driver to be involved in severe crashes.

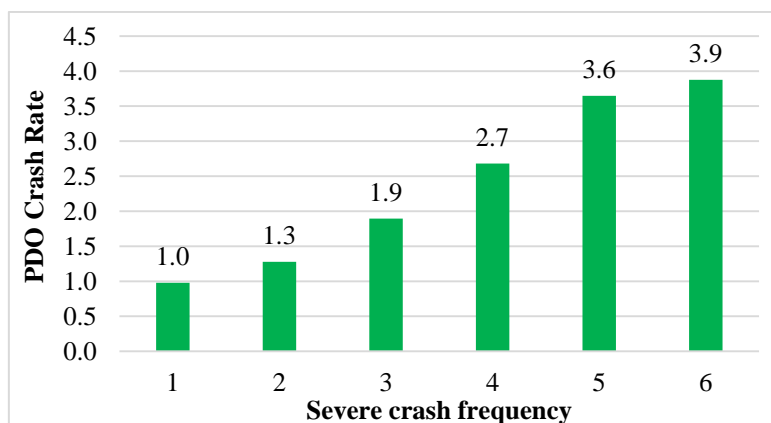


Figure 5: Frequency of severe crashes of drivers and PDO crashes records

Drivers' Demographic Characteristics Impact

The calculated crash rate for male drivers is almost very close to that for female (about 26.2 severe crashes per 1,000 drivers). Considering the driver's age group, Figure 6 shows the crash rates of different age groups of Abu Dhabi registered drivers. It shows higher crash rates for very young and very old drivers. This conclusion is supported with the driving experience of the driver where the mid aged drivers (i.e. with higher driving experience) have relatively lower crash rates since they became familiar with the road circumstances and hence drive more safe. However, the older drivers (i.e. drivers with age group of 65 years old and more) lose their perception/ reaction characteristics which make them more involved in sudden crashes.

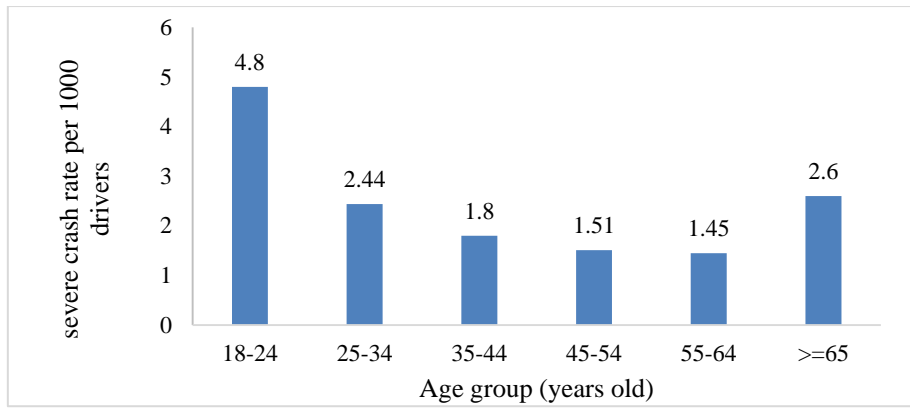
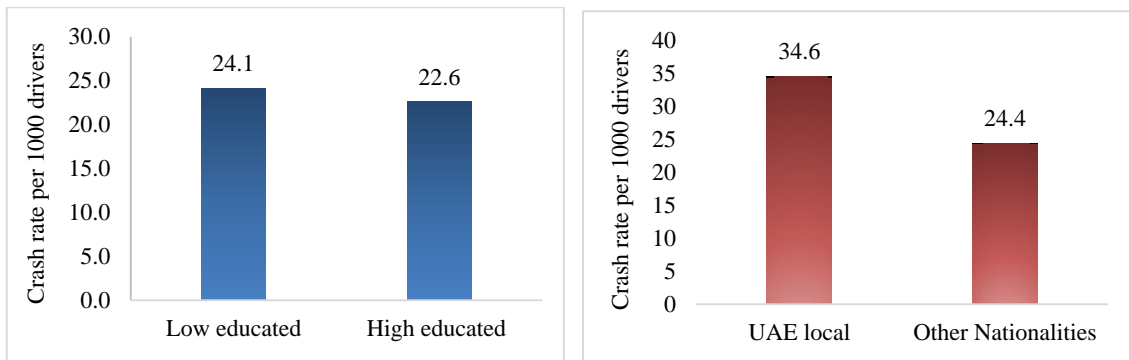


Figure 6: Crash rate of the driver’s age group

Educational attainment of the drivers showed that the low educated drivers have higher crash rate (24.1 crashes per 1,000 drivers) of severe crashes than for the high educated ones (22.6 crashes per 1,000 drivers) as shown in Figure 7-a. In addition, local drivers showed higher crash rate (34.6 crashes per 1,000 drivers) relative to other nationalities in residing in Abu Dhabi as shown in Figure 7-b.



(a) Education level

(b) Nationality

Figure 7: Crash rate of the driver’s based on education level and nationality

Regression Model Estimation

In this section, a regression model was developed to determine the best predictor variables that can be used to identify “high-risk” drivers in Abu Dhabi. Based on Highway Safety Manual (HSM-2010) the negative binomial regression modelling is considered as the best modeling approach to estimate the predictors variables of crash involvements. There for this model were applied in this study. The negative binomial regression model form to predict the total crash frequency is shown in Equation 1:

$$\ln Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \dots + \beta_n X_n \dots \dots \dots (1)$$

Where, Y is the dependent variable (i.e. number of at-fault severe crashes in our case); X_1, X_2, \dots, X_n are the predictor variables; and $\beta_0, \beta_1, \beta_2, \dots, \beta_n$ are the regression coefficients.

The SPSS statistical software package was employed to estimate the model using the customized negative binomial with log link function with the option to estimate the

dispersion parameter rather than setting it to the system’s default value. It accounts for the over-dispersion that is found in the crash data and quantifies an over-dispersion parameter.

Table 4: Model combinations with the investigated predictor variable groups

Model	Group variables 1 Demographic (age, gender, nationality, education, years of experience)	Group variables 2 Speeding Violations (6 types)	Group variables 3 Behavior Violations (14 types)	PDO Crashes
M1	√			
M2	√			√
M3		√		
M4		√		√
M5			√	
M6			√	√
M7	√	√	√	
M8	√	√	√	√

Eight models with different combinations of the examined variables (predictors) were developed. Table 4 summarizes these eight models and the associate examined variables for each model. For example, it shows that the models M1, M3, and M5 used the variables groups of speeding violations, historical behavior violations, drivers’ demographic characteristics, respectively. By accompanied the PDO historical records of the driver to these three groups of variables the models M2, M4 were developed.

Akaike Information Criterion (AIC) value each model which is a measure of the relative quality of statistical models are used to select the best fit model. The AIC compares different models from the same data by adjusting the -2 Log Likelihood statistic for the number of terms in the model and for the number of observations in the sample and can be calculated as follows (Gerbers 1999).

$$AIC = -2 \ln (L) + 2(K + S) \dots \dots \dots (2)$$

Where, L = Likelihood

K = the number of ordered values for the response variable

S = number of independent variables or covariates

Based on AIC values the best model is the model M7 which used all variables of the three groups of violation records of the drivers. One possible justification of that the PDO records of the drivers not shown in the best model is that in AD any very minor crash is consider as PDO and that represent a significant number in AD. Table 5 shows the final significant estimate parameters of the best developed model. As shown in Table 5, all the response variables are statistically significant at 95% (p-value ≤ 0.05).

Table 5: Results of negative binomial regression model

Dependent variable	Regression Coefficient	Standard error	Wald χ^2	P-value	Odds ratio	Odds ratio 95% confidence limits	
						Lower	Upper

Dependent variable		Regression Coefficient	Standard error	Wald χ^2	P-value	Odds ratio	Odds ratio 95% confidence limits	
Intercept		-2.917	0.1030	802.672	0.000	0.054	0.044	0.066
drivers' variables	Gender	-0.290	0.0639	20.672	0.000	0.748	0.660	0.848
	Nationality	0.395	0.0513	59.232	0.000	1.484	1.342	1.641
	Age	-0.023	0.0035	41.171	0.000	0.978	0.971	0.984
	Experience	-0.024	0.0046	27.699	0.000	0.976	0.967	0.985
Violation types	Exceed peed between 50-60kph	0.030	0.0149	4.163	0.041	1.031	1.001	1.061
	Exceed speed more than 60kph	0.027	0.0134	4.070	0.044	1.027	1.001	1.055
	Reckless and running	0.124	0.0269	21.370	0.000	1.132	1.074	1.194
	Mobile use	-0.089	0.0332	7.130	0.008	0.915	0.857	0.977
	Alcohol use	1.253	0.0961	170.023	0.000	3.499	2.899	4.224
	Tailgating	0.229	0.0480	22.750	0.000	1.257	1.145	1.382
	Enter road suddenly	0.309	0.0823	14.066	.000	1.362	1.159	1.600
	Not using Helmet	0.369	0.1342	7.554	0.006	1.446	1.112	1.881
	Overtaking-related	0.094	0.0260	13.047	0.000	1.098	1.044	1.156

Regarding the drivers' demographic characteristics group, gender; nationality; age; and experience were found to be as significant predictors for estimate crash-prone drivers. Females, young, local, and low number of years' in driving experience drivers have high risk to be involved in severe crashes in the future. From the speeding violations group, exceeding speed by more than 60 kph, exceeding speed by values 50-60 kph, and reckless and running violations can be used to define the risky drivers. In addition, from the behavior violation group, the following violations were categorized to identify the risky driving behavior; alcohol use, mobile use, tailgating, entering road suddenly, not using helmet and overtaking-related violations. Accordingly, the accompanied fines and enforcement efforts should be concentrated in such violations in the future.

CONCLUSION

This paper investigated the variables that predict severe crash involvement of drivers with multiple crashes in Abu Dhabi. The study conducted crash rate and frequency analyses for about 20 different drivers' groups of violations and drivers' demographic characteristics. In addition, Negative Binomial Regression modeling approach is used to define the associated variables that can be used to predict the number of severe crashes involvements. The data analysis showed strong relationships between the severe crash rates of the drivers with their historical records of POD crashes and violations. In addition, females, young, local and low number of years' in driving experience drivers have high risk to be involved in severe crashes. The model results showed that 9 violations' types are the best predictors for driver's crash involvements. these violations are: exceeding speed by more than 60 kph, exceeding speed by values 50-60 kph, reckless and running, alcohol use, mobile use, tailgating, entering road suddenly, not using helmet and overtaking-related violations.

Identifying the characteristics of drivers at increased risk of future multiple crash involvement would improve the road safety level by anticipating countermeasure actions against these drivers. To this end, massive enforcement and traffic awareness campaigns have to be designed based to improve the traffic safety levels and deter the risky drivers from anticipating in future crashes or violations.

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PAPER TITLE	Investigating the Impact of Various Climate Change Scenarios on Pavement Service Life		
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ABSTRACT:

Climate change has a particular effect on pavement performance. Traffic and environmental loading will cause the pavement to deteriorate over time. The rate of pavement deterioration is then used for future maintenance and management. Modern pavement design methods such as the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1993) and AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) are based on preassumed vehicular loading and historical climate conditions. In this study, the impact of climate change on the pavement service life and life cycle cost analysis (LCCA) was investigated in the northern and southern regions of Taiwan by using historical climate and ground water depth data. Two of the pavement performance models used by the AASHTO (1993) and AASHTO ME were used to predict the service life of a typical pavement profile of provincial roads under various future climate change scenarios. The results showed that pavement's unbound mechanical material properties, structural capacity, and service life are affected significantly by climate change relative to a baseline profile. The pavement LCC analysis also increases significantly because of the climate change scenario relative to the baseline climate profile.

Investigating the Impact of Various Climate Change Scenarios on Pavement Service Life

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1. INTRODUCTION

Traditionally, pavement performance (deterioration) prediction focuses on the impact of applied traffic loading and assumes that future climate patterns can be simulated from historical data. Today, however, this assumption may no longer be valid because of the influence of global climate change. Climate change scenarios such as increased temperature and precipitation may be considered vital in exacerbating the deterioration rate of pavement materials and structures. Neglecting the impact of extreme events at the design stage may lead to reduced pavement service life and increased future pavement operation and maintenance cost from the agency's standpoint.

In the second half of the twentieth century, climate science has observed significant climate change events that are mainly associated with natural and manmade factors at global, regional, and local levels (1). The fifth assessment report by the Intergovernmental Panel on Climate Change (IPCC) revealed a rise of 0.74 °C in global mean surface temperature and an accelerating rate of warming with sea levels rising over the last 100 years (2). The IPCC has developed long-term emissions scenarios that have been widely used in assessing the impact of climate change. The scenarios represent various alternative images of the future climate with the influence of driving forces such as demographic development, socioeconomic development, and technological change on the future of greenhouse gas (GHG) emissions, as covered in the IPCC Special Report on Emission Scenarios (SRES). Four storylines based on these driving forces produced four sets of scenarios called families A1, A2, B1, and B2 (3). Hsu et al. reviewed information on climate change at the global and local scale in the Taiwan Climate Change Projection and Information Platform Project (TCCIP). The project applied statistical downscaling on the 24 IPCC general circulation model simulations from the fourth assessment report to project future climate change for temperature and precipitation for each season in Northern, Central, Southern, and Eastern Taiwan. Their results showed that the annual temperature in Taiwan increased by 1.4 °C between 1911 and 2009 and that warming has increased significantly at a rate of 0.29 °C per decade over the last 30 years. The near surface temperature projections for Taiwan under the A1B scenario, which is a world of rapid economic growth with balance among all sources, indicate that at the end of the twenty-first century, temperature increases will range between 2 °C and 3 °C. Projections for mean winter precipitation indicate a decrease of 1%–23%; in summer, an increase of 15%–45% has been observed (4).

Pavement performance is subject to both external and internal factors. The external factors of traffic, precipitation, temperature, and depth of the ground water table exert a marked influence on pavement performance. The internal factors are the materials, and infiltration potential of the overall pavement structure, all of which determine the extent to which the pavement reacts to the external environmental conditions (5). High temperatures from extreme events affect the aging of bitumen through oxidation and embrittlement, leading to premature surface cracking and raveling, thus reducing the functional life of the pavement. Soil moisture in the surrounding environment from infiltration by heavy rainfall or flooding through surface cracks weakens the flexible pavement sublayers, rendering them more susceptible to damage under heavy traffic. Thus, understanding how climate change or extreme events may affect pavement structures and ultimately their functional life is crucial (6). Li et al. examined the implications of climate change on pavement performance by using the Mechanistic-Empirical Pavement Design Guide (MEPDG). Four GHG emission models representing future policy scenarios (i.e., A1B, A2, B1, and B2) were used to project future climate change considering temperature and precipitation for 3 analysis years (i.e., 2030, 2050, and 2100) by using MAGGIC/SCENGEN software. MEPDG was used to evaluate the relative change between the baseline climate and future climate change scenarios (7). Pavement performance prediction can be achieved through performance modeling. In general, pavement performance modeling can be categorized into empirical, mechanistic-empirical (ME), and subjective. In the empirical model, certain measured or estimated variables, such as deflection, accumulated traffic loads, and environmental conditions, are related to loss of serviceability or some other measure(s) of deterioration and pavement age, usually through regression analysis. In the ME model, certain calculated responses, such as subgrade strain and pavement layer stress and strain, together with other variables, such as accumulated traffic load and environmental conditions, are related to loss of serviceability or some other measure(s) of deterioration through regression analysis or a model that is calibrated (i.e., the coefficients are determined) through regression analysis (8). One of the most widely used empirical pavement performance prediction models for flexible pavement is the empirical design equation used in the American Association of State Highway and Transportation Officials (AASHTO) 1993 design guide. The model empirically links pavement performance to various external and internal factors such as

accumulated traffic loading, soil properties, and environmental conditions. In the past two decades, this empirical model has been shown to be inadequate for representing the complex failure modes of flexible pavement in the field because of its limited ability to predict pavement performance in the field. In addition, the models developed for the AASHTO Road Test relate key pavement properties and traffic to performance but do not consider the range of climatic effects that can also contribute to pavement distress (9, 10). To address some of the limitations of AASHTO 1993 design equation, AASHTO in 2004 launched the MEPDG. This new design procedure incorporates mechanistic principles, including calculations of pavement stress, strain, and deformation responses, by using site-specific climatic, material, and traffic characteristics (11). In the ME model, the climatic effects such as temperature, moisture content, and other environmental parameters in the pavement structure and subgrade are considered using an enhanced integrated climate model (EICM) (12). Richter and Witczak used the EICM to evaluate moisture prediction capability and determined that the model could sometimes reasonably estimate the in situ moisture content of unbound pavement materials and that it might not work well for sites in arid climates (13). Salem performed a sensitivity analysis of the EICM by varying the crack lengths in the drainage and infiltration model, and the results showed no significant difference in the EICM volumetric moisture prediction. Thus, the EICM could predict the seasonal variation in the subgrade moisture only when the seasonal variation of groundwater at each season is known (14).

Therefore, the present study investigated the impact of various climate scenarios on the service life and life cycle cost (LCC) analysis of provincial and urban flexible pavement roads in Taiwan to develop resilient maintenance and rehabilitation (M&R) strategies. To achieve this objective, historical climate data collected from the northern and southern regions of Taiwan were used as a baseline scenario, and future climate change scenarios projected for 2020 to 2039 in Taiwan's northern and southern regions were compared against the baseline scenario. The EICM was used to estimate the volumetric moisture condition of the unbound layers (base and subgrade) of the pavement structure model. Pavement service life and LCC were estimated under the historical and predicted climate scenarios by using the AASHTO 1993 and AASHTO ME performance models.

2. RESEARCH METHODOLOGY

Two pavement performance models, AASHTO 1993 and AASHTO ME, were employed to predict the performance of two provincial road pavement sections located in the northern and southern parts of Taiwan. EICM Version 3.0 was used to predict the climate effect and compute the unbound material mechanical properties under various climate considerations. The results were used in the AASHTO 1993 design to predict pavement performance. The AASHTO ME, which also uses the EICM as a climate tool, was employed to predict the pavement performance under the same climate considerations to investigate the impact of extreme climate events on the pavement service life. An LCC analysis was also performed for an analysis period of 30 years.

2.1. Climate data collection

Two weather stations, one located in Wenshen District (Northern Taiwan) and the other in Yongkang District (Southern Taiwan), were identified for this study. Wenshen District is in the southernmost part of the Taipei Basin. It lies approximately along the coordinates of 25°N, 122°E (Wenshan-District 2014, May 13) (19). Yongkang District is located along the coordinates of 23°N, 120°E (Yongkang-District 2014, May 26) (20). The 10-year historical climate data from 2003 to 2012 for temperature, rainfall, wind speed, sunlight hours, sunrise, and sunset (recorded on an hourly basis from the two weather stations) were obtained from the Central Weather Bureau. Information on the ground water table depth variation of Taipei and Tainan Station were also obtained from the Department of Water Resources.

The variation in temperature, ground water depth, and rainfall are illustrated respectively in Figure 1 to Figure 3. Figure 1 shows that, throughout the recorded period, the mean maximum temperature in Taipei is slightly higher than that in Tainan. Conversely, the minimum temperature in Tainan is higher throughout the same period. Figure 2 shows that the average ground water depth in Taipei is 5.0 m, which depicts a relatively constant slope over the period. Tainan, however, has a considerably lower ground water table. The water depth ranges between 10.63 and 6.00 m. A low water table was first observed at the beginning of the period, followed by a steady rise to 6 m during 2009, when it subsequently dropped again steadily. The ground water depth in Taipei is 5.63 m higher than that in Tainan. Finally, Figure 3 shows that the total annual rainfall in Taipei ranges between 1656 and 3824 mm. In Tainan, the total rainfall is between 1012 and 3125 mm; during the study period, the highest rainfall was recorded in 2005.

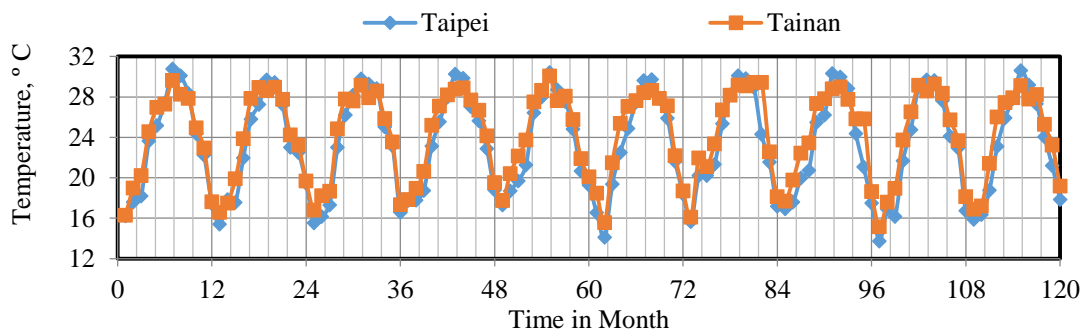


Figure 1. Average monthly temperature for Taipei and Tainan.

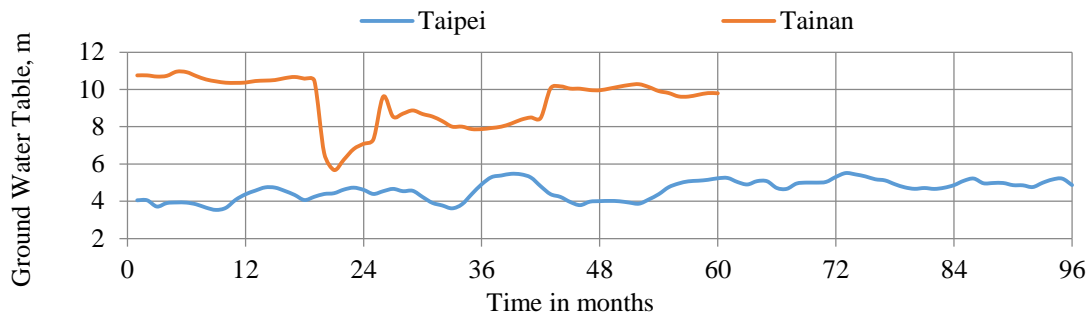


Figure 2. Average monthly ground water depth in Taipei and Tainan.

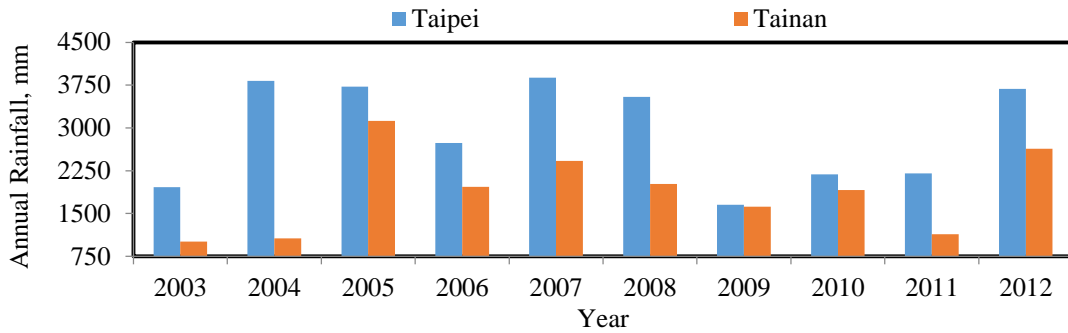


Figure 3. Total annual rainfall in Taipei and Tainan.

2.2. Climate scenarios

The extreme climate events assumed in this study are consistent with the climate change report, *Climate Change in Taiwan: Scientific Report 2011*, published by the National Science Council and based on the research of the TCCIP. The extreme climate events for temperature and rainfall were projected for 2020 to 2039 under Scenarios A1B, B1, and A2 among all the regions of Taiwan. The regional mean changes in temperature in degrees Celsius and rainfall in percent for the future climate under these scenarios were used at 10th and 90th percentile changes for winter (December, January, and February), spring (March, April, and May), summer (June, July, and August), and autumn (September, October, and November). Six climate scenarios, designated A1B.10, A1B.90, B1.10, B1.90, A2.10, and A2.90, were constructed for each study site (Figure 4 to Figure 7).

As described in the SRES (15), the A1 storyline and scenario family simulates a future world with very rapid economic growth, global population peaking during the midcentury and declining thereafter, and the rapid introduction of new and more efficient technology; by contrast, the A1B scenario is a balance among all the sources considered in the A1 scenario. The A2 scenario is a heterogeneous world that results in increasing global population; it has slower economic growth and technological change than the other storylines do. The B1 scenario is a convergent world with the same global population as that in A1. The B2 scenario is a world where emphasis is on local solutions to economic, social, and environmental sustainability. The last two numbers in the designations are for 10th and 90th percentile changes in mean temperature and rainfall, which represent the extreme cases investigated in this study. These changes were applied to historical climate data (2003–2012) to determine their effectiveness for predicting future climate change.

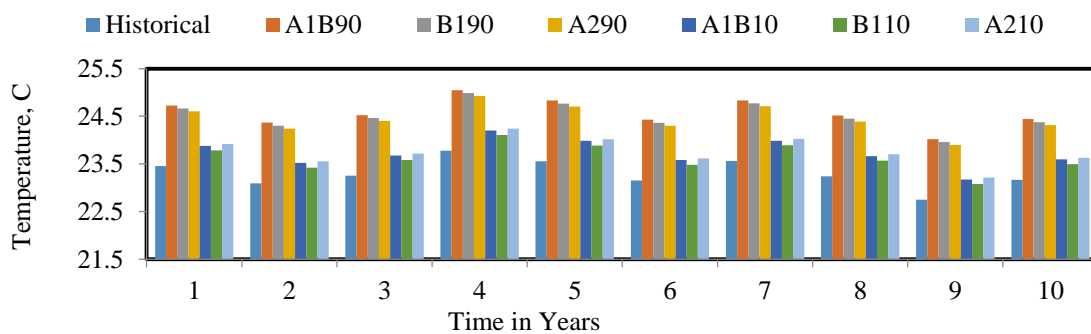


Figure 4. Mean annual temperature for historical (2003–2012) and future climate change scenarios (2020–2039) in Taipei.

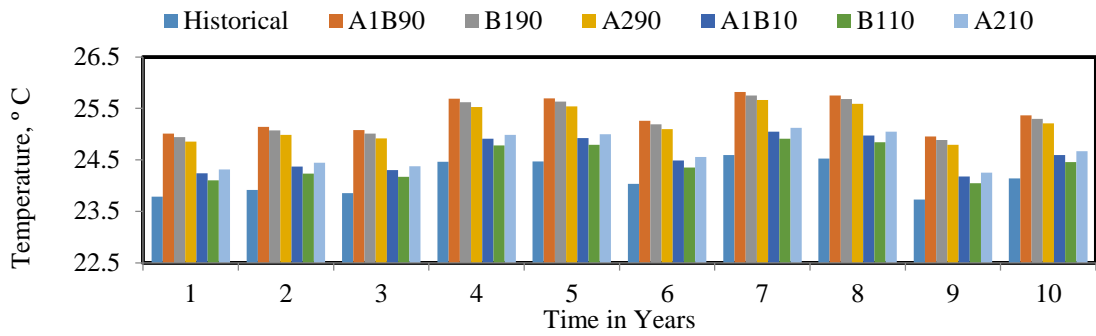


Figure 5. Mean annual temperature for historical (2003–2012) and future climate change scenarios (2020–2039) in Tainan.

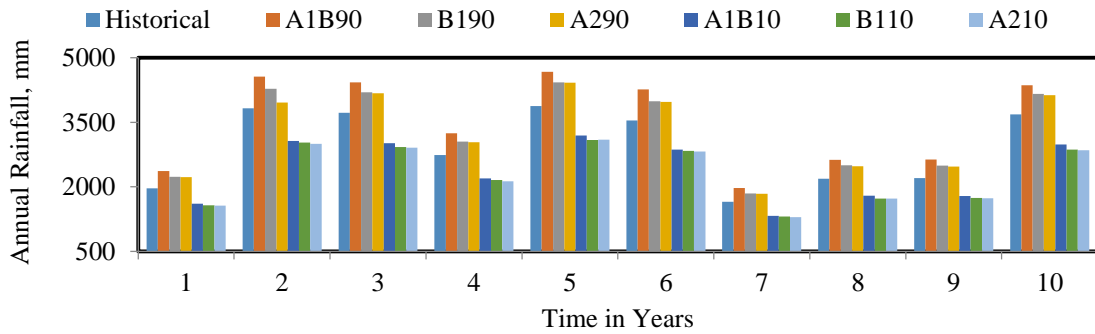


Figure 6. Total annual rainfall for historical (2003–2012) and future climate change scenarios (2020–2039) in Taipei.

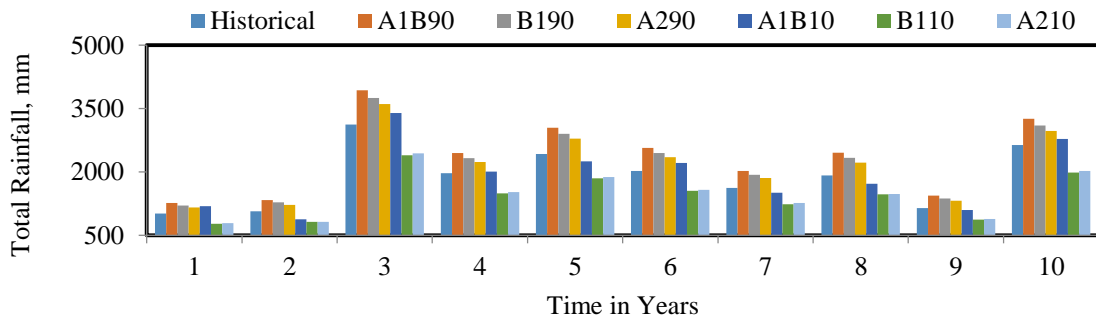


Figure 7. Total annual rainfall for historical (2003–2012) and future climate change scenarios (2020–2039) in Tainan.

2.3. Pavement materials

A typical three-layer pavement system was used as a baseline case in this study. This comprises 15 cm of hot mix asphalt (HMA) and 30 cm (11.81 in.) of base and subgrade layers. The materials for the pavement were selected according to the AASHTO 1993 and National Cooperative Highway Research Program (NCHRP) 2004 design specifications. The typical range of values for soil index properties such as percent passing #4, #200 sieves, Atterberg limit test plasticity index (PI), and liquid limit (LL) (AASHTO T27) were determined from a literature survey. Resilient modulus of 3,100 MPa was assumed for the HMA. Values of 1.64 and 1.09 Watt/(m)(K) as well as 0.38 Watt/(m)(K) were assumed for the surface absorptivity and heat capacity as well as the thermal conductivity, respectively. Typical values recommended for these properties as Level 3 inputs for the AASHTO ME design analysis (16). Base materials corresponding to A-1-a of the AASHTO Soil Classification System and material gradation conforming to AASHTO M147 specifications for road base materials were assumed. Typical value ranges for P4, P200, PI, and LL were obtained from the literature; the average values were used to estimate the material properties. Among the EICM Level 2 inputs, P200 and PI correlated with the other material properties, namely the specific gravity (G_s), optimum degree of saturation (S_{opt}), and porosity (θ), in obtaining the gravimetric water content (w_{opt}) and maximum dry unit weight of the materials under the assumption of standard compaction as per AASHTO T180 (16). Similarly, the material properties for the subgrade layer as Level 2 inputs were estimated using the aforementioned procedure for the base layer. The layer material was assumed to be A-6.

2.3.1. Time series material property for unbound materials

The volumetric water profiles obtained from the EICM analysis under historical and predicted climates were used to estimate the degree of saturation in the base and subgrade layers over time. The resilient modulus, as the fundamental pavement mechanical material property, was estimated as a function of the degree of saturation in the base and subgrade layers. Extant predictive models account for the variation in the resilient modulus with the moisture.

Through the application of these models, the model in Equation 1 can be selected to analytically predict changes in the modulus resulting from changes in the moisture, and this model is currently implemented in the EICM [21].

$$\text{Log}[M_R/M_{R \text{ opt}}] = a + \left[(b - a) / (1 + \text{EXP} \times (\beta + K_s \times (S - S_{\text{opt}}))) \right] \quad \text{Equation 1}$$

where,

- a = minimum of log MR/MR opt;
- b = maximum of log MR/MR opt;
- β = location parameter, obtained as a function of a and b = $\ln(-b/a)$;
- K_s = regression parameter; and
- $S - S_{\text{opt}}$ = variation in the degree of saturation (expressed as a decimal).

Equation 1, which was implemented by the MEPDG, was used to estimate the time series resilient moduli of the base and subgrade layers. The 10 years' monthly resilient moduli values for the base and subgrade layers were thus obtained under the historical and future climate change scenarios. The time series moduli estimated from historical climate data were used to project the resilient moduli of the base and subgrade layers for the 11th year. Thus, the monthly time series moduli of the base and subgrade layers were obtained for the 20-year design period.

2.4. Traffic estimation

Typical four-lane urban roads in Taiwan were modeled for this study. Traffic was analyzed according to the types of truck commonly used for freight transportation in Taiwan (17). These trucks were then classified using the MEPDG vehicle class system and redistributed with the obtained truck factors. An average annual daily truck traffic of 1500 was assumed, and the distribution factors recommended by AASHTO 1993 for the direction (D) and lane (L) were obtained as 0.5 and 0.94, respectively. A plot of the cumulative ESALs versus time is shown in Figure 8.

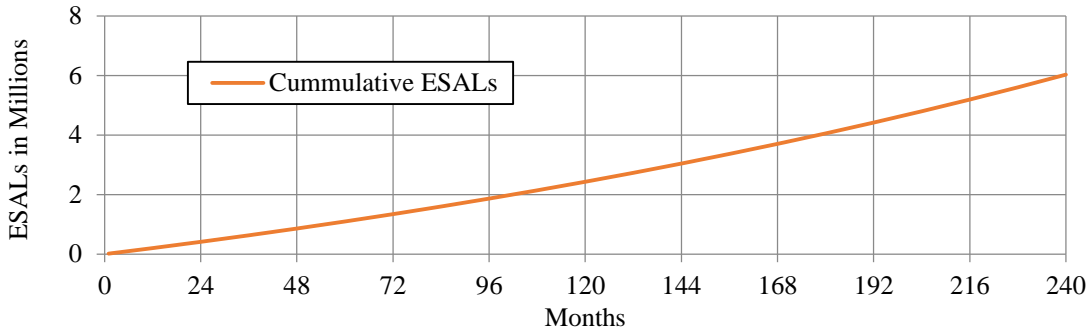


Figure 8. Plot of 18-kip cumulative ESALs versus time for baseline case.

2.5. Pavement performance evaluation

2.5.1. Pavement service life estimation using the AASHTO 1993 design method

The pavement structural number (SN) was used as a measure of the overall structural capacity of the pavement structure at any given period. The objective was to determine the pavement SN at 1-month time intervals for the entire design life. Structural layer coefficient a_1 for HMA was estimated using Equation 2 [22]. The layer coefficient a_2 for the base was estimated from the time series base modulus by using Equation 3, which is provided in the AASHTO (1993). The SN was evaluated using Equation 4. The initial and terminal present serviceability indices assumed were 4.2 and 2.0, respectively. The SN and ESAL values obtained were used to compute the loss of serviceability Δ PSI for the baseline, historical, and future climate change scenarios by using the AASHTO 1993 pavement equation. The input parameters assumed for the equation are 90, -1282, and 0.45 for the reliability, reliability factor, and overall standard error, respectively. The obtained Δ PSIs versus the ESALs were plotted, and the pavement service life was thus determined under the baseline, historical, and future climate change scenarios.

In the estimation of the SN and pavement service life, three cases were considered for both test sites. The value of the drainage coefficient was assumed to vary with the climate scenarios, in which the layer coefficients were modified according to environmental conditions. Drainage coefficients of 1.0, 1.1, and 0.8 were assumed for the mean change in the historical, 10th and 90th percentiles, respectively. This assumption premised that the drainage coefficient will change according to the projected future climate relative to the historical climate. The reason for varying the drainage coefficient was to simulate pavement exposure to various moisture scenarios. For the 90th percentile scenarios, the projected rainfall will increase in the future; thus, the same drainage design might not adequately discharge the water in a short time. This will increase the time of pavement exposure to moisture approaching the saturation level compared with the historical climate, hence the lower drainage coefficient. By contrast, the 10th percentile scenarios project less rainfall in the future, resulting in drier pavement conditions and a shorter discharge time for water. Therefore, in the present study, a higher drainage coefficient was used relative to the historical climate.

$$a_1 = 0.4 \times \log(E_{HMA}/3000\text{MPa}) + 0.44, \quad 0.2 < a_1 < 0.44 \quad \text{Equation 2}$$

$$a_2 = 0.249(\log 10 E_{BS}) - 0.977 \quad \text{Equation 3}$$

where;

a_1 = structural layer coefficient for HMA;
 a_2 = structural layer coefficient for the base layer;
 E_{BS} = layer resilient modulus; and
 E_{HMA} = resilient modulus of HMA.

$$SN = a_1 D_1 + a_2 D_2 m_2 \quad \text{Equation 4}$$

where,

a_1 = structural layer coefficient for HMA;
 D_1 = thickness of HMA;
 D_2 = thickness of the base layer; and
 m_2 = drainage coefficient.

2.5.2. Pavement service life estimation (MEPDG) model

The climate files for the historical and predicted climate scenarios prepared for the EICM were converted into integrated climate model (ICM) climate files. The pavement configuration and unbound material properties described in the preceding sections for the EICM moisture prediction were assumed for the MEPDG analysis, and these were also used in the EICM for climate modeling. Vehicle class and an ADTT of 1500 were assumed for the MEPDG traffic analysis. MEPDG traffic default values were used for the remaining information on traffic. HMA properties were used for the super-pave binder option at Level 2 inputs. The cumulative percentages retained on 3/4-in., 3/8-in., #4 sieve, and percent passing #200 sieves were obtained in accordance with the ASTM D3515 gradation specifications for dense asphalt mixtures. Typical effective binder content, air voids, Poisson's ratio, and total unit weight of asphalt concrete are 5.0%, 4.0%, 0.35, and 2.35 kg/m³, respectively, as obtained from average historical job mix formula reports. MEPDG was used to analyze the pavement distress under the historical and future climate change scenarios. The distress of top-down cracking, fatigue cracking, AC rutting, total rutting, and smoothness, known as the international roughness index (IRI), are defined for HMA. The target values set for the distress are given as (a) top-down cracking = 190 m/km; (b) fatigue cracking = 25%; (c) AC rutting = 6 mm, (b) total rutting = 20 mm, and (c) IRI = 2.69 m/km at a reliability of 90.

2.6. LCC analysis

The service life of pavement depends on various factors including environmental factors such as temperature and precipitation (18). The typical LCC analysis period ranges between 25 and 50 years. In the present study, an LCC analysis was performed for a pavement section of 16 km. under baseline, historical, and future climate change scenarios over a 30-year analysis period. No routine and preventive maintenance activities were assumed to be applied during the pavement service life. Thus, pavement sections were assumed to receive a major rehabilitation at the end of the service life, and the serviceability was assumed to be reintroduced as a new construction status. The labor and user cost was assumed to be constant for all the cases and therefore did not affect the analysis. For the purpose of this study, the LCC analysis considered only the initial construction cost and the cost of rehabilitation at the end of the service life predicted using the AASHTO 1993 and AASHTO ME performance models.

The initial construction cost was determined from the construction material quantities, such as the HMA, base, and subgrade layers (estimated tonnage), for a section of pavement. Unit prices for the materials were obtained as per Taiwan's current unit rates. The pavement was assumed to be rehabilitated to return it to near original condition as soon as it reaches the end of its service life; this was achieved by restoring the HMA thickness to its original thickness. The initial and rehabilitation cost at the current dollar value can then be obtained, and the future cost is discounted to compute the net present value (NPV) as shown in Equation 5. The pavement remaining or the residual value at the end of the analysis period and the LCC under different climate considerations (including the baseline) were thus determined using Equation 6 and Equation 7, respectively.

$$NPV = \text{Initial Cost} + \sum_{k=1}^N \text{Rehab. cost}_k [1/(1+i)^n] \quad \text{Equation 5}$$

where,

i = discount rate; and
 n = year of expenditure.

$$\text{Residual Value} = \text{Last Rehab. Cost} \times [(\text{Service Life} - \text{Activity Age})/\text{Service Life}] \quad \text{Equation 6}$$

$$LCC = \text{Initial Cost} + \text{Total Discounted Rehab. Cost} - \text{Residual Value} \quad \text{Equation 7}$$

3. RESULTS AND DISCUSSION

3.1. Mechanical properties of unbound materials

The material properties of the pavement layers and the resilient moduli of the base and subgrade layers were estimated using the saturation results. The average resilient moduli estimated for the base and subgrade layers over the pavement performance period in Taipei were 197 and 49 MPa, respectively. In Tainan, 205 and 54 MPa were estimated for the base and subgrade layers, respectively, under the historical climate conditions.

3.2. Future climate under 10th percentile change

The percentage decrease in the base and subgrade moduli for the A1B.10, B1.10, and A2.10 scenarios relative to the baseline are shown in Figure 9 and Figure 10. The base and subgrade moduli predicted for Taipei were slightly less than those predicted for the historical climate conditions. By contrast, the modulus values predicted for Tainan for the base and subgrade layers were higher than those for the historical climate conditions. The decrease in the base modulus for Taipei ranged between 10.5% and 11.0%. A decrease in the base modulus values between 6.7% and 7.4% was observed in Tainan. The percent reduction in the subgrade modulus was between 28% and 29.2% in Taipei. Tainan showed a 22.0%–23.9% decrease. The subgrades the percentage decrease in the modulus for A1B.10, B1.10, and A2.10 ranged between 28.0% and 29.2% in Taipei. In Tainan, the percentage decrease ranged between 23.5% and 24.1%.

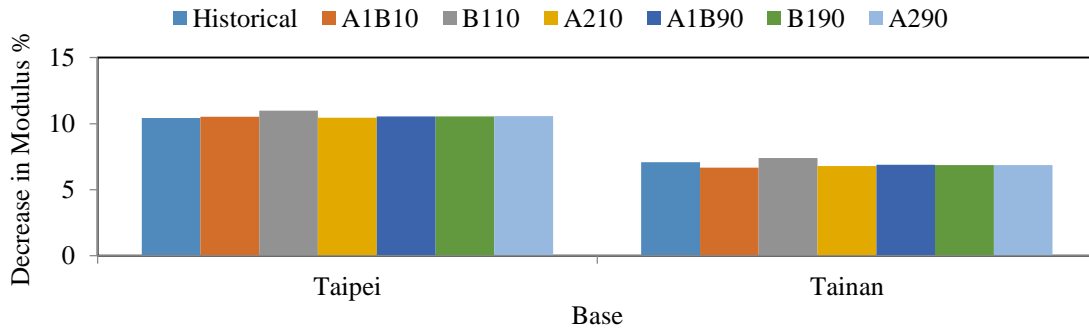


Figure 9. Modulus decrease in the base layer under the historical and future climate scenarios.

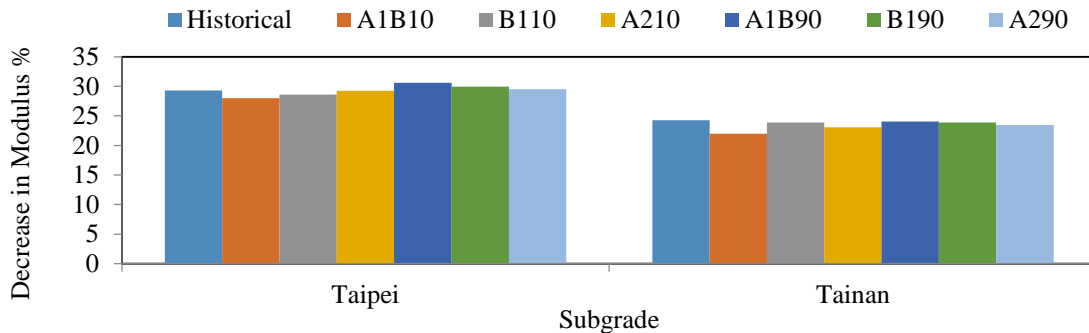


Figure 10. Modulus decrease in the subgrade layer under the historical and future climate scenarios.

3.3. Future climate under 90th percentile change

No significant change was observed in the predicted base and subgrade modulus for the A1B.90, B1.90, and A2.90 scenarios in Taipei and Tainan. The base and subgrade moduli predicted for these scenarios were slightly less than those predicted for the historical climate conditions. This is consistent because the scenarios represent increasing rainfall. The percentage decrease in the base moduli was 10.6% for all three future climatic scenarios in Taipei, which was slightly higher than the moduli predicted for the historical climate conditions. In Tainan, a 6.9% decrease in the base moduli for the three scenarios was observed, which was slightly less (by 0.2%) than the predicted modulus for the historical climate conditions, as shown in Figure 9. For the subgrade modulus, the variations were 1.1% and 0.6% in Taipei and Tainan, respectively, as shown in Figure 10.

3.4. Pavement performance prediction by AASHTO 1993 Performance Model

The SNs predicted for the historical and future climate change conditions are presented in Table 1. Higher SNs were predicted for A1B.10, B1.10, and A2.10 for the historical climate conditions, whereas lower SNs were predicted for A1B.90, B1.90, and A2.90 both in Taipei and Tainan. However, the SNs predicted for Taipei were lower compared with Tainan for both the historical and future climate scenarios. The difference between the SNs predicted for the 10th percentile change and that predicted for the 90th percentile change was respectively 0.47 for Taipei and 0.49 for Tainan. These changes reveal little or no variation in the SN between the 10th and 90th percentile changes. For the A1B.10, B1.10, and A2.10 scenarios, the percent increase in the SN over the historical climate conditions was 7.7% in Taipei and 7.6% in Tainan. The variations between the 10th and 90th percentile mean changes were 4.1% for Taipei and 3.6% for Tainan.

Table 1. Average SN predicted for Taipei and Tainan.

<i>Climate Scenarios</i>	Taipei	Tainan
<i>Historical</i>	4.17	4.21
<i>A1B.10</i>	4.32	4.38
<i>B1.10</i>	4.32	4.37
<i>A2.10</i>	4.33	4.37
<i>A1B.90</i>	3.85	3.89

B1.90	3.85	3.89
A2.90	3.85	3.89

3.4.1. Pavement service life under baseline and historical climate Scenario

The service life for the baseline design, as predicted using the AASHTO 1993 model, was 20 years. The pavement service life predicted for the historical climate conditions was 12.3 years in Taipei and 12 years in Tainan, as shown in Figure 11 and Figure 12, respectively. The service life predicted for Taipei and Tainan was reduced by 7.7 years (39%) and 8.0 years (40%), respectively.

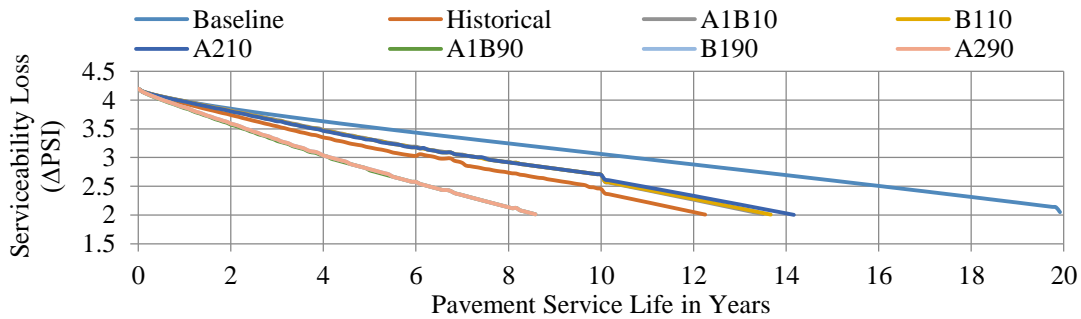


Figure 11. Pavement service life predicted in Taipei.

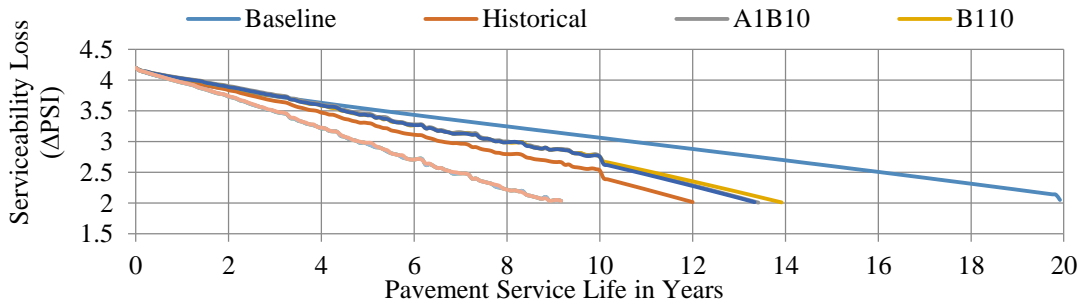


Figure 12. Pavement service life predicted in Tainan.

3.4.2. Pavement service life under the 10th percentile change scenario

The pavement service life predicted for the A1B.10, B1.10, and A2.10 scenarios in Taipei and Tainan are shown in Figure 11 and Figure 12, respectively. The predicted service life ranged between 13 and 14 years in Taipei and Tainan, respectively. However, the service life predicted for these scenarios showed a decreasing trend relative to the baseline case; furthermore, increases relative to the historical climate conditions in Taipei and Tainan were observed, as shown in Figure 11 and Figure 12, respectively. Relative to the baseline in Taipei, the pavement service life was predicted to drop by 32%, 31%, and 29% for the A1B.10, B1.10, and A2.10 scenarios, respectively. In Tainan, the pavement service life was predicted to drop by 33% for A1B.10 and A2.10 and by 30% for B1.10, as shown in Figure 13. The variation in the service life reduction was only 3% in Taipei and Tainan. Service life increased relative to the historical climate conditions by 6% for A1B.10, 7% for B1.10, and 9% for A2.10 in Taipei. In Tainan, the service life increased by 7% for A1B.10 and A2.10 and 10% for B1.10 relative to the historical climate conditions. Variation of 3% was observed in Taipei and Tainan.

3.4.3. Pavement service life under the 90th percentile change scenario

For all climates, the service life predicted for these scenarios was 8.5 years in Taipei and 9.0 years in Tainan. The predicted service was reduced significantly relative to the baseline and historical climate conditions in both Taipei and Tainan. Figure 13 shows the service life reduction as percentages for Taipei and Tainan. A 57% service life reduction was predicted for all the scenarios in Taipei. In Tainan, a 53% decrease in service was predicted for A1B.90 and a 54% decrease was predicted for B1.90 and A2.90 relative to the baseline. In relation to the historical climate conditions, the service life decreased by 19% for the three scenarios in Taipei. In Taiwan, the service life decreased by 13% for A1B.90 and 14% for B1.90 and A2.90.

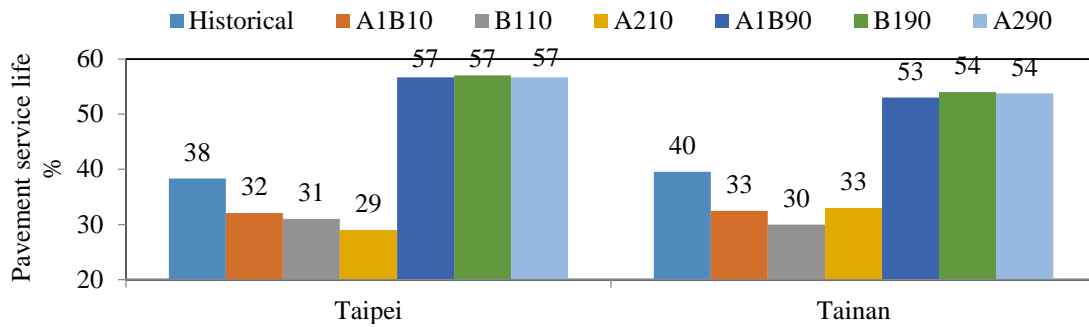


Figure 13. Percent reduction in pavement service life for the historical and future climate conditions relative to the baseline.

3.5. Pavement performance prediction by M-E performance model

Five pavement distresses performance were predicted by AASHTO M-E model under various climate scenario. The pavement distress predicted for the top-down cracking, fatigue cracking, AC rutting, total rutting, and IRI were similar for all the future climate scenarios and historical climate conditions in Taipei and Tainan. The total rutting and IRI predicted were far lower than their design limits of 19 mm and 2.69 m/km, respectively, for all the cases in Taipei and Tainan. In all the cases, only HMA rutting and top-down cracking were predicted to have exceeded their design limits of 6.35 mm and 190 m/km, respectively. The HMA rutting failed at Year 20 for all the climate cases considered in Taipei and Tainan. However, top-down cracking failure was predicted at less than half of the pavement age in both Taipei and Tainan, as shown in Figure 14 and Figure 15, respectively. In Taipei, distress failure varied slightly between 99 months (8.0 years) and 104 months (8.5 years) under all the climate scenarios considered in this study. The shortest pavement service life was predicted for A1B.90. In Tainan, the predicted distress failure was 103 months (8.5 years) for all the climate scenarios.

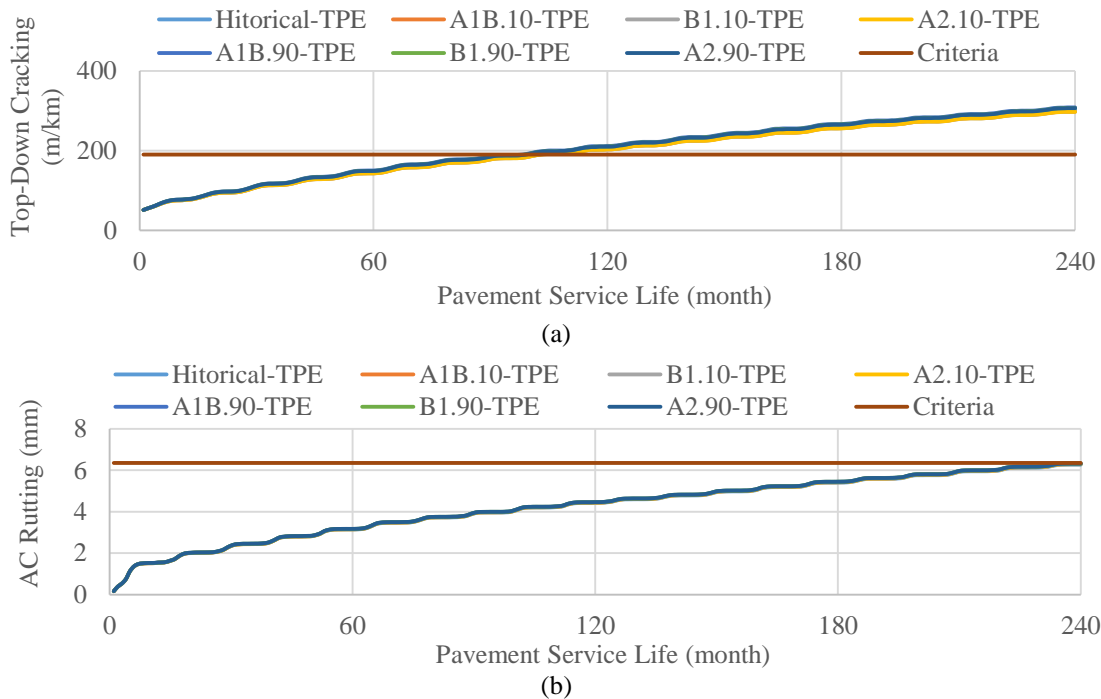
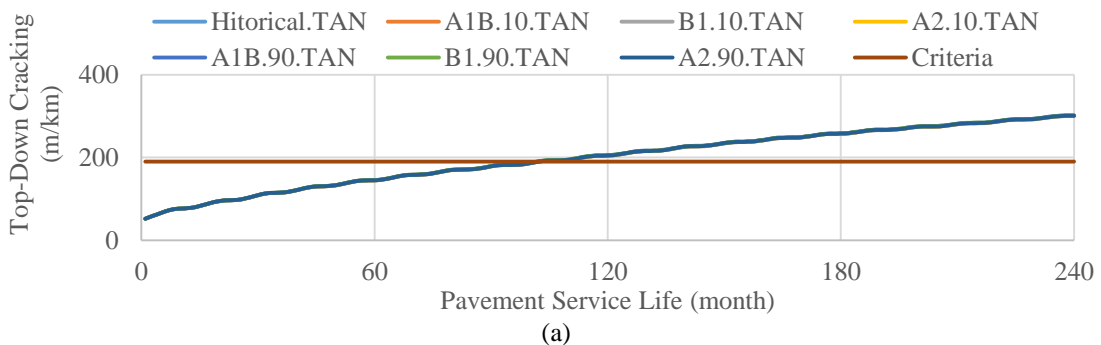


Figure 14 Pavement distresses prediction for (a) Top-Down Cracking and (b) AC Rutting of Taipei Site by MEPDG performance model.



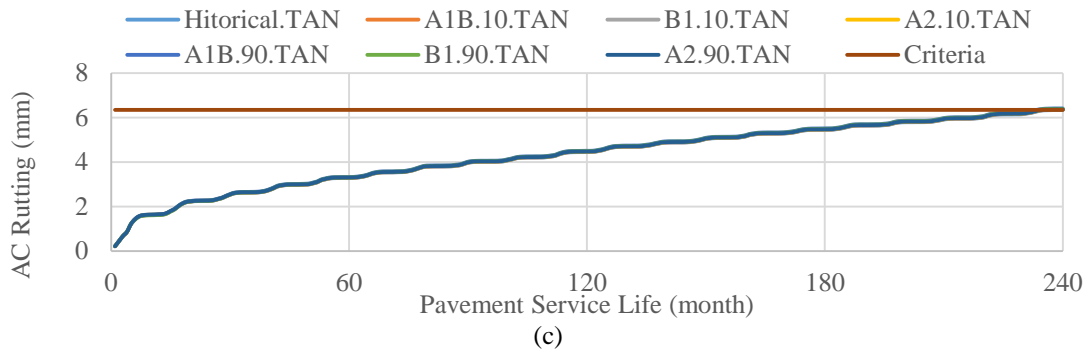


Figure 15 Pavement distresses prediction for (a) Top-Down Cracking and (b) AC Rutting of Tainan Site by MEPDG performance model.

3.6. LCC analysis

The life cycle cost analysis for all the cases considered in the study was performed in analysis period of 30 years. The performance curves for all cases on which life cycle cost analysis was performed are shown in Figure 16 and Figure 17 for Taipei and Tainan under the AASHTO 1993 model. The total cost of pavement for the different climate scenarios including the historical and baseline cases are presented in Table 2. The total cost of pavement for the baseline is NT\$152 million. The percent increase in pavement cost for the climate scenarios was determined relative to the baseline case. The cost of the pavement for the historical climate conditions increased by 54% for Taipei and Tainan. The percent increase in cost was 42% for A1B.10 and 38% for B1.10 scenarios in both Taipei and Tainan. The percent increase in cost for the A2.10 in Taipei was 38% and was 46% increase in cost in Tainan. The percent increase in cost for the A1B.90, B1.90, and A2.90 scenarios was 92% in Taipei and 65% in Tainan, as shown in Figure 18. With the ME model, the increase in the pavement cost for all the climates in Taipei and Tainan were 92% and 65% of the baseline cost, respectively. The pavement cost in Tainan was 27% lower than that in Taipei for a similar prediction for the 90th percentile mean change in the AASHTO 1993 model. The life cycle cost under the MEPDG model is shown in Figure 19. The increase in pavement cost for all the climates in Taipei is 92% of the baseline cost and 65% in Tainan. It is seen that Tainan is 27% lower in pavement cost compare to Taipei similar to the prediction for 90 percentile mean change in AASHTO 1993 model.

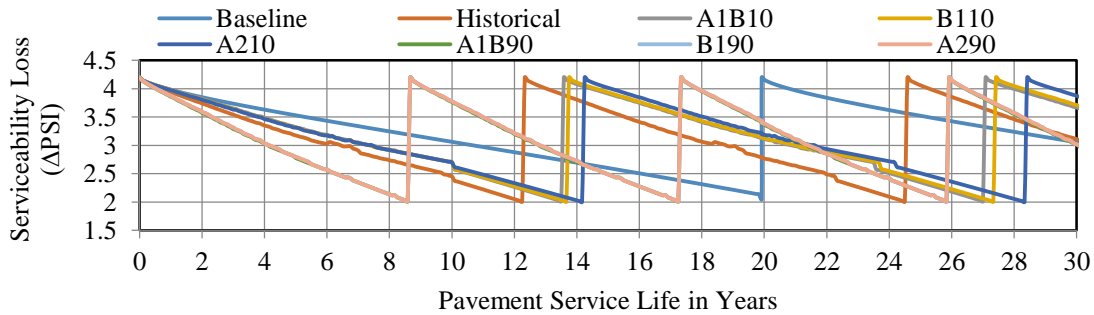


Figure 16 Pavement life cycles in Taipei

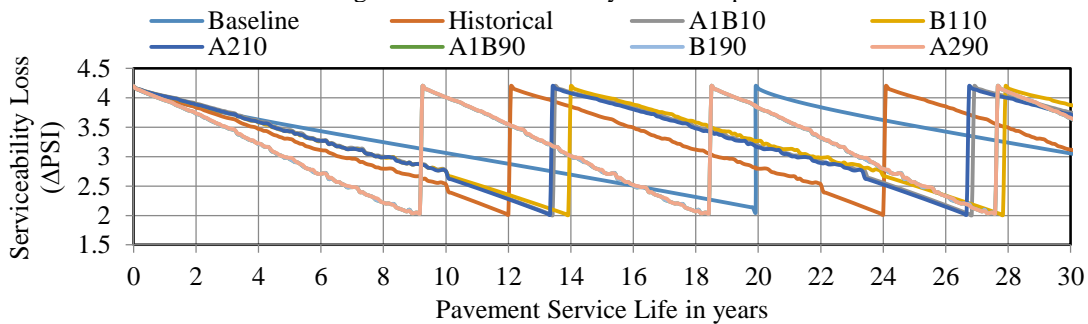


Figure 17 Pavement life cycles in Tainan

Table 2. Total pavement life cycle costs in millions (NT\$)

<i>Climate Scenarios</i>	Taipei	Tainan
<i>Baseline</i>	152	152
<i>Historical</i>	234	234
<i>A1B10</i>	215	215
<i>B110</i>	209	209
<i>A210</i>	209	221

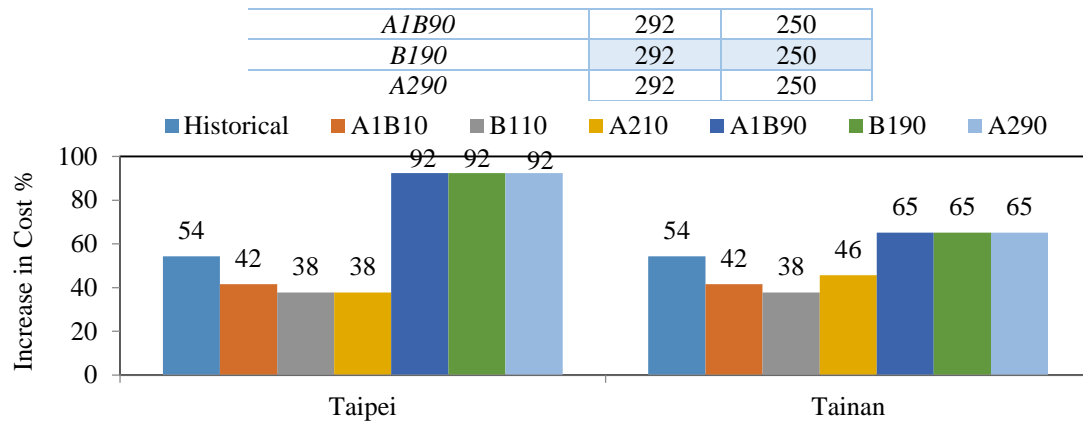


Figure 18 Percent increase in pavement cost (AASHTO 1993 model)

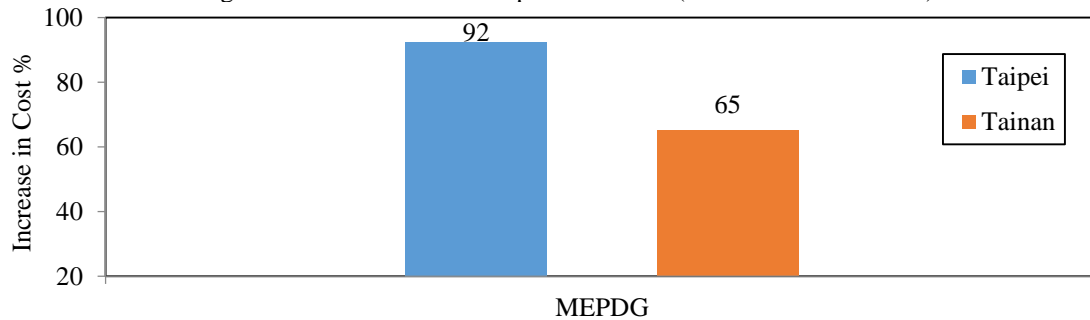


Figure 19 Percent increase in pavement cost for all climates (MEPDG model)

4. CONCLUSIONS

The primary goal of this study was to investigate the impact of various extreme climate change scenarios on the flexible pavement service life and LCC to develop resilient M&R strategies from the perspective of highway agencies. During the study, two widely used pavement performance models in pavement design, AASHTO 1993 and MEPDG, were employed to investigate the service life of a typical three-layered pavement system for Taiwan's city and county roads in the northern and southern regions under various future climate projections. The use of the EICM, which is the main climatic tool in the MEPDG performance model, was also incorporated into the AASHTO 1993 performance model for climatic considerations. The study revealed that by incorporating the EICM into the AASHTO 1993 performance model, the variation of the unbound material mechanical properties (resilient modulus) and pavement structural capacity under the various climate changes were addressed. The two models, AASHTO 1993 and MEPDG, also addressed the pavement service under the various climate scenarios, including the historical climate conditions, upon which LCC models were established, showing that future maintenance and operations are affected by the changing climates. Therefore, the framework developed may be a suitable model that can be followed by highway agencies to incorporate climate change into the planning, design, construction, and maintenance of new and existing pavement structures.

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6 Critical Steps to Manage Your Pavement Assets without Breaking the Bank

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Abstract

With oil pricing falling, it has created ripple effects on businesses. Companies are tightening their budget and monitoring costs more closely. In managing pavement assets, many agencies are becoming more cost-conscious on infrastructure investment as well. The worst strategy will be to defer all maintenance. However, deferring maintenance will only exacerbate the problem and create larger fiscal potholes.

By following the six critical steps, it doesn't matter whether you are just about to implement a new pavement management system or managing an existing system, whether you are in public sections or private entities, every step is designed to reduce costs, improve cost-efficiency, and promote infrastructure sustainability. In return, these steps help improve customer satisfaction and deliver accountability and transparency to motoring public, and minimize lifecycle costs on managing and maintaining pavement assets.

Introduction

Having a sustainable transportation infrastructure is the ultimate goal for asset managers who plan, manage, operate, and maintain. However, to reach the goal, managers will have to overcome many obstacles. Over the last 30 years, the growing concern is about financial sustainability to preserve the current transportation systems. There is an alarmingly increasing recognition that when budget is tight, short-term decisions are made on investment, maintenance, and preservation of infrastructure. Pavement asset, the backbone of the transportation system, as a result, suffered from the lack of long-term sustainability solutions. Coupled with the continued public demand for transportation services, we are seeing more potholes in the streets and more damages to our vehicles resulting in higher vehicle operating costs. Transportation professionals recognize these consequences and have gradually adopted asset management approach to optimize the public's return on their investment. However, to effectively implement a pavement management system (PMS) can be a daunting task if not properly planned. Hence, it is the objective of this paper to demystify and simplify the business process to manage your PMS.

There are six critical steps or business processes that made up the framework of an effective PMS as shown in Figure 1, namely inventory, condition assessment, needs assessment, prioritization, investment analysis, and feedback.

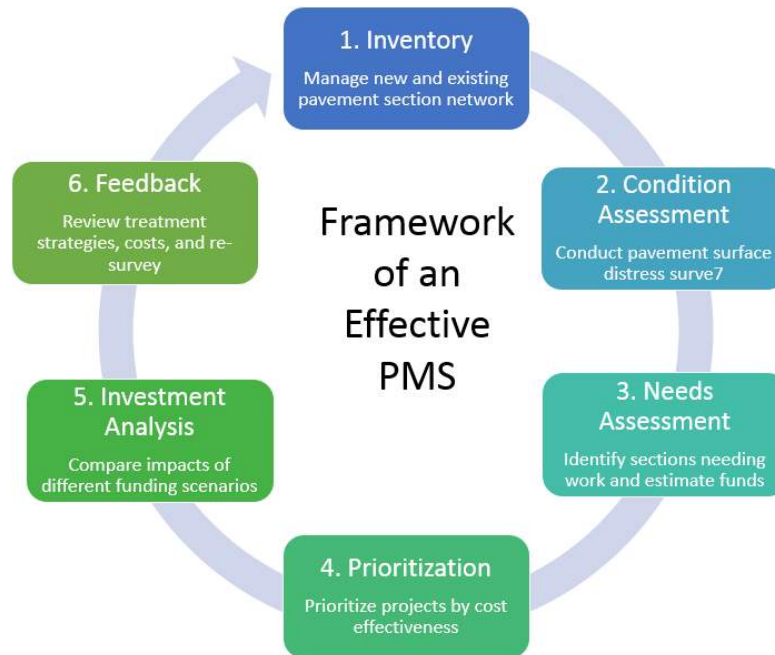


Figure 1. Framework of an effective PMS

Step 1: Inventory

To setup an effective PMS, you will need to setup an inventory system of the road network that supports the agency's goals and needs. If you are starting from scratch, it is necessary to know what types of records to maintain. Too often, mistakes are made to collect too many attributes that ultimately impede the implementation and discourage ongoing maintenance of data. Here are the key questions to ask:

1. What level of agency decision will be supported?
2. Are resources available to support decisions?

There are typically three levels to support pavement management decisions. First, at strategic level, decision is made to support the overall long-term sustainability of the pavement management program. This is where investment decisions and policies are made at the highest level within an organization. The detail of data at this level is very low. Second, at the network level, strategies are identified based on agency's goals. At this level, the overall maintenance needs are considered at the entire road network. Typically, at network planning level, multiyear of maintenance and rehabilitation (M&R) programs are developed. Even though the detail of

data has increased moderately, the required data is still very manageable. For example, maintenance policy may include just thin asphalt concrete overlay instead of more specific treatment like 0.75" (19mm) ultra-thin bonded wearing course. Finally, at project level, decisions made are localized to specific locations and design specifications. Data collected at this level maybe very detailed. Agencies, large or small, should weigh in on how much information is needed at this level, and the resources to maintain it.

For majority of the agencies, large or small, using PMS that designed as a network-level planning tool is sufficient. The following guidelines will help pavement managers to determine the detailed of inventory:

- Identify minimum data requirements for network-level decisions
- Data collection is costly; collect only what is needed.
- Consider PMS software that is easy to use and maintain.

Step 2: Condition Assessment

Once inventory is established, it is imperative to determine the type of pavement rating protocol to use. It is true that there are many pavement rating protocols in the market. Some has risen due to local needs, while others are more subjective rating. However, it is important not to reinvent the wheel by creating a custom rating for in-house use. The problem is this in-house expert team may retire, change job or be promoted, taking the institutional knowledge along.

Some main characteristics to consider are:

- Distress information provides early intervention of preventive maintenance
- Rating protocol is supported by recognized organization
- Rating protocol is widely used
- Rating protocol can be easily adopted for manual and automated surveys

The industry standards are the FHWA's *Distress Identification Manual for the Long-Term Pavement Performance Program* and *ASTM D6433- Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys*. The LTPP is geared toward state highways, while ASTM D6433 is suitable for local municipalities. The ASTM D6433 is based on pavement surface distress, severity, and quantity. It uses a composite pavement distress index called Pavement Condition Index (PCI) with a rating scale of 0-100, where 100 means new pavement, while zero means pavement needs reconstruction.

As far as conducting the pavement condition survey, agency can choose to do walking survey by using 2,500 ft² (232 m²) sample size (Shahin, M.Y., 2005). By using a sampling technique, agency will reduce substantial effort in resources. For agencies performing in-house survey, the most efficient use of staff time is after construction season is over. Pavement distress survey training

is available at the Metropolitan Transportation Commission (MTC) or at the Colorado State University. For most agencies, it is recommended that roads with heavy traffic such as collectors and above are surveyed every two years, while low volume roads or parking lots can be surveyed at four to five-year interval.

If you have heard the saying “I don’t trust the data”, it is possible that the PMS has fallen to the “garbage in, garbage out” symptom, which is the worst enemy of an effective PMS. Hence it is equally important is to establish a data quality management plan. In recent years, there have been several publications that discussed about best practices on data quality management (NCHRP Synthesis 401). The *Practical Guide for Quality Management of Pavement Condition Data Collection*, published by FHWA in 2013, is probably the best guide available today. This guide documents the procedures and frequency of pavement survey, quality control for data collection, and data assurance.

The key takeaways for condition assessment are:

- Commit to condition assessment at regular interval
- For network-level planning, assessment by sampling is the most cost-effective way
- Set up a data quality management plan

Step 3: Needs Assessment

It may be easy to come up with a list of streets to fix annually for a small network. When it comes to developing multiple years of M&R projects, however, the pavement managers that “know the streets like the back of their hands” will be overwhelmed. This is because pavements deteriorate constantly, by projecting pavement conditions to future years, coupled with traffic, environmental, social, and economic factors, the decisions are increasingly harder to make. This is when a PMS software will be useful to develop the short term capital improvement program and the long term transportation investment plan.

For each treatment setup in the decision tree, trigger rules must be established as shown in Figure 2, to define when a treatment is triggered once a set of pavement condition is met. Agency will be able to establish treatment type and trigger values based on local maintenance policies. The PMS software should be flexible to setup network-level type of treatment for preventive maintenance, rehabilitation, and reconstruction.

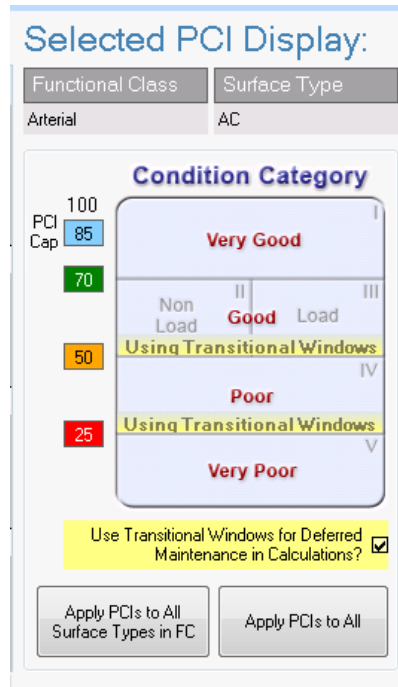


Figure 2. Setting up treatment trigger rules (StreetSaver[®])

Based on the decision tree established, the PMS tool should be capable of integrating a pavement preservation approach (Tan et al, 2011). There will be two lists of streets that needed work generated. The first list should consist of streets that require pavement preservation, including preventive maintenance (PM). This type of treatment is applied to pavement in relatively good condition, typically at PCI around 70 as shown in Figure 3. It is a cost-effective way to extend the service life of pavement without adding structural capacity. The other list consists of streets that need minor and major pavement rehabilitation. These treatments are applied typically for PCI between 25 to 70, while reconstruction for PCI below 25.

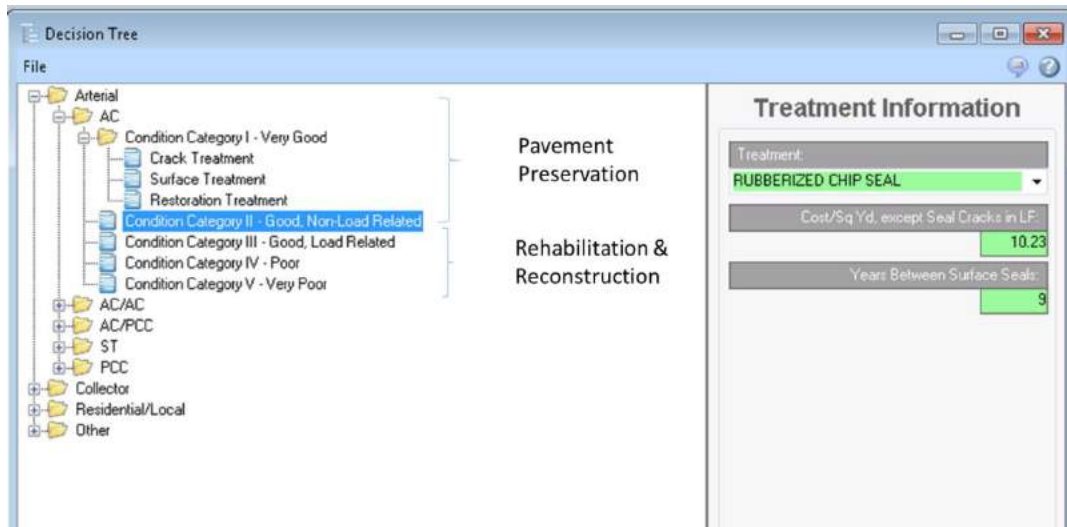


Figure 3. Decision tree illustrates integration of pavement preservation (StreetSaver®)

Along with the lists, there should be treatment costs to fix these street segments, categorized by preventive maintenance, minor and major rehabilitation, and reconstruction. This entire M&R plan, which consists of streets to be treated with costs, constitutes the road maintenance needs assessment. It usually includes the entire road network. As such, if M&R is performed, it will improve the overall network PCI to 80's, which is considered the state of good repair.

Key takeaways:

- Decision tree must be able to support preventive maintenance, minor and major rehabilitation, and reconstruction
- Decision tree promotes pavement preservation

Step 4: Prioritization

In ideal situation, agency will have sufficient funding to address all of its pavement needs. In reality, however, only a few agencies have adequate funding to meet their needs. Consequently, many agencies are relying on their PMS to allocate the limited funding in the most cost effective way. It is crucial to understand how the PMS prioritizes. By all means, avoid PMS that performs ranking since the common approach to ranking is to select the worst condition of the roads as the highest priority.

Instead, there are plenty of good PMS software provide near optimal solutions. The underpinning of cost effectiveness is to maximize benefits while minimize costs. The benefit or effectiveness, as illustrated in Figure 4, is defined as the area under the improved performance curve. To perform the benefit-cost analysis, the area is divided by cost to derive with the

benefit-cost ratio. Preventive maintenance, which is typically applied at good condition, will have the highest ratio when compared to reconstruction, when the condition is poor or failed. In the strict economic sense, PMS that emphasizes on preventive maintenance will have better condition over longer term, giving better return of investment as it costs less to maintain.

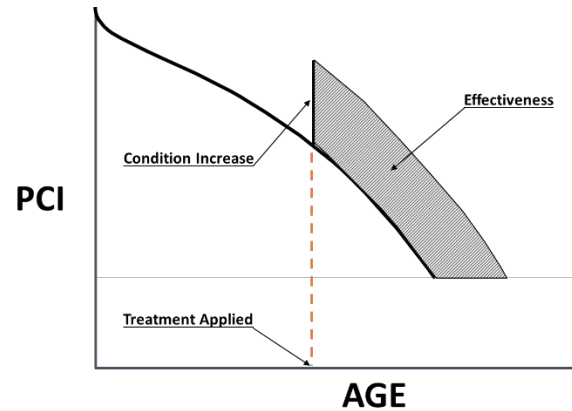


Figure 4. Illustration of cost effectiveness

In summary, the key takeaways are:

- Select a PMS that has a strong emphasis on preventive maintenance
- Avoid PMS that relies on ranking that is based on priority. Instead, focus on cost-effectiveness approach.

Step 5: Investment Analysis

At its most basic level, a PMS software should be able to answer questions such as:

- What is the existing overall pavement condition?
- How much maintenance dollars are currently invested?
- How much investment is needed to achieve the state of good repair

What set apart an effective PMS software from an ordinary one is the robustness in investment analysis. Some of the commonly use of pavement management results are (AASHTO 2012):

- Assessing maintenance needs for local, regional, and statewide (<http://SaveCaliforniaStreets.org>)
- Illustrating the impact of funding alternatives
- Spatially display the results in GIS maps
- Identifying candidate projects for maintenance and rehabilitation
- Setting performance-based funding allocation (Romell and Tan 2010)
- Setting performance targets (Tan 2015)
- Long-term planning for sales tax measures and regional transportation plan of metropolitan planning organization (MPO) (Plan Bay Area 2040)

- Sharing regional pavement condition for accountability and transparency (The Pothole Report 2011)

In the United States, since the last transportation act enacted in 2012, there is an increasingly focus of performance management on pavement and bridge assets. All state highway agencies and MPOs are required to set performance targets for these assets. Table 1 illustrates the use of various key performance indicators by MTC in the San Francisco Bay Area to monitor the effort on pavement preservation (Pavement Preservation Index), asset sustainability (which is also known as Investment Ratio), and the amount of backlog as compared to the asset value (Tan 2015).

Key Performance Indicators													
County	Jurisdiction	Current Conditions		Pavement Preservation Performance				Maintenance and Rehabilitation Investment Level					
		Network PCI	Network RSL	A	B	A/B	Pavement Preservation Index	C	D	C/D	E	F	E/F
				\$PM/Lane Mile	% Actual PM	% PM Needs		Actual M&R /Lane Mile	Needs/ Lane Mile	Investment Ratio	Current Backlog (millions)	Network Asset Value (millions)	Backlog/ Asset Value
	Regional Benchmarks	66	18.1	\$ 1,336	17%	16%	1.06	\$10,400	\$27,000	39%	\$5,645	\$38,814	15%
Alameda	ALAMEDA	66	17.7	\$ 1,271	13%	15%	0.88	\$9,800	\$26,900	36%	\$32	\$229	14%
	ALAMEDA COUNTY	71	19.9	\$ 671	18%	28%	0.67	\$3,600	\$16,200	22%	\$55	\$647	8%
	ALBANY	58	14.0	\$ 1,247	10%	13%	0.78	\$12,700	\$29,800	43%	\$9	\$41	22%
	BERKELEY	58	16.2	\$ 263	2%	11%	0.20	\$11,600	\$32,400	36%	\$77	\$298	26%
	DUBLIN	87	28.5	\$ 3,124	50%	79%	0.62	\$6,300	\$5,600	113%	\$4	\$180	2%

Table 1: Key Performance Indicators by local jurisdiction (Tan 2015)

Some key takeaways:

- Look for PMS software that provides easy to run and yet robust analysis
- Time savings in conducting analysis can be refocused on educating the decision makers

Step 6: Feedback

An effective PMS requires a long-term commitment to an agency’s pavement management program. This includes commitments to personnel resources to manage and upkeep the program, budgets for condition assessment, and ultimately sustainable investment for pavement needs. The upkeep, which completes an effective PMS framework, is the feedback loop to an active pavement management program. Unfortunately, this is also one of the steps, along with condition assessment, that will be eliminated when budget is tight. When this happens, the database is not kept up to date. This will result in erroneous prediction of pavement condition, inaccurate M&R work plan, and misguided investment plan. Hence we keep hearing the plight of “I don’t trust the data”.

Understanding that underlying a PMS software is a work-in-progress database, it is paramount to keep the database current. Here are some of the tasks need to act upon:

- Verify street segment inventory is accurate, such as functional class, surface type, length, width.

- Review decision tree periodically
- Update treatment unit costs annually
- Reassess data quality management plan periodically

Conclusion and Recommendations

It is true that a PMS software varies significantly in cost, complexity, and flexibility. However, when it comes to implementing a PMS, it does not have to break the bank. The six critical steps discussed in this paper provide a roadmap to manage a PMS efficiently. Managing a PMS is a long-term commitment; it is vital for an agency to identify PMS champion early. Pavement managers have to evaluate the agency's needs and goals. They also have to weigh in the level of staff support – IT, GIS, planning, maintenance, and engineering. By understanding the six critical business processes, they will be able to manage a PMS effectively, and ultimately attain buy-in from key holders that provide sustainable funding to meet pavement needs.

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**Establishing Optimal Long Term Funding Allocation Systematic Approach
based on Network Needs & Availability of Funds**

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1 INTRODUCTION

Road Controlling Agencies (RCAs) around the world are facing continuous challenges sustaining network funds. Given the fluctuation of market forces and instability of economies around the world. Effective management of a roads network requires that levels are set at least sufficient to keep the core of road assets in a stable condition for long term. This requires that ongoing maintenance is funded, and that adequate provision is made for any strengthening works required. More than this minimum level will be required if the network is to be expanded or improved. Roads are usually funded through budget allocations determined as part of the annual government budgeting process.

Roads with different hierarchies and functional classifications may be managed by different road administrations. Sometimes they will have their own sources of funds (investments). For others, funding will come from the national sources. Particularly for roads of lower hierarchy, the provision of funds will often be shared between national and local / regional sources. It is common for central governments to fund all work on national or trunk roads. In all cases, mechanisms need to be in place for allocating and disbursing funds between the different administrations. They need to be simple, transparent and encourage consistency of standards between the different administrations.

2 RESEARCH SIGNIFICANCE & CONCEPT

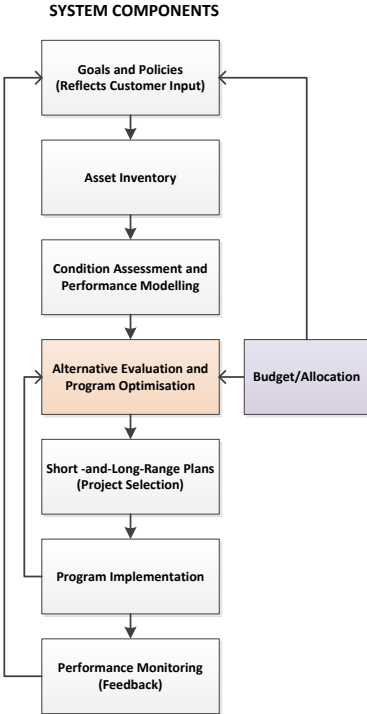
There is currently no method for the allocation of funding to road network management which considers the three key elements of deterioration, available funds and political influence. There is also no framework which includes a metric for the consequences of unscheduled influences. This research will advance the research into funding allocation and distribution research, and examine current practices with the intention to create a state of the art budgetary framework. This framework will adopt a new approach to the allocation of all resources, including financial, natural, manufactured, and human, within a budgetary framework which has a function of various operation and renewal scenarios. It is also intended that this research will improve the confidence of allocated funds.

The key element of this research study is to develop a model by multi-criteria analysis with a deterministic outcome and a model based on preferential analysis to determine optimality. The data will be summarised in templates and imported from various sources into the system tool, however data must be harmonised and synchronised to ensure compatibility with output files. This research will be unique in that it will provide the road network owner's viewpoint and be driven by value for money whilst meeting the fundamental principles of sustainability, innovation, and risk management.

3 RESEARCH HYPOTHESIS

The Alternatives Evaluation and Program Optimisation to Budget Allocation has been considered by this research and is demonstrated in Figure 1. In this research the budgetary components will be linked to the overall network performance based on Forward Works programme efficiency and annual depreciation of assets consumption. This will represent the consumption of assets value over the useful life versus the political influence to reduce funds on a basis of a lump sum allocation. The proposed system will provide various allocation components broken down into categories. This will include allocations for operation or renewals based on asset functional classification and required level of service, or summarised per route or a corridor, and then aggregated for the rest of the network. Hence different scenarios can be developed as opposed to a single allocated lump sum; Transport Association of Canada (2013).

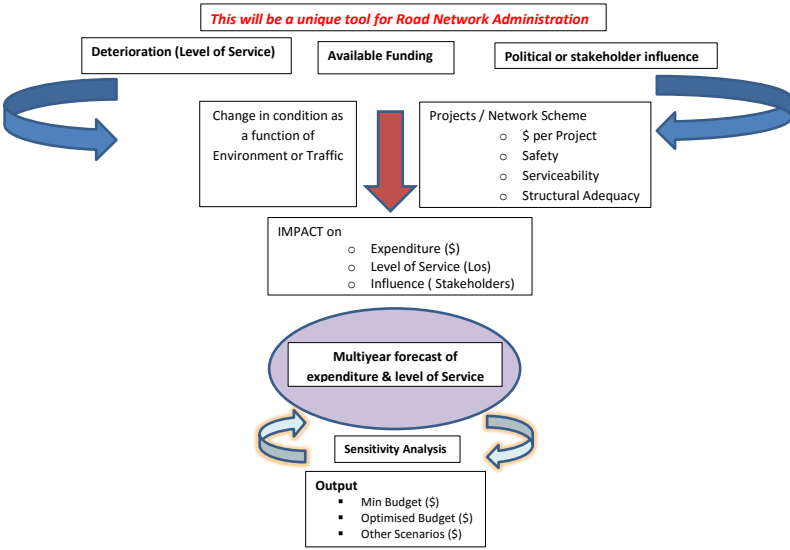
Figure 1: Alternatives Evaluation and Programme Optimisation



Preliminary research has identified the need to create a tool which is suitable for all levels of road administration. The hypothesis is that this tool will significantly affect a ‘planning authorities’ ability to evaluate the impact of changes to budget, level or service, or a combination of these factors. The second hypothesis is that this tool requires both qualitative and quantitative measures to be incorporated, including the cost and level of service of a road network for a multi-year plan, and the functions of deterioration, budget and political influence.

Figure 2 illustrates this idea and considers the “Alternatives Evaluation and Programme Optimisation to Budget Allocation” concepts. This figure has been developed by the researcher as part of the research. This diagram illustrates that the budgetary components will be linked to overall network performance based on Forward Works programme efficiency and an annual depreciation of assets model. In combination, this represents the consumption of asset values over the useful life cycle versus the political influence to reduce funds on a basis of a lump sum allocation.

Figure 2: Proposed Alternative Evaluation and Programme Optimisation



The research into the designing of this tool refers to the new Canadian Pavement Asset Design and Management Guide; TAC, (2013) and prioritises the Target Levels of Service which are expressed as an average condition of all pavement sections for a given network and for specific different roadway classes; a similar matrices of various asset types, roadway classifications will be developed using appropriate indices.

To meet the target network level of service, this research will also use Key Performance Indicators (KPIs). Table 2 illustrates a sample of KPIs recently developed as part of new Canadian Pavement Asset Design and Management guide; TAC, (2013). However there will be further roadway classifications necessary to account for motorways, expressway, and freeways. The table below also highlights the different levels of actual, desirable and minimum acceptable Level of Service (LoS).

Table 2: Canadian KPIs – (TAC 2013)

Roadway Type	Target or Desirable (Average PCI for all Sections)	Minimum Acceptable (Average PCI for all Sections)	Minimum Acceptable (PCI for individual Sections)
Arterial	80	65	55
Collector	70	60	45
Local	60	55	40

The funding authority can choose a level of service and see how KPIs reflect this level of service. In this way, the funding authority can perform a sensitivity analysis to assess how the budget impacted by different conditions and the optimal operation through a comparison of different budget scenarios generated by the proposed system.

It is expected, that this research, with the key output of a new system will be equipped with an audit trail for better management of road networks so that operations, maintenance and rehabilitation are minimised while the design and construction are maximised to full sustainability. Hence, the proposed alternative evaluation and program optimisation will be focusing on outcomes of delivery and improving the level of service, while maximising network safety resilient and sustainability. Table 3 illustrates a sample of Policy Objectives and Implementation Targets based on KPI’s recently developed as part of new Canadian Pavement Asset Design and Management guide; TAC, (2013). Accordingly, the research will be developing a similar matrix and more comprehensive to fit the local conditions for any authority that will demonstrate the different levels of actual, desirable and minimum acceptable (LoS).

Table 3: Policy Objectives and Implementation Targets Based on KPI’s (TAC 2013)

Policy Objectives	Key Performance Indicators	Implementation Target
Level of service	<ul style="list-style-type: none"> Network level of service (smoothness functionality and utilisation) network condition Provision of mobility (average travel speed by road class) 	<ul style="list-style-type: none"> Maintain 90% or greater of network in fair or better category (e.g. IRI ≤ 2) Rush hour traffic average speed minimum of 50% of posted speed limit
Safety	<ul style="list-style-type: none"> Accident reduction (percent) Bridges (% of number with reduced load postings) 	<ul style="list-style-type: none"> Reduction of fatalities and injuries by 1% or greater annually Number of reduced load postings to less than 5% of the network
Asset preservation	<ul style="list-style-type: none"> Asset value of road network (\$) 	<ul style="list-style-type: none"> Annual increase in written down replacement cost of 0.5% or greater
Sustainability	<ul style="list-style-type: none"> Recycling of reclaimed materials (asphalt concrete, etc.) - % Emissions levels 	<ul style="list-style-type: none"> Maintain at 90% or greater Maintain at levels < 90% of standards

The “Optimisation” is another key element of this research which will be based on multi – criteria analysis (MCA) and takes into account a range of criteria which are both qualitative and quantitative in nature and which reflect the economic, safety, serviceability and sustainability characteristics of the process outcomes. The proposed system will also involve development of cross asset scoring system which focus on higher level objectives to optimise budget allocation while improving level of service. Figure 3 below depicts the concept of broader optimisation, the system tool will incorporate the advanced state of the art concept of cross asset optimisation; NAMS, (2006).

Figure 3: Cross Asset Optimisation

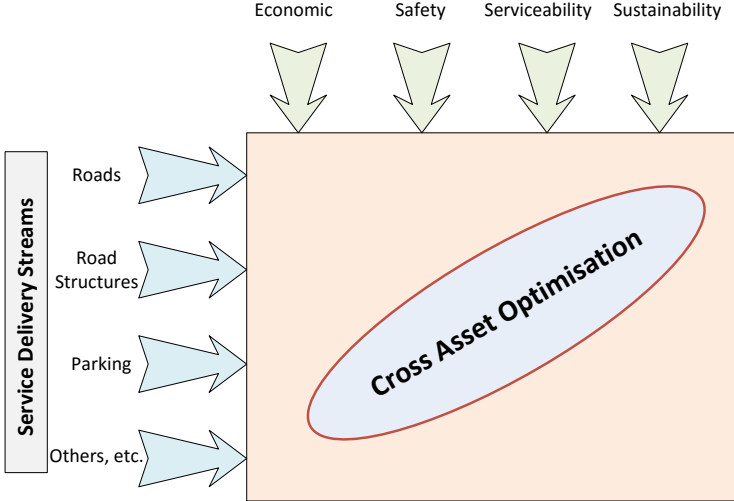
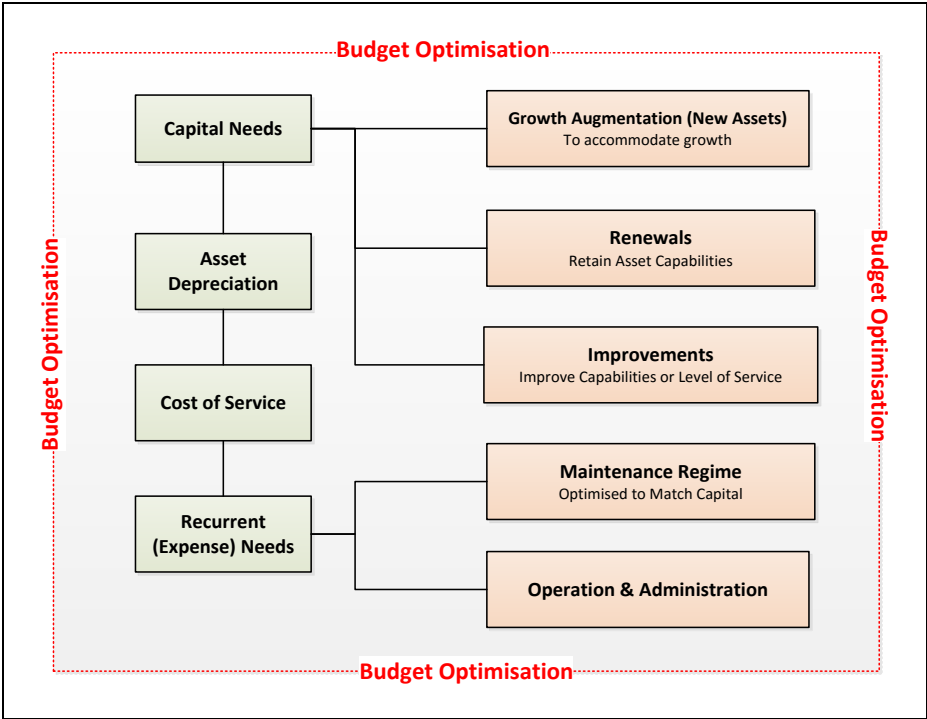


Figure 4 below illustrates the ‘Research Optimisation’ concept and depicts the overall concept of this research and also highlights the broader concepts of “Alternative Evaluation and Program Optimisation” for infrastructure assets. This concept considers the interaction between all network expenditures whether they are an integral component of operational or capital investment of asset.

Figure 4: Research Optimisation Concept



The proposed system will provide various allocation components and breakdown whether funds are needed for operation or renewals based on asset functional classification and required level of service.

Furthermore, this system also has the ability to summarise by route or a corridor, then be aggregated for the rest of the network. In this way, different scenarios can be developed as opposed to single allocated lump sums.

4 BUDGETING ADMINISTRATIVE FRAMEWORK

Modern budgeting systems were developed as a means of exerting legislative control over resource allocation decisions by the executive. This was achieved by dividing responsibility for and authority over the resource allocation process between institutions whose competencies and relations were defined in law, supplemented by exhaustive rules and procedures.

Ministry of Finance	Responsible for the management of public expenditure, including the formulation of a consolidated state budget and accounts, and the management of government's cash resources.
Spending Agencies	Responsible for the planning, management and delivery of public services, and the preparation and management of agency budget. Spending agencies are usually headed Ministers, occasionally by public officials.
Cabinet	Collectively formulates government policy. Implementation of government policy is the responsibility of individual Ministers. Cabinet approves the Government's budget.
Legislature	Analyses the Government's budget proposal and accounts, through the work of specialist committees, and enacts the budget in law. In Congressional systems, the legislature may amend the Government's budget proposal. In Parliamentary systems, it usually may not.
Auditor	Verifies compliance with the budget law and procedures regarding the use of public funds. The Auditor usually reports directly to the legislature, though in some cases may be considered part of the Ministry of Finance and report to Government through the Minister.

5 STRATEGIC BUDGETING

Road administrations are increasingly operating under a unified budget. Sometimes there is no distinction between "Capital and Operational" or any other budget categories. A lump sum budget is awarded and decisions on its expenditure between different types of works are made by the road administration itself on the basis of policy framework or needs; Robinson R, Danielson U and Snaith M (1998). Local budget categories may be used to assist in managing funds. While a unified budget will enable the needs of the network to be considered as a whole, the possibility of optimizing expenditures may not be optimal and could vary year by year. This research proposal focuses upon the breakdown of road budget categories, which will subsequently be used to determine a unified lump sum.

6 LINKING BUDGETS TO BUSINESS OBJECTIVES

When the revenues available to the road sector are significantly less than the amount required to maintain the road network in a stable long-term condition and to undertake justified improvements, the road agency should prepare an explicit long-term financing plan showing the size of the financing gap and the options for bridging the deficit. If a financing gap is identified changes to the financing plan should also include strategies to reduce the shortfall. The objectives need to consider the scope for increasing the revenues available to the road sector by simplifying road user taxes. Other possibilities include restructuring the taxes, reducing tax avoidance, evasion and leakage; Heggie, I G (2000). This research should assist to in closing the gap and creating a systematic process in a wide range of scenarios.

7 THE OBJECTIVES & OUTCOMES OF THE RESEARCH

This research will design and demonstrate the implementation of an optimal road funding system tool that will assist in predicting long term road infrastructure financial plans. The research will deliver a system tool which:

- Is suitable for an international marketplace (e.g. The World Bank, Investment Authorities) and be applicable to advanced and emerging economies;
- Is compatible for large and small infrastructure networks managed and funded by local, regional, and central governments;

- Can make a positive contribution to the technology for funding maintenance management of road networks within the road reserve;
- Is open, flexible and can be customised according to network characteristics. A multi-objective optimisation process capabilities will also be incorporated.

This move toward consistency requires the development of standard techniques and data systems, within the context of a fully integrated road and bridge management system. The system should be capable of accommodating the types of allocation and fund distribution currently required, including:

- Development of budget totals based on relating expenditures to changes in overall system performance;
- Development of regional distributions through the use of economic analysis that equitably;
- Compares the overall value of the investment by jurisdiction; and
- Development of functional distribution tools to calculate and compare changes in road user costs associated with various investment strategies.

8 RESEARCH LITERATURE REVIEW

It is important to differentiate between financing and funding. The term funding, as used in this research, refers to how infrastructure is paid for. Ultimately, there are only two sources of funding for infrastructure, government investment or direct user charges. This is opposed to financing, which refers to the way in which debt and/or equity is raised for the delivery and operation of an infrastructure project.

The budget process has two principal components:

- To decide how much funds are needed;
- To decide how to allocate the funds those are actually awarded.

Under such arrangements, therefore each authority competes for funds and at least in theory, funds are allocated to finance those expenditures with highest economic or social return, however such allocation are invariably highly politicized and allocations are often far from economically optimal. Politicians at national level are more likely to reflect general social needs than at the local level, where vested interests tend to have greater influence. Expenditures for maintenance, in all sectors inevitably lose out to higher profile capital investment projects, which contribute to the under funding of roading maintenance, as a result roading maintenance expenditures are often based on historical precedent; each year's budget is based on that of the previous year, with an additional allowance to cover inflation. This is a limited and poor basis for budgeting since it is arbitrary.

A better robust approach is for the budget application to be assessed on a rational assessment of economic need that relates to the objectives specified in the policy framework. One approach to the needs-based budgeting is for budgets to be based on life cycle costs, upgrading and reconstruction costs and road users costs over the life of the road by choosing the optimal level of maintenance. If roads are maintained too soon, then the full value of the existing pavement will not be attained and maintenance costs will higher than any reduction in operating costs, hence, total transportation costs are higher; conversely, if roads are maintained too late, the consequent maintenance will be more expensive or the value of asset may be lost. Therefore, the standards and intervention levels specified in the policy framework should reflect the need for maintenance. If there is to be consistent within the policy, the road maintenance budget, then the cost of the work needed to correct defects should be funded by the budget. Some Road Funds are committed to a fixed percentage allocation but this is not necessarily the best solution, and cannot always be affected anyway; Chan W T, Fwa T F, ASCE M and Tan J Y (2003).

A review of the funding allocation process used in a number of countries around the world has been undertaken, such as New Zealand, Australia, North America, Europe, Asia and Africa, it important to note, while this review may discuss how the government funding agency distributing the fund, the reality compiling a bid by the authority reflecting how detailed or a sophisticated process being used by the funding authority. Detailed literature review including references is available however due to limitation of paper submission, only a summary with key highlights are provided.

9 OPTIMAL PRACTICE WORLD WIDE – SUMMARY REVIEW

For efficient and consistent allocation of monies, priorities should be on the basis of economic cost-benefit principles, selecting those projects that demonstrate the highest economic rates of return. In practice the task is not that simple. Political interests may impose certain regional allocations, and rural roads need to be justified on the

basis of other criteria because their economic case will inevitably be weak. The process of allocating monies between types (trunk, district, urban and rural or feeder) is a continuing problem that has not been satisfactorily resolved. Some Road Funds are committed to a fixed percentage allocation, but this is not necessarily the best solution, and cannot always be affected anyway.

Resource allocation decisions are made unilaterally by the Ministry of Finance. In many cases, however, allocations are made in consultation with the Ministry of Transport. The consultation process can assist the Ministry of Finance in determining the appropriate total level of funding to be made available for road and bridge improvements. This determination must consider a total maintenance and rehabilitation requirements to support the desired level of overall system condition and performance for the country road and bridge systems.

10 BUDGET CATEGORIES

Budget Categories, the financial provision of roading network in many countries is divided into:

Capital (Capex) which relate to the construction of new roads, and sometimes the reconstruction, rehabilitation, strengthening and resealing or renewals of existing roads;

Recurrent (Operational) – the other category is a provision for the regular maintenance of the existing roading network, such as surfaces, off carriageway features, and for dealing with various contingencies, staff cost may also be paid under this category.

Sometimes the strengthening and renewals may be paid within the recurrent budget. In some roading agencies, the budget categories have breakdowns into more details such as a budget to maintain road structure, corridor maintenance, drainage facilities etc.

Where budgets awarded under different budget categories are usually less than bids for the roading agency, this means a lack of ability to vary funds from one budget category to another and will prevent the optimal allocation of resources under overall budget constraint.

11 ALLOCATION MECHANISMS

Roads with different hierarchies and functional classifications may be managed by different road administrations. Sometimes they will have their own sources of funds (investments). For others, funding will come from the national sources. Particularly for roads of lower hierarchy, the provision of funds will often be shared between national and local / regional sources. It is common for central governments to fund all work on national or trunk roads. In all cases, mechanisms need to be in place for allocating and disbursing funds between the different administrations. They need to be simple, transparent and encourage consistency of standards between the different administrations. Three basic methods are commonly used for:

- Simple Allocation Formula;
- Indirect Assessment of Needs;
- Direct Assessment of Needs.

Simple allocation formula assigns funds on the basis of pre-defined percentages to different parts of the network. Such allocation mechanisms are simple and transparent, but over time, are related only weakly to current need or use of the network.

Indirect assessment of needs; is used where there are no reliable data for measuring need directly or where the cost of doing so would be disproportionate to the size of the budget being allocated. This method is used mostly for the allocation of budget to low cost/low volume roads, criteria used in the indirect assessment will include for example;

- The land area covered by the administration;
- Road density;
- Population;
- Industrial / agricultural production or potential.

These factors are weighted accordingly to their perceived importance, this approach provides a pragmatic basis for allocating funds in appropriate cases that is cost – effective, and acknowledges the socio-political aspects of the decision-making.

Direct assessment of needs; can be of different degrees of sophistication, as its most comprehensive, it will involve using the results of a detailed condition survey of all needs to determine work requirements. These are then costed to derive the budget requirement. This approach requires the use of treatment selection methods such as deterioration modelling.

Simpler methods; involve deriving norms for expenditure on roads in different hierarchies or of different surface types. Road lengths in each administration are simply multiplied by the relevant norms to give the budget allocation. Thus, there are several direct assessment methods. The methods chosen should suit the capacity of the level of government concerned. The key is to have clear objectives and then to appraise in a systematic way the extent to which each intervention contributes to realizing these objectives. It is often difficult to change existing allocation methods because there will be strong resistance from those who will lose out "political interest". There may also be pressures to maintain a "regional balance" that may actually distort the optimal allocation of funds.

12 PRACTICAL BUDGETING TECHNIQUES

The purpose of resource allocation is to determine the appropriate total level of capital and maintenance investment that is to be made available for road repair and rehabilitation, bridge reconstruction and rehabilitation, and new construction, usually on an annual basis. Distribution is the manner in which total funds allocated for highway and bridge repair are made available to sub-national jurisdictions, road systems, and types of improvement.

Six identifiable patterns of resource allocation can be identified. The defining characteristic of these six patterns is the degree of shared responsibility between the Ministries of Finance and Transport (or their equivalents) in the allocation process.

In the first pattern, the responsibility for allocation, especially for the national road system, is retained in governmental hands. For example, the Ministry of Transport in Canada is totally responsible for resource allocation on the national road network. In Great Britain, the Department of Transport has the responsibility on a central as well as on a regional level. The procedure for resource allocation in the United States is very much reflected by the interaction of responsibilities between the Federal and State levels. In New Zealand, the government funding agency (NZTA) taken a leading role in allocating and distributing funds.

In the second pattern governmental jurisdictions are still in charge of allocation, while the distribution procedures are transferred to national, regional, and local road administrations. Germany, Japan, Norway, Portugal, Spain, and Switzerland belong to this allocation/distribution category.

In the third pattern, autonomous bodies are involved. This describes the allocation/distribution process in Italy, with the Autonomous State Roads Administration (ANAS).

Finland and Sweden represent the fourth discrete pattern. Although the financial responsibility remains in the hands of the government, the Road Administrations have a strong impact. This is consistent with the "management by objectives" philosophy that these countries have adopted.

The fifth pattern related to wealthy governments (economy based on oil & gas revenue), the political decision basically controls the funding allocation and distribution.

The sixth pattern, developing countries, the overall planning process appears to be inconsistent and very far from optimal in meeting some sensible framework of objectives.

Typically, the Central Government generally defines the total annual roadway rehabilitation and maintenance budget. In addition to the initial budget allocation, the central government may also determine the distribution of those funds by governmental jurisdiction, road system and, in some cases, by major category of road improvement type, although the involvement of the central road authority varies by country.

Decisions are made using a combination of technical analysis to achieve efficiency in fund allocation, and political, social, technical and economic considerations to achieve funding equity and balance among competing interests and political jurisdictions. This combination of technical and political considerations appears to exist in some fashion. As was true with the degree of central government involvement in the allocation process, however, the variations among countries in the relative mix of technical and political considerations are broad.

Under the political level(s) the managing responsibilities of road administrations differ between countries. Some countries use "management by objectives", "directed autonomy", or "zero based budgeting" philosophies in carrying out their responsibilities; other countries are more directly tied to the Ministry of Transport which permits only "limited autonomy" to their road administrations.

13 PROPOSED FRAMEWORK

The conceptual framework of the proposed system is outlined below:

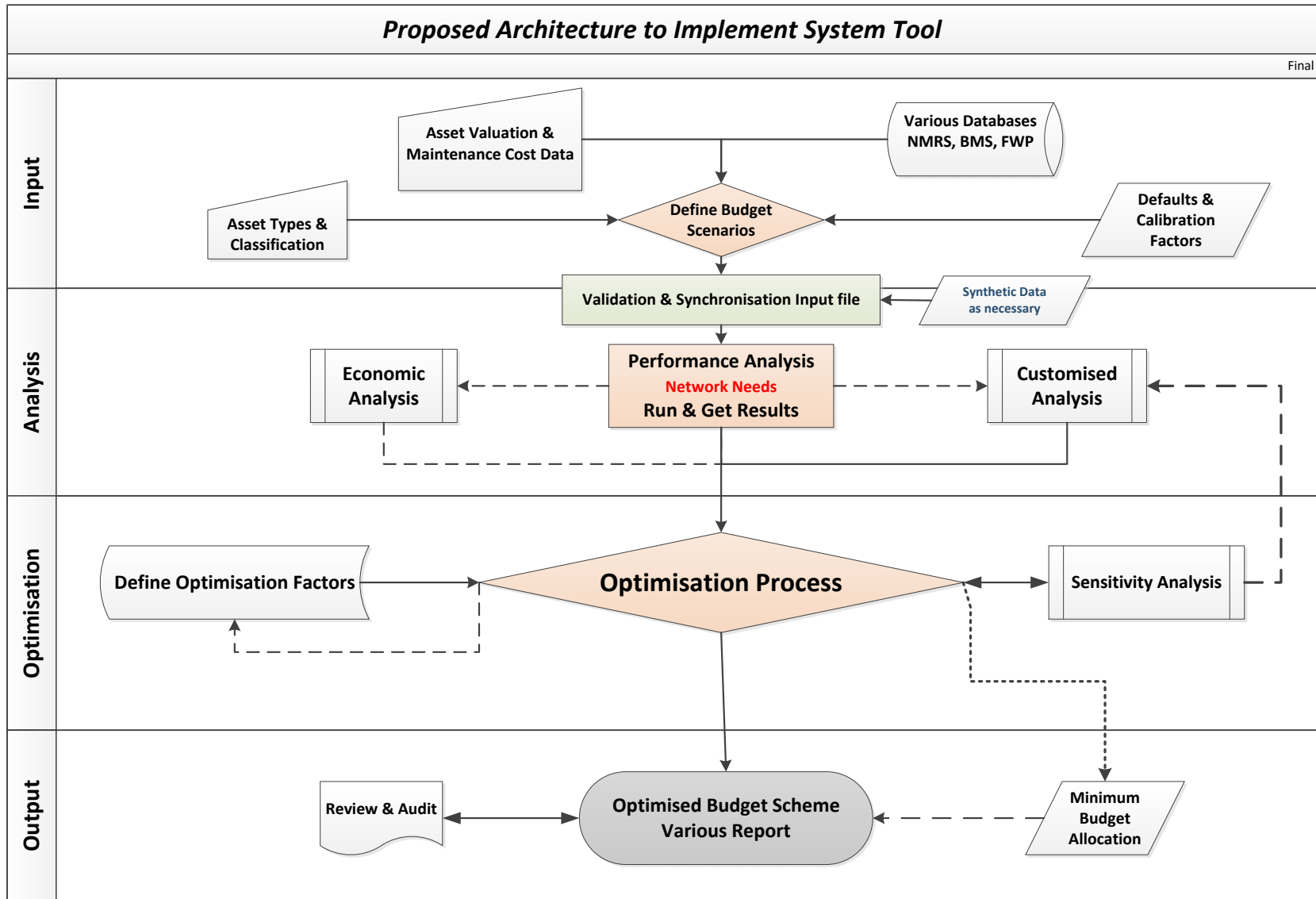
Process	Function Requirements
<i>Input Data</i>	<ul style="list-style-type: none"> • Define asset data type • Network condition prediction data & • Forward works programmes • Asset valuation & economic efficiency determination • Maintenance cost data • Annual budget considering split (Capex vs Opex) • Work categories definitions, classification of activities • Develop and prepare integrated input file
<i>System Matrices Setup & Global Values</i>	<ul style="list-style-type: none"> • Key analysis factors • Infrastructure Development Index • Budget configuration; operation, maintenance, improvement, new and reconstruction • Calibration of analysis matrix • Infrastructure gaps • Funding assistance rate • Development of Risk Matrix values & risk profile graph • Project implementation performance
<i>Analysis & Budget Scenarios</i>	<ul style="list-style-type: none"> • Validation of input data (out of range) • Performance-Based Allocation Formula • Economic and customised analysis • Aggregate data into bands • Budget Categories, Approved with Condition, committed allocation • Summarisation of maintenance cost • Perform scenario analysis • Programme review and variations
<i>Optimisation</i>	<ul style="list-style-type: none"> • Definition of critical optimisation factors such as strategic fit, the effectiveness and the economic efficiency • Budget distribution per category • Alignment of infrastructure projects • Budget scenarios vs. condition • Optimisation vs. committed programme
<i>Outputs</i>	<ul style="list-style-type: none"> • Final outputs (multi scenarios) • Multi-year forecast of expenditure by activity class • Minimum allocation • Reporting (default, customised, summary, per asset type) • Capex vs. Opex distribution • Optimal scenarios • Audit (review allocated vs. actual resources) • Category review • Trends (increased / decreased) and future spikes

14 SOFTWARE PLATFORM

The research study and the development of the software tool will advance simultaneously as part of the research project. The research also intends to carry out a pilot testing to ensure a smooth implementation.

System Architecture

The system architecture is depicted in the following chart.



15 THE WAY FORWARD

The following screen shots illustrate the front end of the system that is currently under development. This front end of the system reflects the research findings to date. The system platform will be based in various data sources such as valuation, predictive modelling, long term & annual plans together with other source of data. More details will be presented during the conference as the study is progressing and further outcomes are achieved.



Data Output

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Optimal Funding Allocation

View Analysis Input Data Summary

View Budget Scenarios vs Condition

View Budget Distribution

Back

Version 1.2

Alan Roland
January 2014

Data Audit

University of the Sunshine Coast
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Optimal Funding Allocation

Review Allocated vs Actual Resources

Review Capex vs Opex

Review Categories

View Audit Summary

Back

Version 1.2

Alan Roland
January 2014

System Settings and Definitions

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Optimal Funding Allocation

Define Asset Data Types

Data Type	Data Group	Code	Description	DispSe	Included
AssetType	OtherInfrastructure	WaterNetwork	Water Network	4	<input type="checkbox"/>
AssetType	OtherInfrastructure	WasteWaterNetwork	Waste Water Network	5	<input type="checkbox"/>
AssetType	OtherInfrastructure	UtilityNetwork	Utility Network	6	<input type="checkbox"/>
AssetType	RoadInfrastructure	RoadNetwork	Road Networks	1	<input checked="" type="checkbox"/>
AssetType	RoadInfrastructure	RoadStructure	Road Structures	2	<input checked="" type="checkbox"/>
AssetType	RoadInfrastructure	DrainageStructure	Drainage Structures	3	<input checked="" type="checkbox"/>

Record: 14 1 of 6 Filtered Search

System Settings and Definitions

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Optimal Funding Allocation

Define Asset Data Types

Data Type	Data Group	Code	Description	DispSe	Included
AssetType	OtherInfrastructure	WaterNetwork	Water Network	4	<input type="checkbox"/>
AssetType	OtherInfrastructure	WasteWaterNetwork	Waste Water Network	5	<input type="checkbox"/>
AssetType	OtherInfrastructure	UtilityNetwork	Utility Network	6	<input type="checkbox"/>
AssetType	RoadInfrastructure	RoadNetwork	Road Networks	1	<input checked="" type="checkbox"/>
AssetType	RoadInfrastructure	RoadStructure	Road Structures	2	<input checked="" type="checkbox"/>
AssetType	RoadInfrastructure	DrainageStructure	Drainage Structures	3	<input checked="" type="checkbox"/>

Record: 14 1 of 6 Filtered Search

System Settings and Definitions

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Optimal Funding Allocation

Define Asset Data Types

Define Analysis Factors (Matrix)

Data Type	Data Group	Code	Description	DispSe	Included
AnalysisFac	AreaType	Urban	Urban Area	1	<input checked="" type="checkbox"/>
AnalysisFac	AreaType	Rural	Rural Area	2	<input checked="" type="checkbox"/>
AnalysisFac	AreaType	CBD	Centra Business District	3	<input checked="" type="checkbox"/>
AnalysisFac	AreaType	Other	Other Suburbs	4	<input checked="" type="checkbox"/>
AnalysisFac	AssetUsage	Tourist		1	<input checked="" type="checkbox"/>
AnalysisFac	AssetUsage	Industrial		2	<input checked="" type="checkbox"/>
AnalysisFac	AssetUsage	Commercial		3	<input checked="" type="checkbox"/>
AnalysisFac	AssetUsage	Residentia		4	<input checked="" type="checkbox"/>
AnalysisFac	Authority	Federal		1	<input checked="" type="checkbox"/>
AnalysisFac	Authority	Regional		2	<input type="checkbox"/>
AnalysisFac	Authority	Local		3	<input checked="" type="checkbox"/>
AnalysisFac	Authority	DoT		4	<input checked="" type="checkbox"/>
AnalysisFac	Authority	Other		5	<input checked="" type="checkbox"/>
AnalysisFac	DiscountFactor	0		1	<input checked="" type="checkbox"/>
AnalysisFac	DiscountFactor	5		2	<input type="checkbox"/>
AnalysisFac	DiscountFactor	10		3	<input checked="" type="checkbox"/>
AnalysisFac	DiscountFactor	15		4	<input type="checkbox"/>
AnalysisFac	DiscountFactor	20		5	<input type="checkbox"/>
AnalysisFac	HeavyVehiclePercent	0		1	<input checked="" type="checkbox"/>
AnalysisFac	HeavyVehiclePercent	3		2	<input checked="" type="checkbox"/>
AnalysisFac	HeavyVehiclePercent	6		3	<input checked="" type="checkbox"/>
AnalysisFac	HeavyVehiclePercent	10		4	<input checked="" type="checkbox"/>
AnalysisFac	HeavyVehiclePercent	20+		5	<input checked="" type="checkbox"/>
AnalysisFac	PoliticalProfile	Low		1	<input type="checkbox"/>
AnalysisFac	PoliticalProfile	Medium		2	<input type="checkbox"/>
AnalysisFac	PoliticalProfile	High		3	<input type="checkbox"/>
AnalysisFac	SafetyLevel	Low		1	<input checked="" type="checkbox"/>
AnalysisFac	SafetyLevel	Medium		2	<input checked="" type="checkbox"/>
AnalysisFac	SafetyLevel	High		3	<input checked="" type="checkbox"/>
AnalysisFac	Terrain	Flat		1	<input checked="" type="checkbox"/>
AnalysisFac	Terrain	Rolling		2	<input checked="" type="checkbox"/>
AnalysisFac	Terrain	Hilly		3	<input checked="" type="checkbox"/>
AnalysisFac	Zone	1		1	<input checked="" type="checkbox"/>
AnalysisFac	Zone	2		2	<input checked="" type="checkbox"/>
AnalysisFac	Zone	3		3	<input checked="" type="checkbox"/>

Record: 14 1 of 35 Filtered Search

16 ACKNOWLEDGEMENTS

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Alan has been working with government agencies, consultants and contractors, and currently is working with the Department of Transport in Abu Dhabi, as the Network Inventory and RAMS Specialist. Alan is currently a PhD candidate working on this research.

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KEYWORDS:

Pavement, Structural Design, Flexible Pavement, Mechanistic-Empirical Pavement Design, Pavement Layer Thickness.

ABSTRACT:

Highways in the Kingdom of Saudi Arabia (KSA) have developed in the last forty years to a state of the art roadway network. Owing to the fact that KSA is a large country with an area exceeding 2.2 million km² comprising various topographical and geological conditions, several temperature zones and a wide range of traffic loading spectrum; Several highway functional and structural problems have been observed. The major reported defects include permanent deformation in the form of rutting and cracking in the form of fatigue and temperature cracking. KSA is currently implementing a national research study aiming to create or adopt a complete structural pavement design system to satisfy KSA existing conditions including material properties, roadbed characteristics, climate, and traffic loads; the research is under progress. This paper will summarize the proposed research plan and summarize the progress so far.

Structural Pavement Design for Saudi Arabian Roads: Outline of the Proposed Research

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1.0 Introduction

Highways in the Kingdom of Saudi Arabia (KSA) have developed in the last 40 years to a state of the art roadway network. The main objective of Saudi Ministry of Transport (MOT), is to build safe and economical successful pavements without major functional or structural defects for the design life of the road. However, owing to the fact that KSA is a large country with an area exceeding 2.2 million square km comprising various topographical and geological conditions, several temperature zones and a wide range of traffic loading spectrum several functional and structural problems in the roads have been observed. The major reported defects include permanent deformation in the form of rutting and cracking in the form of fatigue and temperature cracking. The major factors that are affecting the behavior of highways include materials, temperature, traffic loading and both mix and pavement designs. KSA government through King Abdulaziz City for Science and Technology (KACST) has launched a two years research project to create or adopt a complete structural pavement design system to satisfy KSA existing conditions including material properties, roadbed characteristics, climate, and traffic loads. This paper summarizes the research plan and methodology.

2.0 Literature Review

Pavement structural design is a complex process since it must balance selection of a wide range of loads, materials, layer thickness combinations and climate in one hand and performance with cost on the other. Traffic loading is a varied mix of vehicles, axle types, and axle loads with distributions that vary with time throughout the day, from season to season, and over the pavement design life. Pavement materials respond to these loads in complex ways influenced by stress state and magnitude, temperature, moisture, time, loading rate, and other factors. It should be no wonder, then, that the profession has resorted to largely empirical methods like the American Association of State Highway and Transportation Officials (AASHTO) guides for pavement design (AASHTO, 1993). Several developments over recent decades have offered an opportunity for more rational and precise pavement design procedures.

2.1 Review of Flexible Pavement Design Principles

Before the 1920s, pavement design consisted basically of defining thicknesses of layered materials that would provide strength and protection to a soft, weak subgrade. Pavements were designed against subgrade shear failure. Performance became the focus point of pavement designs. Methods based on serviceability were developed based on test track experiments. New design criteria were required to incorporate such failure mechanisms (e.g., fatigue cracking and permanent deformation). The mechanistic design approach is based on the theories of mechanics and relates pavement structural behavior and performance to traffic loading and environmental influences.

2.2 Empirical Methods

An empirical design approach is one that is based solely on the results of experiments or experience. The first empirical method used for flexible pavement design date to the mid-1920s when the first soil classification was developed. In 1929, the California Highway Department developed a method using the California Bearing Ratio (CBR) strength test. The CBR method related the material's CBR value to the required thickness to provide protection against subgrade shear failure. The CBR method was improved by U.S. Corps of Engineers (USCE) during the World War II and later became the most popular design method. In 1945 the Highway Research Board (HRB) modified the PR classification. The classification was applied to estimate the subbase quality and total pavement thicknesses. Several methods based on subgrade shear failure criteria were developed after the CBR method.

A few design methods were developed based on the theory of elasticity for soil mass. The first one published was developed by the Kansas State Highway Commission, in 1947. Later in 1953, the U.S. Navy applied Burmister's two-layer elastic theory and limited the surface deflection to 6.35 mm. Other methods were developed over the years, incorporating strength tests. More recently, resilient modulus has been used to establish relationships between the strength and deflection limits for determining thicknesses of new pavement structures and overlays. After 1950, experimental tracks started to be used for gathering pavement performance data. Regression models were developed linking the performance data to design inputs. The empirical AASHTO method (AASHTO, 1993) is the most widely used pavement design method today.

2.3 Mechanistic-Empirical Methods

Mechanistic-empirical (M-E) methods represent one step forward from empirical methods. The induced state of stress and strain in a pavement structure due to traffic loading and environmental conditions is predicted using theory of mechanics. Empirical models link these structural responses to distress predictions. Several studies over the past fifteen years have advanced mechanistic-empirical techniques. Main issues of interest include:

- Characterization of input parameters such as traffic loading, layer material and subgrade foundation properties, climate, and other design features.
- Sensitivity of performance models to specific inputs.
- Validation and calibration of the pavement distress prediction models (USA based to KSA conditions).
- Pavement distress types of interest for the various pavement types were; thermal cracking; load related (fatigue) cracking; ride quality (IRI); and rut depth in bituminous layers.

3.0 Research Objectives

The main objective of the proposed study is to develop a complete pavement design system (Manual) for Saudi roads that would take into consideration changes of the materials along with prevailing environmental and traffic loading in various regions of KSA; this will be achieved through:

- (1) Identifying the design governing factors prevailing in Saudi regions in terms of materials, temperature and traffic loading.
- (2) Evaluating the current practice of pavement structural design of Saudi expressways including design governing factors.
- (3) Studying and evaluating the latest pavement design techniques to recommend the most suitable technique to the local Saudi conditions.
- (4) Setting guidelines for pavement structural design including recommendations for suitable thickness of the bituminous layers and design alternatives to satisfy KSA conditions in terms of traffic, environment, and materials characteristics.

The findings and recommendations of this research are of great importance to KSA, since billions of Riyals are being spent to build roads all over the country. On the other hand premature failure is evident in some Saudi roads which could be attributed to the lack of a proper pavement design technique that is based on local environmental, traffic and materials conditions and limitations. It is of extreme

importance in this regard to develop or adopt pavement structural design that will provide an acceptable performance as well as a balance between initial and maintenance cost to optimize the life cycle cost of roads in KSA. All government and private parties working in the road industry in Saudi Arabia will benefit from this research.

4.0 Research Design and Methodology

This research will be conducted in the following Six main tasks:

- 1) Literature review.
- 2) Field work and distress surveys.
- 3) Laboratory work.
- 4) Pavement design techniques evaluation.
- 5) Review of Current Practice of Testing and Measurements.
- 6) Data analysis and recommendations.

Task-1: Literature Review:

Available literature will be reviewed and summarized for the benefit of the project. The research team will communicate with KACST to provide any literature review that was conducted in any previous efforts. However, there has been a lot of recent work in the USA for updating the MEPDG procedure such as inclusion of stabilized base layers in the analysis and modeling. Therefore the literature review will focus on literature published in the last five years. Literature review will cover:

1. Pavement design techniques in use by MOT and other related highway agencies in KSA.
2. Local and International pavement performance studies. This will include reviewing the study that was performed by Saudi Aramco in cooperation with the Department of Civil Engineering of King Fahd University of Petroleum and Minerals entitled “Testing and Evaluation of Sulfur Asphalt Concrete Mixes and Sulfur Asphalt Test Roads” in December 2009.
3. Pavement design governing factors will include, but not limited to, traffic, climate, material, distress prediction studies. Also, contact area, drainage, and groundwater as these are required inputs for the MEPDG procedure. Typical tires used in the KSA network will be documented and drainage/water table conditions will be listed and used as part of the procedure.
4. Local and International traffic loading studies will be reviewed to confirm that estimations in KSA (Traffic Classes) are sufficiently adequate and reliable for use in pavement design.
5. Review of pavement structures and mix design used in local, regional (UAE), and international highways (USA).
6. Current methods for the design and analysis of pavement structures and international literature on innovative asphalt concrete mix technologies.
7. The impact of using binder modification like polymers and sulfur (Sulfur Extended Asphalt) on the required pavement thickness.
8. The research team will review the best method to recover and test aging of binders (AASHTO T164 including methods of centrifuge extraction, reflux extraction, and vacuum extraction).

The findings from literature review and review of International practices will be summarized and used to identify best practices to be used in this project and make modifications to the work plan, if needed.

Task-2: Field Work and Distress Surveys:

The ultimate goal of this project is to evaluate the effect of pavement thickness on serviceability and durability of roads in terms of rutting, fatigue cracking, thermal cracking and ride quality (IRI). Therefore, MOT pavement management system will be utilized to identify existing pavement sections on some of Saudi expressway network in Central, Western, Northern and Eastern provinces of KSA. Sections will be identified to cover various geology, materials characteristics, climatic conditions, and

traffic loads. Selected sections will include good well-performing (control) and poor performing failed sections. Performance data that the MOT has will be reviewed to select such sections. Test sections will be selected with different materials including mixes with neat, polymer modified and sulfur extended (SEA) asphalt binder. Each test section will be approximately 1.0 km long and will be selected on an expressway carrying different traffic loads. A factorial experiment for flexible pavement distresses, material, layer thickness, traffic loading, environmental conditions, supporting layer and roadbed soil will be prepared to cover and provide treatment selections for the study.

Sections will be evaluated for pavement distresses such as rutting, cracking and riding quality. Central, western, northern and eastern provinces will be the main areas to be studied in this project; other areas and provinces will be considered as appropriate. In addition to evaluating the selected pavement sections for the cause of pavement distresses, they will be statistically compared to identify possible correlation between pavement thickness and those distresses. The collected distress data will also be used to calibrate the transfer functions and the material damage models in the MEPDG. Moreover, pavement sections will be sampled and tested to identify common pavement design parameters.

Task-3: Laboratory Work:

Material characteristics will be obtained from samples collected from the selected test sections across the kingdom and from new materials and mixes including SEA and other different possible combinations of materials that can be used in KSA. Asphalt mixtures will be tested for dynamic modulus, flow number and rutting resistance. Various variables including air voids, aggregate gradation, performance grade, and polymer type will be investigated. The subgrade and aggregate base course materials will be evaluated for compaction, strength and classification.

Task-4: Pavement Design Techniques Evaluation:

Local and International structural pavement design techniques will be reviewed and employed in this study including American Association of State Highway and Transportation Officials (AASHTO), Mechanistic-Empirical Pavement Design Guide (MEPDG) and other mechanistic methods.

The research Team will conduct a thorough review of the Mechanistic-Empirical Pavement Design Guide, including sensitivity analysis of each of the governing factors. Specifically, the review will indicate how the current calibration and validation of the guide (USA based) will be applicable to KSA conditions. As the MEPDG requires aging model of bitumen and since the conditions in KSA are very warm and are almost always sunny conditions, it is necessary to perform a study of oxidative aging trends in two locations of the Kingdom, one in the coastal region and one in the middle of the country. The study will include taking cores from pavements at various ages and slicing the cores to study the hardening of bitumen from various depths. This is required to calibrate models in the MEPDG to the conditions of KSA. Also the research will study variation of pavement temperature with depth of pavement. As asphalt layers are very good insulators and thus the top few centimeters are much warmer than the lower layers. Although there are models for some other countries such as the US and South Africa conditions, it is not known how the extreme summer conditions affect pavements in KSA. So it is required to study the variation of temperature with depth and adjust the models used in the MEPDG software if the research team finds them not representatives of the KSA conditions. The pavement temperature variation with depth study will be carried out at the same location of the study of oxidative aging.

Task-5: Review Current Practice of Testing and Measurements:

The research team will review the current road sample testing and quality control practice used by MOT and the Ministry of Municipal and Rural Affairs (MOMRA). This is required to assure that local laboratories of MOT and MOMRA can provide the required properties and input of the mechanistic pavement design that will be recommended by the research. Therefore, the team will check what measurements will be needed and based on that check the laboratories and suggest including all that are missing.

Task-6: Data Analysis and Recommendations:

Collected data and information will be analyzed to achieve the research objectives and give the final recommendations.

5.0 Project Status and Accomplishments

The project started January 2016 and has progressed about six months up to date. In this period the project accomplishments were as follows:

1. Establishment of the overall work breakdown structure (WBS) to identify all tasks and subtasks and build up the project plan.
2. Initial review of the pavement structural design governing factors prevailing in Saudi regions in terms of materials, temperature and traffic loading. The research plan was structured to take this into consideration. This is evident since the research plan will cover four regions and heavy trafficked roads in KSA. The Experimental design is shown in Figure 1. Pavement sections of expressway network in KSA were selected to start the field work, collection of samples and distress surveys.
3. The research team after discussion with MOT found that the current practice of pavement structural design of Saudi expressways is based on the fixed section approach, and no actual analysis and calculation is done for different projects in various regions of KSA to come up with the pavement structure based on project level traffic, materials and environment.
4. The research team has through review and discussion about the latest pavement design techniques locally, in the GCC and in different parts of the world. At this stage the research team and the consultants found that the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed in United States of America (USA) is one of the largest and recent efforts in the international know how. M-E design procedures “by far” can be classified as the state-of-the-art and the state-of-the-practice in pavement design. Several M-E procedures are available worldwide. However, the AASHTO developed M-E design procedure is considered one of the most sophisticated yet easier to adopt M-E procedures. Thus, implementing an M-E design procedure to develop pavement structural designs in Saudi Arabia is a step forward and strategic movement in the right direction for KSA Ministry of Transport. Therefore the research team recommended using it in Saudi Arabia after doing the necessary calibration and validation for local conditions.
5. Start of the laboratory work to build the local material library that is necessary for the successful implementation of the M-E pavement design.

		Distress				
		Rutting		Cracking		
		Traffic				
		Region	Low	High	Low	High
Thickness	Thin	E	X1	X2	X3	X4
		C	X5	X6	X7	X8
		W	X9	X10	X11	X12
		N	X13	X14	X15	X16
	thick	E	X17	X18	X19	X20
		C	X21	X22	X23	X24
		W	X25	X26	X27	X28
		N	X29	X30	X31	X32

- E = Eastern, C = Central, W = Western, N = Northern regions of Saudi Arabia.
- City Sections are: X1, X3, X5, X7, X9, X11, X13 and X15.
- Low Traffic less than 20 million ESALs (20 year).
- High Traffic more than 20 million ESALs (20 year).
- Thin Pavements (15 cm of asphalt layers or less). Thick Pavements (greater than 15 cm of asphalt layers).

Figure-1: Research Experimental Design

6.0 Acknowledgement

This research project was financially supported by King Abdulaziz City for Science and Technology (KACST) project no. OA-34-1, the research team is grateful to KACST for supporting this important research. Also the team is thankful to MOT and Saudi Aramco and all other agencies and individuals for their support and assistance provided to the research.

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KEYWORDS:

Rejuvenator, R.A.P, Tensile Strength Ratio, Deformation Strength Test, Dynamic Immersion Test.

ABSTRACT:

Recently, the usage of RAP(reclaimed asphalt pavement) has increased as concern with environmental effects. In order to use RAP for asphalt pavement construction, a rejuvenator is required to improve asphalt mixture properties. The research team has worked to develop warm-in place recycling asphalt pavement technology. For this research, the research team has developed a rejuvenator. To evaluate the developed rejuvenator, the research team has used two types of rejuvenators for this study. To compare performance of two rejuvenators, three test methods were adopted such as deformation strength test, tensile strength ratio, and dynamic immersion test.

Performance Evaluation of Developed rejuvenators for Warm-in place Recycling asphalt pavement

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1. INTRODUCTION

Recently, as environmental issues such as the greenhouse gas emission have been increasing globally, the pavement construction technology is studying to increase recycling of wasted materials in various ways. One of them, the reclaimed asphalt pavement (RAP) has been actively promoted in order to a cycle of reusing materials that optimizes the use of natural aggregate. Reclaimed asphalt pavement (RAP) is a useful alternative to virgin materials because it reduces the need to use natural aggregate, which is a scarce commodity in some areas of the Korea as well as best practices for increasing the use of RAP in asphalt pavement mixtures while maintaining high-quality pavement structures. It also saves the amount of money of new asphalt binder required in the production of asphalt paving mixtures. Currently, although the amount of recycled asphalt is increased annually in Korea, the amount of usage of RAP was not comparatively increased. The important issue is that rejuvenator cost is expansive because most of them were imported. Therefore, in this study we compared the RAP using developed rejuvenator for warm-mix. and verified the possibility for its application through of conducted the deformation strength test and tensile strength ratio test, dynamic immersion test.

2. MATERIAL

Asphalt mixture's become susceptible to cracking and fracture as the hardness increases due to over time (Lim et al., 2012). Rejuvenator is the material that makes the ageing asphalt binder restore its physical properties of the recycling asphalt mixture such as ductility and viscosity in order to maintaining existing aggregate gradation and binder contents. (Kim et al., 2007). Rejuvenators used in this study is the two types, type1 is developed with adding base oil for decrease harmful gas emissions and another type2 is current used in pavement market now.

3. EXPERIMENTAL METHODS

3.1 Mix design of R.A.P

In order to evaluate the performance of the two type's rejuvenator, we carried out mixing, changing the mixing ratio from 0 to 7.9%. Table 1 shows mix-design of one specimen for estimate to deformation strength and tensile strength. RAP and rejuvenator were blended at a temperature of 130°C, mixture was compacted by marshall compactor. Figure 1 shows two type of rejuvenator. Figure 2 is a reclaimed asphalt pavement extracted from wasted asphalt.

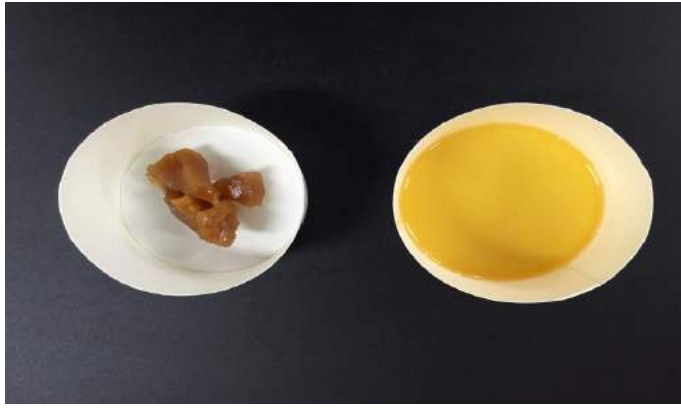


Figure 1. Conventional(L) and Developed(R) Rejuvenators



Figure 2. Reclaimed asphalt pavement

Table 1. Mix Design

Type	Recycling Aggregate		Content of Rejuvenator (%)
	Coarse Aggregate (g)	Fine Aggregate (g)	
A	600	600	0
B	600	600	2.4
C	600	600	5.1
D	600	600	7.9

3.2 Deformation Strength Test

Asphalt is generally viscosity-elastic behavior at ordinary temperature. Whereas at a high temperature it has appears the plastic behavior. Therefore analysis of deformation behavior under vehicle load is very difficult. Plastic deformation of the asphalt such as rutting is mainly generated in the high temperature condition. This rutting-resistance may be approximately estimated by measuring the deformation strength. Deformation strength of the RAP added Rejuvenator was conducted in accordance with the test method developed in domestic. After specimen with Diameter of 100 mm, a thickness of 62.5mm is positioned 60 degrees water bath for 30 minutes, remove the moisture on the surface and load at rate of 30mm/min and then measure the maximum load and displacement. Substituting the measured load and displacement in the following equation, finally the deformation strength was obtained(Byung-Jin Cho et al. 2008). Figure 3 is a sample of deformation strength test. Figure 4 is a jig for measuring the deformation strength.

$$S_D = \frac{0.32 P}{(10 + \sqrt{20y - y^2})^2}$$

where, S_D = Deformation strength (MPa)
 P = Load (N)
 y = Displacement (mm)



Figure 3. Specimen for deformation strength



Figure 4. Measurement jig for deformation strength

3.3 Tensile Strength Ratio

Generally, water is caused damage to the asphalt pavement and is the cause of reducing durability. Therefore in order to evaluate moisture-resistance of recycling asphalt mixture using the rejuvenator, this study was conducted by measuring the ratio of tensile strength between the water treated mixture and atmosphere treated mixture with KS F 2398. After classifying 6 specimens compacted by marshall compactor into 3 and curing in atmosphere and water at degree of 60°C for 24 hours, this experiment was carried out in the way measuring tensile strength. Figure 5, Figure 6 each shows Marshall compactor for making specimen, and Marshall stability tester. Tensile strength ratio(TSR) is obtained by substituting the tensile strength of each condition to the following equation.

$$S_t = 2 \frac{P}{\pi t D} \quad \text{where, } S_t = \text{Tensile strength (MPa)}$$

P = Load (N)
t = Thickness of specimen (mm)
D = Diameter of specimen (mm)

$$\text{TSR} = \left(\frac{S_{tm}}{S_{td}} \right) \times 100$$

where, TSR = Tensile strength ratio (%)
S_{td} = Tensile strength of curing in atmosphere (MPa)
S_{tm} = Tensile strength of curing in water (MPa)



Figure 5. Marshall automatic compactors



Figure 6. Digital stability testing machine

3.4 Dynamic Immersion Test

This experiment was performed to evaluate the resistance of the separation between the asphalt binder with the aggregate when the water is added to the asphalt mixture. The test method was to have asphalt mixture having a constant size (11mm~8mm) and distilled water rotated 60 times per minute for 24 hours in test bottle. After that, we measured loss amount of weight. Based on the results, scaling resistance of the rejuvenators was evaluated. Figure 7 shows the asphalt mixture and glass bottle including a glass rod and Figure 8 shows the experimental setup.



Figure 7. Sample for D.I.T and Experimental equipment

Figure 8. Experimental set-up

4. TEST RESULTS

4.1 Deformation Strength Test

Deformation strength test result for evaluate properties of the plastic is shown Fig 10. As shown in the graph, when using a constant ratio rejuvenator was the value exceeding the standardization of warm-mix asphalt pavement. However, without using rejuvenator, value of deformation strength was 6.9Mpa. This means that asphalt mixture harden with the passage of time, and rejuvenator acts to soften the binder.

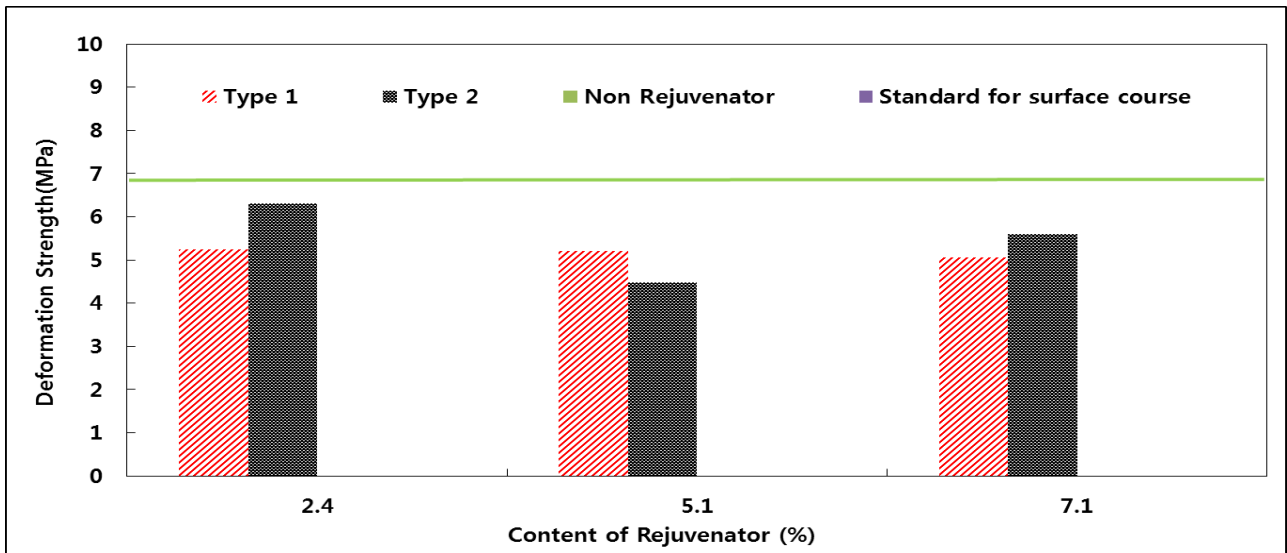


Figure 9. Deformation strength test's result graph

4.2 Tensile Strength Ratio

The result of tensile strength ratio (TSR) test to measure moisture sensitivity is shown in Figure 10. First, in the case of the mixture without addition of the rejuvenators, TSR value was 0.69. This value does not reach the domestic standardization of warm-mix asphalt pavement. However, when using a developed rejuvenator was the value exceeding the standard value with respect to all the addition ratio(2.4, 5.1, 7.1%). This means that the recycling asphalt mixture using developed rejuvenator has sufficient resistance to moisture.

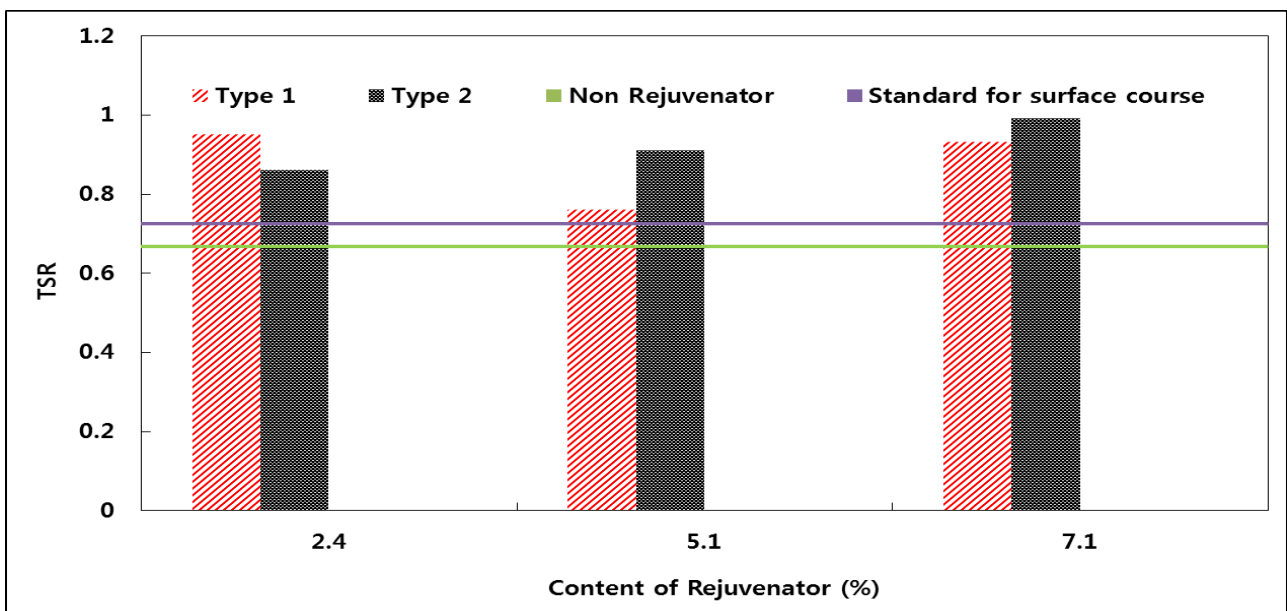


Figure 10. Tensile strength ratio's graph

4.3 Dynamic Immersion Test

The result of Dynamic immersion test to evaluate the scaling resistance of asphalt mixtures is shown in Figure 11. The experiment was conducted according to Europe standard(EN-12697-11). When using specimen without rejuvenator, on average, a loss of 10g is generated in the whole 150g. However, increases of rejuvenator addition rate to reduce the amount of loss caused by water. Moreover, the newly developed warm-mix rejuvenator has more excellent scaling resistance over water, compared with conventional rejuvenator. Figure 12 shows asphalt mixture before and after dynamic immersion test.

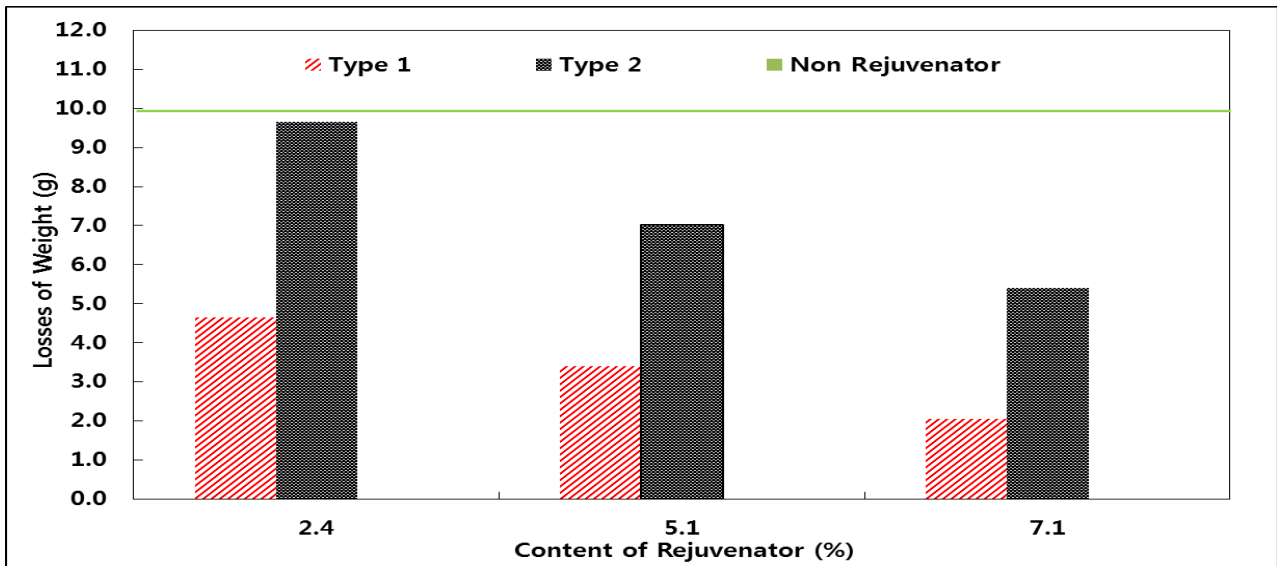


Figure 11. Losses of Weight (D.I.T)

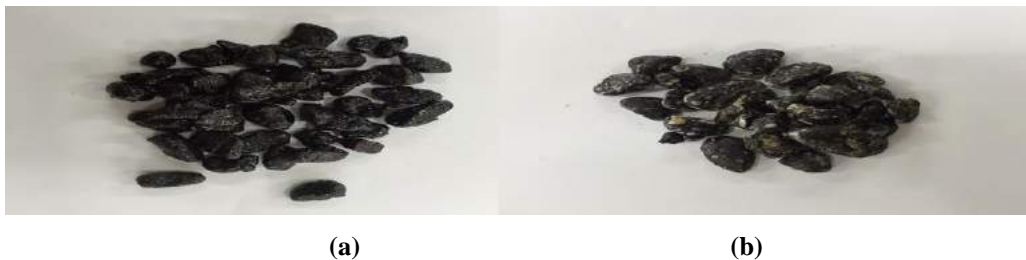


Figure 12. Asphalt admixture of according to experimental. ((a) before immersion, (b) after immersion)

5. CONCLUSION AND FUTURE RESEARCH WORKS

The following result is a performance to evaluate possibility of application over rejuvenator and propose solutions about environmental problem. The result of performance testing for recycling asphalt mixture on the indoor test is as follows.

1. In the result of deformation strength test, addition of rejuvenator on R.A.P had been derived softening of asphalt binder. According to this response, asphalt mixture's Deformation strength was decreased. This result represents improvement in the resistance of plastic deformation.
2. In TSR test result performed to evaluate moisture-resistance, in case of using rejuvenator, the TSR values were more than standardization for warm-mix asphalt pavement. This means that it is possible to improve the moisture sensitivity of the recycling asphalt pavement through rejuvenator.
3. In the result of dynamic immersion test for the scaling resistance evaluation, the resistance of scaling was

clearly improved in accordance with the increase in addition rate of rejuvenator and newly developed rejuvenator with polymer is superior to conventional rejuvenator over the resistance of scaling.

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PAPER TITLE	Maintenance of Infrastructures Using GIS and Point Cloud Data		
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KEYWORDS:

Maintenance, MMS, GIS, Point Cloud, Ledger

ABSTRACT:

In recent years, many of the infrastructures that were made in the high-growth period are aging, in Japan. At the same time, number of maintenance engineers for the infrastructures has been becoming insufficient. In order to solve these problems, we have developed a computer system to improve efficiency and accuracy of maintenance work on roads and structures, utilizing GIS (Geographic Information System) and point cloud data. Using this system, variety of data related to structures such as drawings, inspection results and repair or reinforcing records are managed through a digital map on the GIS. This system also has the ability to utilize the point cloud data measured by a MMS (Mobile Mapping System) for such as creation of 2-D and 3-D CAD data or FEM (Finite Element Method) models, and simulation of inspection or repair work. Moreover, inspection work can be supported by detecting displacement or damage of structures using point cloud data.

Maintenance of Infrastructures Using GIS and Point Cloud Data

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1 INTRODUCTION

In recent years, many of the infrastructures such as bridges and tunnels that were made in the high-growth period are aging, in Japan. At the same time, number of maintenance engineers for the infrastructures has been becoming insufficient.

In order to solve these problems, we have developed a computerized system named as “InfraDoctor” to improve efficiency and accuracy of maintenance work on roads and structures, utilizing the “Geographic Information System”, hereafter referred to as GIS, and point cloud data. Using this system, variety of data related to structures such as drawings, inspection results and repair or reinforcing records are managed through a digital map on the GIS. This system also has the ability to utilize the point cloud data measured by a MMS for such as creation of 2-D and 3-D CAD data or FEM (Finite Element Method) models, and simulation of inspection or repair work. Moreover, inspection work can be supported to be more efficient and accurate by detecting displacement or damage of structures using point cloud data.

In this paper, various functions of the “InfraDoctor” are introduced and a further development plan of total management system for infrastructure is shown.

2 OUTLINE OF “INFRADOCTOR”

The “InfraDoctor” has been developed in order to improve efficiency and accuracy of maintenance work on structures such as bridges, tunnels, buildings and so on, utilizing GIS and point cloud data. This computerized system consists of some elements listed and explained as follows.

2.1 GIS platform

The “InfraDoctor” has a GIS platform. As the portal of the system, a digital map is shown in the browser on the computer screen. Through this digital map of the GIS platform, any data on various ledgers can be retrieved easily and quickly.

The “maintenance ledger” has information concerning number, shape and material of structures and appendages. The “inspection history ledger” has inspection-result records. The “repair-history ledger” has records of repair and reinforcement work carried out based on the inspection results. Each data on those ledgers has coordinates and connected to the digital map. Those data also can be searched using various keywords such as name, type, material and so on, of the structures. Using other information connected to the searched data, they can be sorted easily and

quickly for various purposes, such as confirming the quantity and locations of same-type structures or parts, side-by-side comparison between inspection results or repair work history, for example.

The point cloud data and the all-around video data to be described later are also managed on the GIS platform and can be shown on the computer screen by specifying the location from the digital map on the browser.

2.2 All-around Video

The all-around video images are captured using a vehicle called “MMS” (see Picture 1), which stands for “Mobile Mapping System”, while travelling. The all-around video is recorded as still images at every 4m of interval, if the MMS is travelling at 60 kilometers per hour. The resolution of an image is about 12 mega pixels.

Using this all-around video, the condition of structures and their surroundings can be confirmed through computer screen without actual investigation at the site. Because the all-around video enables objects on a road to be seen from not only the direction that the MMS is moving ahead but also any direction from the point of MMS, the condition of the site can be grasped quickly and precisely at the office. The backside of a sign board can be looked back as seen in Figure 1, or sealing of a tollgate can be looked up, for instance.



Picture 1. MMS.



Figure 1. GIS browser with all-around video image.

2.3 Three-dimensional Point Cloud Data

The MMS mentioned above is installed with 2 laser scanners, and it captures three-dimensional point cloud data while travelling at an ordinary speed same as other vehicles. Those laser scanners emit laser beams a million times a second and capture the location of an object which reflects the laser beams from the scanner. A laser beam can usually reach to an object about 800 meters away from the MMS. The MMS also equipped with a GNSS (Global Navigation

Satellite System) antenna and an IMU (Inertial Measurement Unit), and the error of measured coordinates due to vibration or rotation of the vehicle are revised using those devices. In general, degree of error for coordinates of an object is about 10 centimeters for absolute value of location with respect to the surveying coordinate and a few millimeters for relative error regarding the position of each point recorded in one measurement, if the target is an expressway road bridge with the height of about 10m from the surface road. The degree of error, however, is changed depending on the distance between the target object and the laser scanner, velocity of the MMS, the condition of road surface which affects the vibration of MMS, tall buildings or structures which interrupts or reflects radio waves for GPS antenna from satellites, and so on.

When a structure is modelled in three dimension using point cloud data, the MMS should be driven several times around the target structure in order to eliminate its blind spot. Therefore, a task called “registration” is required to integrate those point cloud data groups measured separately. It has been confirmed that an error of a few tens of millimeters may occur during a registration work. In order to solve this problem, a program which can carry out the registration work while correcting the error automatically is now under development and being prepared for commercialization.

The MMS is also equipped with some HD (High Definition) digital cameras and the color data on the image captured by those cameras can be exported to each point in the point cloud. Because of this color, point cloud looks like a video image as seen in Figure 2 and this makes it easier and more precisely to recognize the shape and positional relationship of structures.

When the MMS cannot capture all the necessary point cloud of a target structure such as a bridge on a wide river or in steep mountainous area without roads available, a portable scanner fixed on a tripod, a human carrying type scanner or drone equipped with a scanner can substitute and the point cloud data captured by those different scanners can be integrated with that from the MMS.



Figure 2. GIS browser with point cloud data.

3 APPLICATIONS OF POINT CLOUD DATA

3.1 Detection of Damage and Displacement

Damage on a concrete surface such as flaking or delamination can be detected using the point cloud, by measuring the difference of position between measured points and a “standard surface” which is an average plane of the concrete surface created from the point cloud data. This method is expected as a remote inspection technique for those damages where the close visual inspection is difficult and spalled concrete might cause damage on people or cars under the structure such as bridges on an intersection with heavy traffic or railroads.

In a trial measurement, some flakes on a concrete surface with 3 or 4 millimeters of protrusion were detected when measured by a scanner on the MMS about 10 meters away (See Figure 3). In this trial, point cloud data measured in one measurement was used in order to eliminate the error caused by the registration work. More study is needed to acquire higher precision with this inspection method.

It is possible to find displacement, deformation or lack of the appendages, structural members or the structure itself when subjected to a large external force, such as a large earthquake or tsunami, by comparing a point cloud measured after the event with the one formerly recorded.

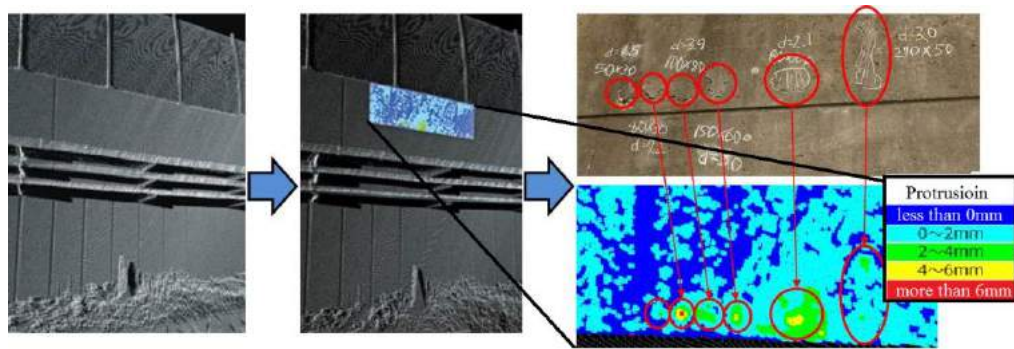


Figure 3. Detection of damage on concrete surface.

3.2 Three Dimensional Measurement

It is possible to measure any size or distance of structures in cooperation with point cloud data and the two-dimensional map, on the GIS browser of the “InfraDoctor”. Using this function, it is possible to confirm the locational relationship between structures where people cannot easily reach or that owned by different organizations and not on the same drawing. And also, it is possible to confirm the three-dimensional construction space for such as repair or reinforcement work, or a clearance between structure and the traffic space.



Figure 4. Clearance check on point cloud.

This system also has a function to create a contour map by giving a color to each point sequentially different depending on its position in the height direction. This function is applicable to measurement of inclination or rutting depth on the pavement surface (See Figure 5), and confirmation of the location of a ponding place or flooding range.

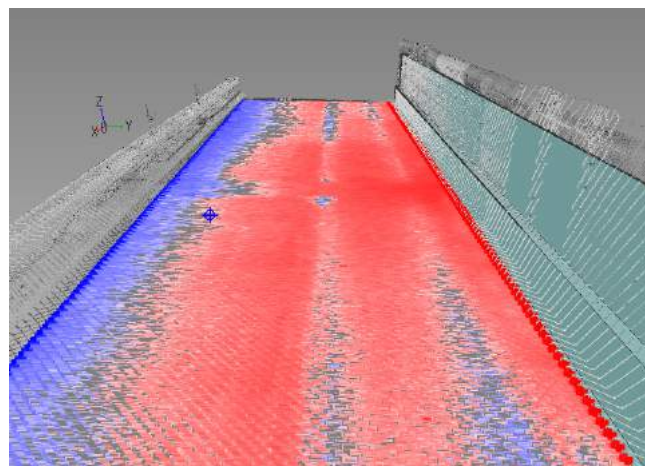


Figure 5. Contour of rutting on pavement.

3.3 Creation of Two-Dimensional CAD Data

By cutting the point cloud data in any cross-section, it is possible to create a CAD drawing in two-dimension. In conventional applications, a drawing is created by manually tracing the outline of the cross-section. On the other hand, the “InfraDoctor” can automatically generate outlines and make it much easier to complete a drawing. Since each point of the point cloud has coordinates, size and distance measurement can be outputted at any position and a general drawing of a structure with the dimensions, as seen in Figure 6, can be generated semi-automatically in a short time.

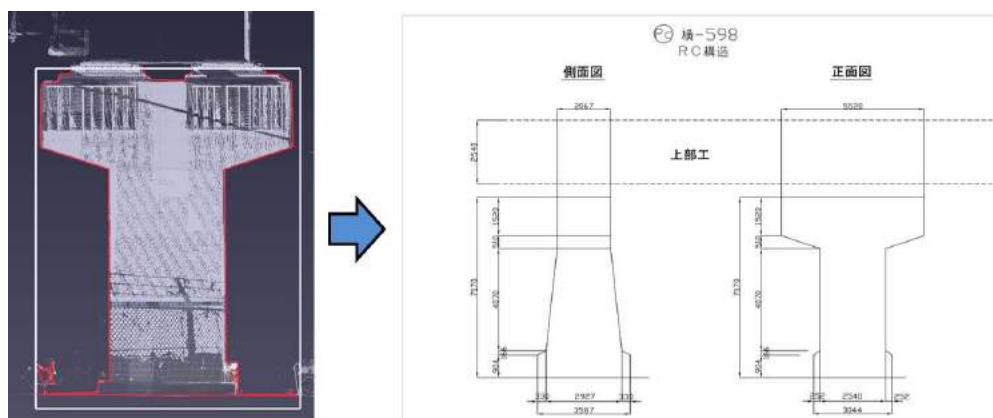


Figure 6. Two-dimensional CAD data creation (bridge pier).

There are about 700,000 bridges longer than 2 meters in Japan, but original drawings of approximately 300,000 old bridges among them have been lost, allegedly. This situation is surely hindering maintenance work for bridges. As for the appendages and the reinforcing members attached to a structure after completed, drawings are often not well conserved. Therefore, creating drawings of the current state of the structure using this computerized system will contribute greatly to the efficient maintenance work.

3.4 Creation of Three-Dimensional CAD Model

It is also possible to create three-dimensional CAD model, shown in Figure 7, in a short time by automatically generating the planar or curved surface of a structure from the point cloud data, using the “InfraDoctor”. Then, a three-dimensional FEA (Finite Element analysis) model can be created from the three-dimensional CAD data. This function is effective to analyze the strength of a structure and to carry out a design of repair or reinforcement, using the shape of structures in the current situation with the damage or deformation caused by natural disasters or other external loadings.

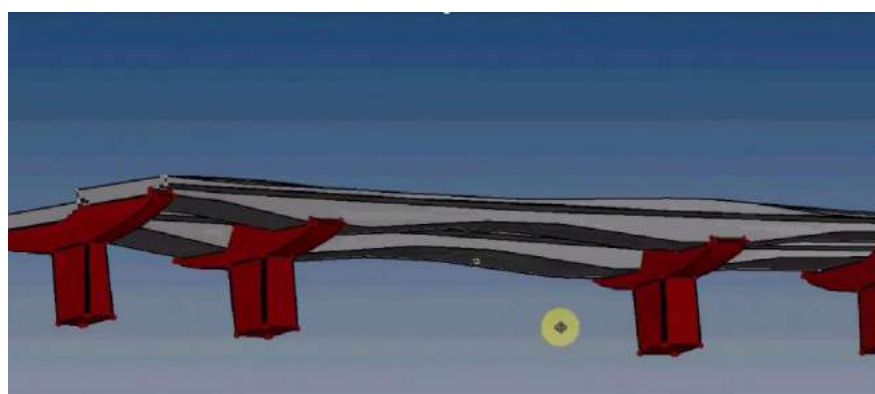


Figure 7. 3D CAD model created from point cloud data.

3.5 Three-Dimensional Simulation

The point cloud contains coordinates of any objects in the space measured by the MMS, such as structures without drawings or owned by different organizations, as explained in § 4(2). Various simulations can be carried out using the point cloud data with integrated CAD data of structural members or vehicles, and so on.

In an inspection simulation using CAD data of inspection vehicle on the point cloud (see Figure 8), for instance, it is possible to confirm the interference between the inspection vehicle and the obstacles, such as surrounding structures or appendages. By performing this simulation in advance, it is possible to greatly shorten the inspection time on site. In addition, a three dimensional simulation is also very effective for construction planning work such as loading large members into a limited space.



Figure 8. Inspection simulation.

Using this simulation function, you can see the circumstance on the road just as you are driving a car. This “driving simulation” is effective to confirm the safety of road alignments or traffic control.



Figure 9. Driving simulation.

4 FUTURE DEVELOPMENTS

Various data made or obtained at the time of construction, such as design statements, drawings, material data, methods or conditions of construction work, inspection results including construction error, repair records before open, and so on, will be managed on the “InfraDoctor” as the initial data of the structure. It is expected that more timely and accurate examination for repair or reinforcement of a structure can be executed by understanding the present status in a short time, using those initial data, the inspection records and the analysis-modelling function explained above, when any damage or deformation of the structure under use is found.

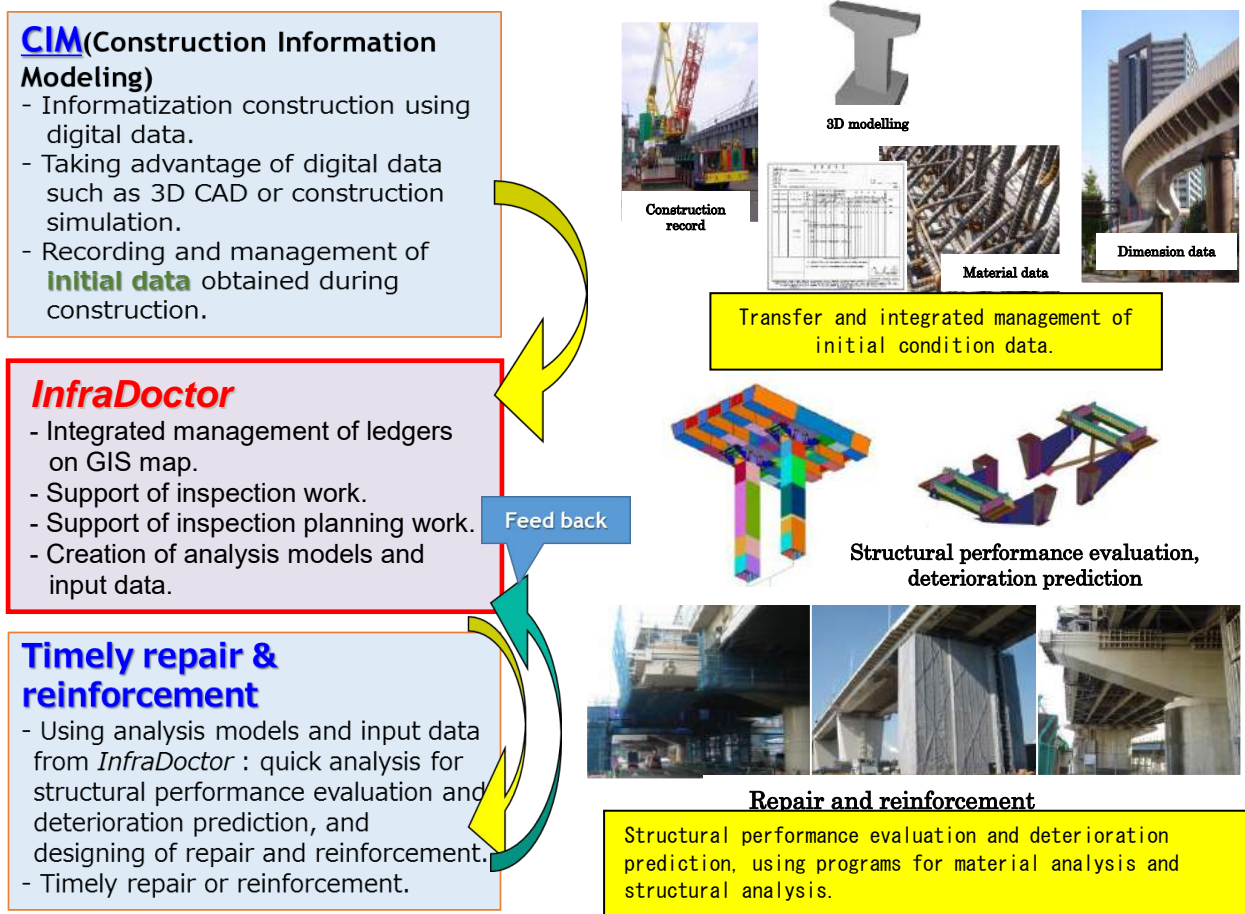


Figure 9. Concept of future development

Also, a study has been started to predict deterioration of structures or materials, by matching inspection data that is obtained using various new inspection technologies under development with analytical result introduced above, using an “AI” (Artificial Intelligence).

5 CONCLUSION

The “InfraDoctor” is a computer system to support operation and maintenance of structures in many ways, by managing various ledger data regarding structures on the GIS platform and utilizing all-around video and point cloud data. In the Metropolitan Expressway Group, this computer system is now under a trial operation in order to demonstrate that the application of this system can make the operation and maintenance work more efficient and accurate. The result of the trial will be reflected to improve the system further. It is expected that the system contributes to more efficient infrastructure management not only in Japan but also overseas including Asian countries, in the near future.

ROAD ACCIDENTS IN URBAN AND RURAL AREAS OF TANZANIA

AUTHOR: BRUNO KINYAGA

1. INTRODUCTION

Road traffic injuries claim more than 1.2 million lives each year and have a huge impact on health and development. They are the leading cause of death among young people aged between 15 and 29 years, and cost governments approximately 3% of GDP. Despite this massive – and largely preventable – human and economic toll, action to combat this global challenge has been insufficient. The third Global status report on road safety shows that low and middle-income countries are hardest hit, with double the fatality rates of high-income countries and 90% of global road traffic deaths. Vulnerable road users – pedestrians, cyclists and motorcyclists – make up half of these fatalities, by WHO Global Status on Road Safety, 2015 Report).

Motorcycles transport commonly known as (boda boda) in Tanzania has been growing up in the recently years in both urban and rural areas. This means of transport has been used by a large number of people in the country due to the facts that it is the fastest means of transport for someone to reach anywhere. Up to June 2014 the number of motorcycles that has been registered was about 1.2 Million and currently it's about 1.5 motorcycles been registered in the country.

2. COUNTRY PROFILE

Tanzania is an East African country about 5° south of the Equator with a population of about 45 million in 2012 (National Bureau of Statistics, March 2013). Tanzania is a relatively large country located in East Africa with a total area of 945,087 square kilometers. Tanzania has 30 administrative Regions as well as about 128 districts.

Figure1. Tanzania Map



3. STUDY AREA

The study was conducted in Dar es Salaam region (Tanzania), where most of people use this mode of transport. The population of Dar es Salaam is 4,364,541 according to the official 2012 census, increasing at 5.6 percent per annum from 2002 to 2012.

Study was conducted in three representative districts (Kinondoni, Ilala and Temeke) within Dar es Salaam region. The selected districts are well known in the use of motorcycles transport service. A large number of respondents involved in the study areas are those who engaged in providing the service (motorcyclists) and the users of the service (passengers).

Table1. Districts of Dar es Salaam Region and its Population

District	Population (2012)	Area km²
Ilala	1,220,611	210
Kinondoni	1,775,049	527
Temeke	1,368,881	656
Total	4,364,541	1,393

4. METHODOLOGY

The study used both qualitative and quantitative methods of research by conducting physical surveying on sample districts. A cross-section descriptive study was conducted from June to September 2014 whereby different people interviewed using structured questionnaire. The Data obtained was entered into a statistical package for social studies (SPSS) program (version 15) for clearing, coding and statistical analysis.

5. FORMULAE

The study covered 289 people obtained through sampling techniques calculated by using single population proportion formula,

Formula

$N = Z^2 p (1-p) / E^2$ whereby, N = is the minimum sample size required

Z= 1.96 at 95% confidence interval

E = is a margin of sampling error rate 5%

P = Proportion of motorcycle crash accidents victims (25%)

Substituting these values to the equation above;

$N = 1.96^2 * 0.25(0.75) / (0.05)^2$; N= **289**

6. FINDINGS

This study found that, most of the crashes occurred between motorcycles and motor vehicles were (70-80%) compared to other studies (50-55%). Crashes between motorcycle and motorcycle are of (10%) compared to other studies (7%), motorcycles and pedestrian (5%) compared to other studies (10%), lone motorcycle (5%) compared to other studies (11%), motorcycles and bicycles (5%) compared to other studies (3%).

Table2. Motorcycle accidents occurred in Tanzania from January 2015 to June 2015 compared with accidents occurred from January 2014 to June 2014.

Year	No. Accidents	No. Deaths	No. Injuries
2015	4079	1747	4826
2014	3170	1423	3622

Figure2. Contribution factors to motorcycle accidents.

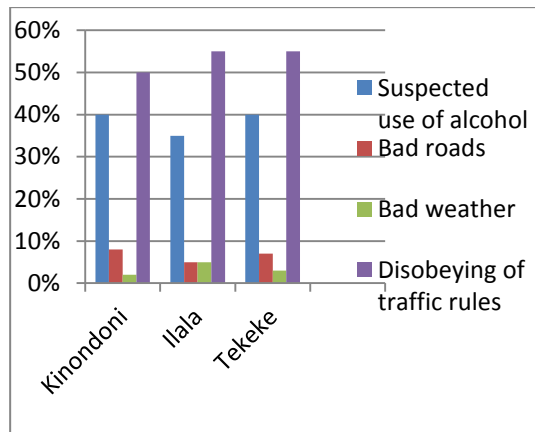


Figure4. Age group of motorcyclist's v/s the percentage of accidents associated.

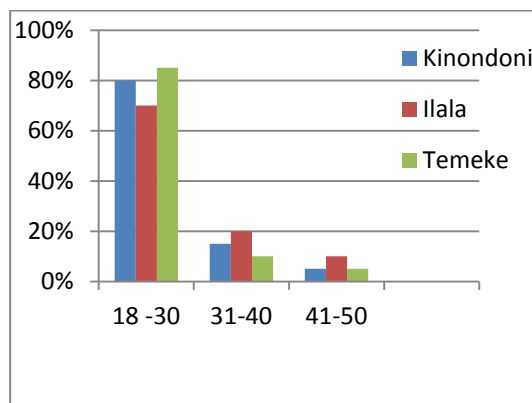


Figure5. Motorcyclist behavior of carrying more than one passenger without helmets



Table3. Motorcyclists with riding license and without license

	Kinondoni	Ilala	Temeke
With License	(%) 60	(%) 70	(%) 50
Without License	40	30	50

Figure6. Accident mechanism; Motorcycle - Motor vehicle Crash



7. ISSUES IMPACTING ROAD SAFETY IN MOST DEVELOPING COUNTRIES

- Lack of enforcement on road traffic laws and regulations.
- Speed management, which lies at the heart of an effective approach to reducing deaths and injuries, is notably poor in many countries.
- Lack of road safety education to road users
- Fake licenses “*A lot of fake licenses*”
- Corruption
- Lack of education for operators
- Roads are not perceived as dangerous
- Lack of accident registration(Data Collection and Management)
- Roads continue to be designed and built without sufficient attention to the needs of the most vulnerable road users.

8. CONCLUSION

Road traffic injuries and deaths place a heavy burden on national economies as well as on households. In low and middle income countries, they particularly affect the economically active age group, or those set to contribute to family, society and the workforce in general. Unfortunately road safety has not been taken as a serious issue. Probably these problems seem to be more urgent or important but if we analyze the cost related to the victims, or serious injured people that will not work anymore or may be depending on the government helps to live due to the consequences of the accidents, the road safety topic becomes more important than they originally thought.

9. RECOMMENDATIONS

- Strictly enforcement of road traffic laws that instruct the riders to attend pre-riding courses and to make assurance that they have been tested by traffic police before possessing a riding license.
- Conducting several check on alcohol use to motorcyclists during riding.
- Initiate road safety education to road users.

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- The Chartered Intitute of Logistics and Transport (CILT)- Tanzania
- transaid
- Surface and Marine Transport Regulatory Authority -Tanzania

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PAPER TITLE	High Speed Data Collection and Automatic Crack Mapping at the Network Level on the Bundaberg Region		
TRACK			
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KEYWORDS:

GIS, profile laser data, pavement distresses, automatic crack detection, integrated assets management

ABSTRACT:

Bundaberg Regional Council (BRC) and Pavement Management Services (PMS) have worked together in the collection of multiple surface pavement parameters and asset identification of the BRC sealed (over 1,900 km) and unsealed (over 1,100 km) networks.

This paper shows how the integrated data collection using multiple systems (laser profiling sensors, 3D pavement cameras, right of way cameras and a navigation system for georeferencing purposes and integrated asset management), can benefit road agencies for managing their road network.

In order to visualize the data, PMS proposed a custom tool that synchronises and shows all the collected information. The data can be visualised at different levels, from road level for the network decisions to distress level for maintenance tasks.

The information is shown using not only images, charts and tables, but also specific tools to show the information through a web map interface, with capabilities to zoom in at the crack (mostly automatically detected) level on the pavement images.

This paper shows how the high speed data collection system helped BRC staff for better managing their road network by assisting them in the decision making process. It also shows the advantages of visualizing the results in an integrated environment to better understand the relationship between different elements (and associated parameters) that coexist in a local road network.

High Speed Data Collection and Automatic Crack Mapping at the Network level on the Bundaberg Region

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1 INTRODUCTION

The challenge of managing our road pavement network has become an important and the most critical role for a road asset manager or an engineer in charge of monitoring pavement network behavior.

This paper showcases the advantages of the high speed road data collection systems with simultaneous collection of multiple datasets that provides a comprehensive set of results with homogeneous referencing system. This methodology will allow review of all the information in an integrated viewer interface, where the relationship between the different data (e.g. relationship between rutting and fatigue cracking) can be easily seen.

In addition, the integration of the automatic crack detection system has shown a way to improve the consistency and repeatability of the pavement distress rating at the network level.

2 PROJECT SUMMARY

Bundaberg Regional Council (BRC) and Pavement Management Services (PMS) have worked together in the collection of multiple surface pavement parameters and asset identification of the BRC road network. It will allow obtaining a general, but at the same time deeper knowledge of the BRC road network condition that is accessible to the BRC staff in a simple and centralised way.

2.1 THE REGION AND THE NETWORK

Bundaberg Regional Council manages the road network of Bundaberg Region located in the Wide Bay-Burnett area of Queensland, Australia, about 360 kilometers north of Brisbane. As a reference it covers a surface of 6,450 km² and has a population of around 100,000 people.



Figure 1. Location of Bundaberg region in Australia

The BRC road network measures around 3,000 km and can be split on two types of roads based on their surface: sealed and unsealed roads. The sealed network comprises of around 1,900 km divided in approximately 4,300 segments and the unsealed network is about 1,120 km (mainly in rural areas) divided in approximately 1,070 segments. The criteria used for segmentation is based on intersections with other roads/streets or significant changes in the road characteristics. The location of the sealed and unsealed networks can be seen in blue and orange respectively in Figure 2 (left side).

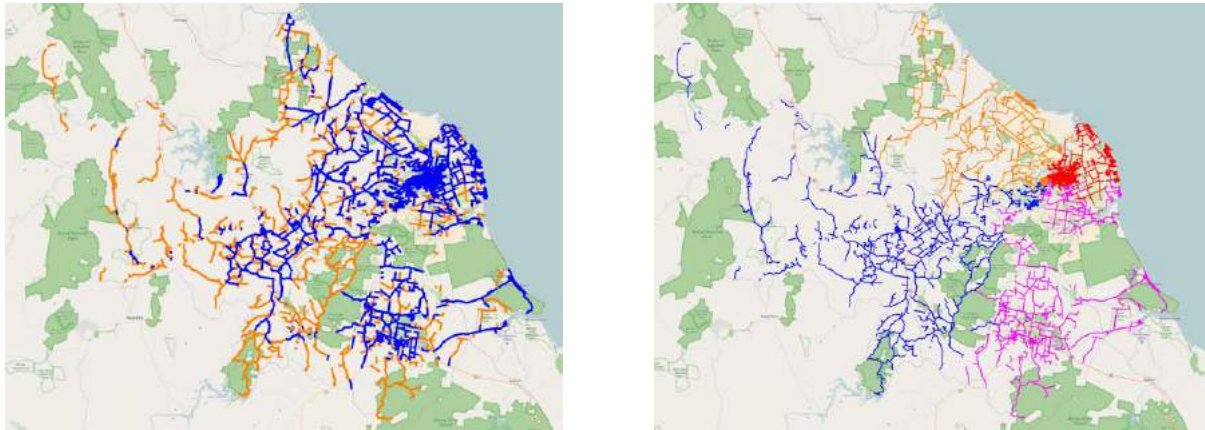


Figure 2. BRC Road Network. Sealed and Unsealed (left). Planning of Collection (right)

2.2 BRC PROJECT OBJECTIVES

The main objectives of this project were to provide an overall condition of the BRC networks (sealed and unsealed) in terms of the data summarised in Table 1 below. That information should be managed in an integrated manner through a viewer interface that simultaneously handles the different collected datasets.

Category	Sealed network	Unsealed network
Surface characteristics	Roughness & Rutting & Texture	N/A
Pavement distresses (visual condition data)	Cracking: (Long. and Trans. Fatigue and Block) Ravelling - Stripping Patches Potholes	Shape loss Cross section Corrugation Potholes Embedded stones
Inventory data	Surface, pavement and formation width Kerbs and channel Barriers Traffic signs Floodways and Bridges Gates and Grids	Gravel and formation width Floodways and Bridges Gates and Grids Traffic signs

Table 1. Summary of Parameters to be Collected

3 METHODOLOGY

3.1 PROJECT SETUP

Due to the different nature and parameters to evaluate in both networks, the data collection and processing have been split to be managed independently. PMS used an ARAN system and a 4WD (four wheel drive) fitted with five cameras for the data collection of the sealed and unsealed networks, respectively.

BRC has provided for both networks GIS data with information regarding the start and end location of each segment. It was an excellent tool for planning the collection and also for processing and accurately segmenting the resulting data according to the BRC existing segments. The sub division of the networks for collection is shown in Figure 2 (right side).

3.2 DATA COLLECTION EQUIPMENT

In order to perform the data collection along the **sealed network**, PMS ARAN LCMS System (Laser Crack Measurement System) has been used. This system allows collecting road surface characteristics, in term of visual defects (right of way video recording), laser data (roughness, rutting and texture), as well as automatic crack detection. This system incorporates some state of the art sensors needed for collection of the following datasets:

- High resolution (HD) right of way images both from a front and a rear camera with a frequency of one set of images every 5 meters of road (independently of the speed of collection).
- Longitudinal profile of both right and left wheel paths obtained by the combination of laser distance measurement sensors and accelerometers.
- A continuous 3D elevation model of the pavement surface 4 meters wide obtained by an LCMS system with a resolution of 1 pixel per 1 mm². This system was used for both the automatic crack detection and the rating of other pavement distresses.
- Accurate location (track) of all the collected datasets obtained by an inertial navigation system (INS) based on an Inertial Measurement Unit (IMU), a GPS unit and distance measurement instrument (DMI) that provides accurate results even when there is no good GPS coverage on some areas.

On the **unsealed network**, a lighter, rougher and simpler to operate piece of equipment developed by PMS was used. It is mounted on of a 4WD unit that allows collecting on the harsh environment of the BRC unsealed network. It provides with a set of distance referenced and georeferenced images every 2 meters of displacement along the road in order to be able to rate the pavement distresses.

This system was comprised of the following components:

- A multi camera (5 cameras) system that collects images of the different angles of the road (front and rear).
- An integrated high precision DMI (distance measurement instrument).
- A differential GPS unit.

The equipment units used for collection on both sealed and unsealed networks can be seen on Figure 3.



Figure 3. PMS Data Collection Equipment. Sealed Network (left). Unsealed Network (right)

3.3 THE DATA COLLECTED

The collected data for the sealed and unsealed networks can be divided in three main areas:

3.3.1 Surface Characteristics (laser data)

The surface characteristics (laser data) are based on the analysis of the longitudinal and transverse profile of the road. The parameters considered for this particular project included **roughness**, by means of the calculation of the International Roughness Index (IRI) transformed into the Australian roughness value called NAASRA Count; **rutting** by measuring the left and right wheel path rutting and **texture** by means of the evaluation of the MPD (mean profile depth) on both wheel paths.

These values provide overall information of the condition of the pavement surface and have the advantage of being obtained in a completely automated way, so the accuracy and repeatability of the results are assured in properly calibrated equipment.

Due to the nature of the unsealed road network no laser system was collected and therefore no roughness and rutting data was processed.

3.3.2 Pavement Distresses (Automated Crack Detection and Visual Surface Condition)

For the sealed network both cracks and other pavement distresses (ravelling, patching and potholes) were identified. These two types of defects were evaluated differently. For cracking, an automated crack detection system based on 3D images of the pavement was used. On the other hand, a visual inspection based on the right of way (ROW) images and pavement images was carried out to post rate pavement distresses.

Depending on the criteria of the organizations that manage a road network and also based on the most common defects that can be found in the road networks, different levels of severities and different classifications have to be considered when setting up a schema for rating the pavement distresses. In this particular project to define the different types of distresses along the **sealed network**, several references like the Condition Assessment & Assets performance guidelines from the IPWEA and the Distress Identification Manual from SHRP/FHWA were used. Based on these documents BRC and PMS defined a custom rating schema to evaluate the pavement condition. The summary of the ratings and types of distresses for the sealed network are shown in Figure 4 (left side).

On the **unsealed network** the rating of pavement defects was based on the Standard Visual Assessment Manual for Unsealed Roads by Jones, D & Paige G. The following defects have been considered relevant to the analysis of the BRC unsealed network: a) Cross Sectional Profile, b) Drainage, c) Potholes, d) Corrugation and e) Embedded stones. For all these distresses the extension and severity of each of the distresses was evaluated base on the ROW images collected. Figure 4 (right side) shows a summary of the different distresses and severities.

Defect ID	Defect / Distress Type	Low Severity	Hight Severity
1	Crocodile Cracking	% of pavement area with cracks < 3mm	% of pavement area with cracks ≥ 3mm
2	Block Cracking	% of pavement area with cracks < 3mm	% of pavement area with cracks ≥ 3mm
3a	Transverse Cracking	% of pavement area with cracks < 3mm	% of pavement area with cracks ≥ 3mm
3b	Longitudinal Cracking	% of pavement area with cracks < 3mm	% of pavement area with cracks ≥ 3mm
4	Pothole (Unrepaired)	% of pavement area with potholes	Not Measured
5	Patching	% of pavement area with patches	Not Measured
6	Stripping/Ravelling	% of pavement area with Stripping or Ravelling	Not Measured

	Degree 1 X 2 3 4 5	Degree 3 1 2 X 4 5	Degree 5 1 2 3 4 X
Cross Sectional Profile	Very good 	Flat 	Very uneven
Drainage	Well above ground 	Level with ground 	Water flows along road
Potholes	< 10mm deep 	20 – 50mm deep 	> 75mm deep
Corrugation	Not felt or heard in a vehicle 	Felt and heard, Speed reduction necessary 	Felt and heard, must drive very slow
Embedded stones	Not felt or heard in a vehicle 	Felt and heard, Speed reduction necessary 	Felt and heard, must drive very slow

Figure 4. Distresses Rating Schemas. Sealed Network (left). Unsealed Network (right)

3.3.3 Assets Inventory and Condition Assessment

In addition to the pavement indicators discussed above, the survey included georeferenced inventory of several road related assets (considering the condition of the assets for some of them). The assets that were to be identified are summarized on Table 1 above.

In order to conduct the inventory, a code of the elements based on the Manual of Uniform Traffic Control Devices, was setup, and the schema for defining the different condition levels for the rated elements was also prepared.

3.4 PROCESSING WORKFLOW

PMS developed a database to store and deliver the collected pavement condition data and asset inventory for the unsealed network. In addition, a PMS video player containing all the collected videos of the unsealed network has been included.

Regarding the sealed network the following methodology summarised the workflow that has been followed.

- Preparation Road list for collection
- Field data collection
- Importing of field data
- Segmentation of field data
- Data processing:
 - Laser data processing
 - Automatic crack detection
 - Other pavement distress rating
 - Asset inventory elements location and rating
- Compilation of results
- Preparation of deliverables
 - Integrated viewer
 - Tabular data
 - Georeferenced data

Figure 5 below shows one of the tools that have been used for QC the collected data, which is also used for processing and QC the laser data. Some of the main tasks of this workflow are briefly explained on the next sections.

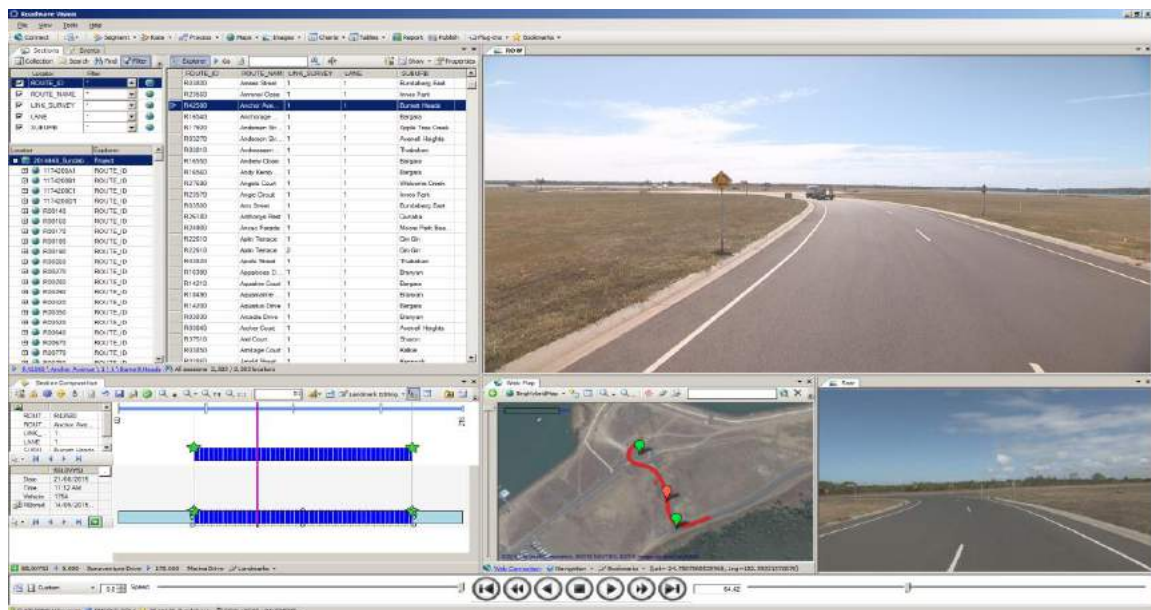


Figure 5. Processing Software (General View)

3.4.1 Segmentation

One of the most important steps in order to process the information and deliver it with the proper referencing (geographical location and also linear reference [chainage]) is the segmentation of the collected information. PMS and BRC have used an automated segmentation process that assigns the collected data to the planned collection segment, rubber-banding (stretching or shrinking) the results to the planned location and chainage references.

Figure 6 shows a sample where the planned collection length is compared to the real collected length (showing an accuracy of over 99.4% in this particular case). The use of these tools assures the proper location of the collected data and also allows verifying the quality of the data providing the ability to update any inconsistency that maybe found between the planning and reality.

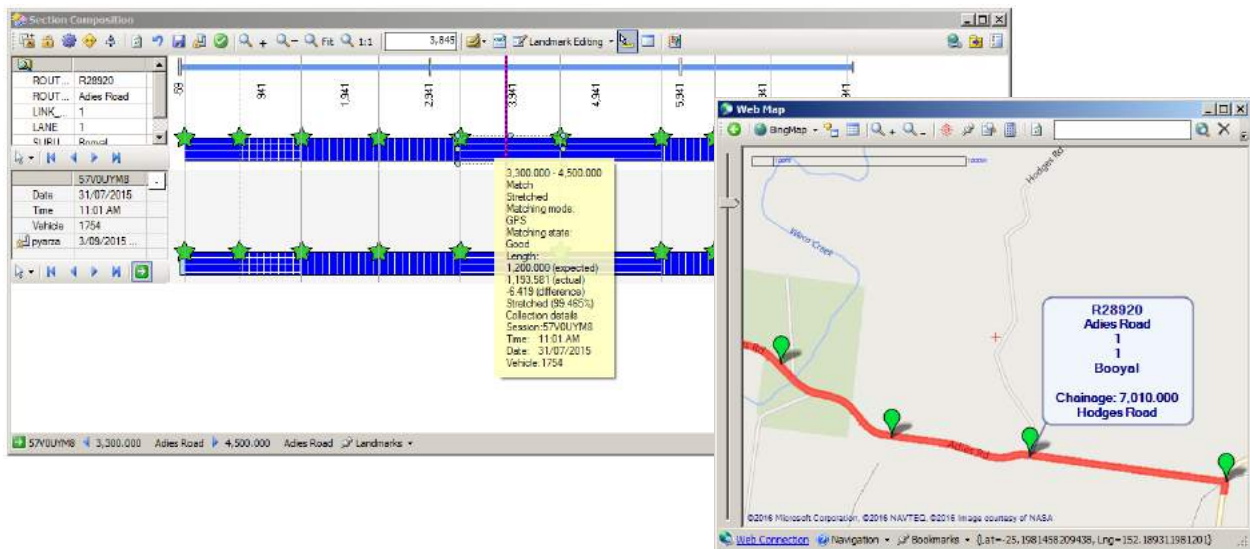


Figure 6. Segmentation. Auto Matching Results

3.4.2 Automatic Crack Detection

In order to fulfill the requirements of this project PMS has used a system that automatically detects and classifies the different crack types that can be found in a road network.

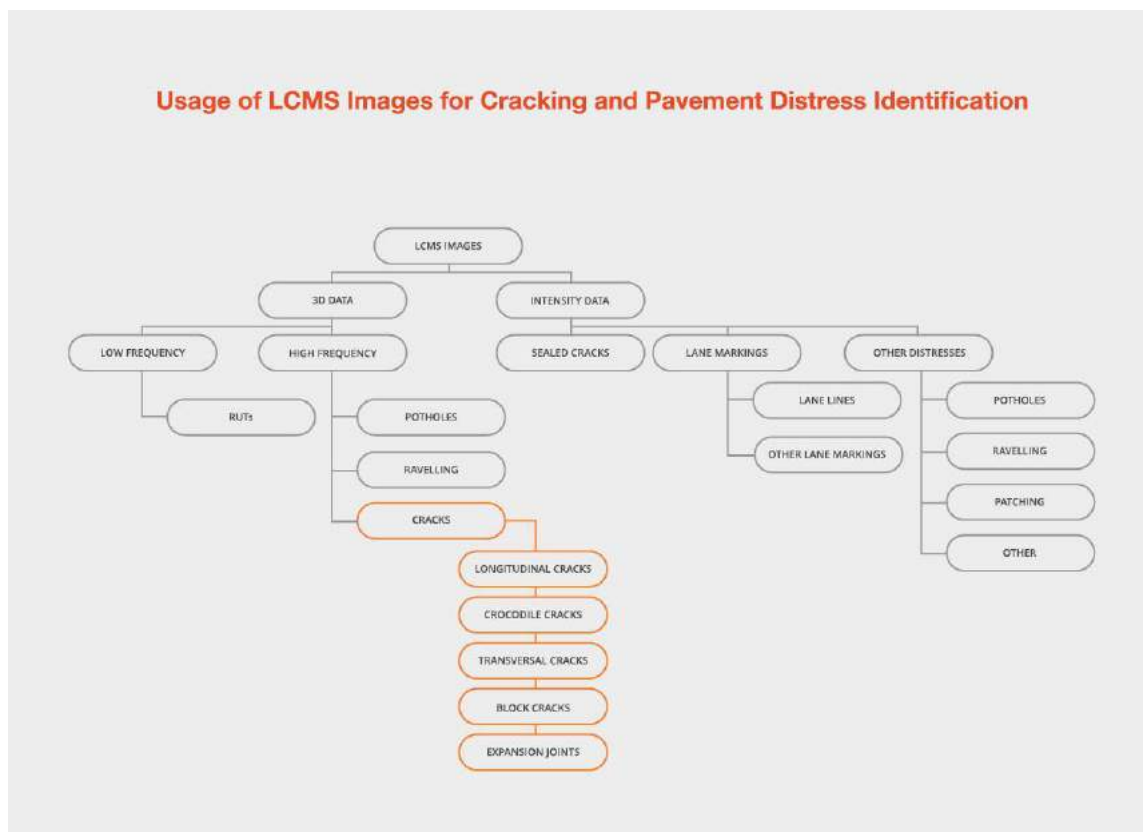


Figure 7. LCMS Images. Uses for the Different Types of Images

Figure 7 shows all the different types of distresses that can be identified on the LCMS pavement images, based on both the 3D range images and the intensity images obtained by the system.

Based on the capabilities of the LCMS system and the distress classification that has been defined to this project, the automatic processing of the pavement images has been completed.

Among the automatic tasks that are done by the crack detection system, the main ones are the following:

- Detecting the cracks from the set of 3D images collected by the LCMS equipment
- Classifying/grouping the cracks regarding their type (fatigue, block, longitudinal and transverse) and their severity (width for linear cracks and density for fatigue and block cracks)
- Aggregating the results in order to provide indicators that deliver an overview of the network condition at different levels of detail (from the crack details to the network level average values)

Figure 8 (top left) shows some samples of the automatic ratings obtained after the automatic crack detection process is completed.

Depending on the requirements of different projects, other configurations can be setup, enabling classification of the cracks according to their location along the transverse profile (outer wheel path, between wheel paths and inner wheel path) or regarding what metrics (length, surface, % of area affected) to be considered when quantifying the results.

3.4.3 Other Distresses Ratings

Other distresses like patching, ravelling or potholes are also manually recorded and rated using both the right of way (front and rear images) and the high resolution pavement images. Some samples of the obtained results can be seen on Figure 8 (bottom left).

3.4.4 QC and QA Processes of the Pavement Distress Ratings

Further to the automatic image processing algorithms used for detection and classification of the cracks and the distresses, a manual QA/QC procedure took place in order to assure the quality of the results.

Figure 8 (right) shows the general workflow followed to ensure the quality of the results obtained both at the collection level and also at the automatic distress detection level.

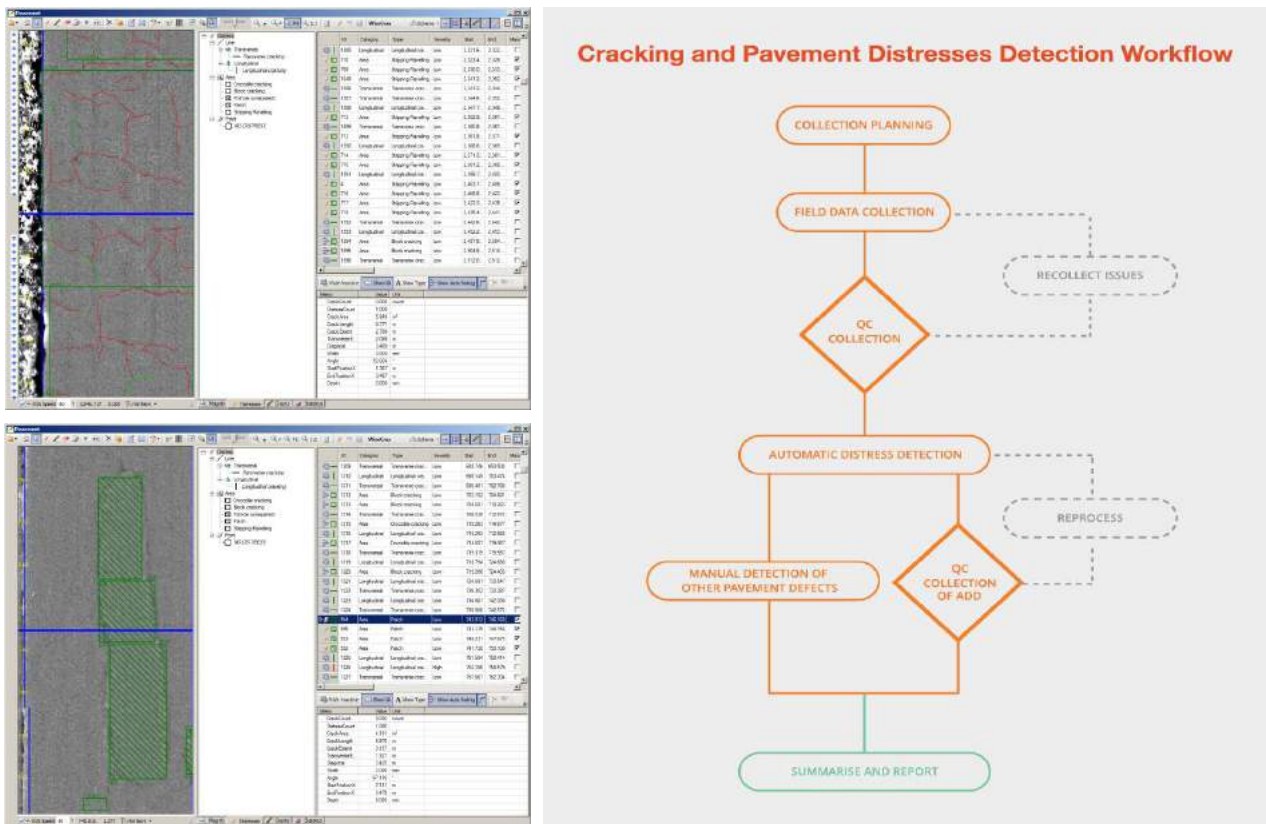


Figure 8. Distress Rating. Automated Crack Detection (top). Rating of Patches (bottom). QC Workflow (right)

4 RESULTS

4.1 INTEGRATED MULTI-DATA VIEWER

In order to manage all the information shown in the previous sections, an application for visualising all the road related information has been developed. This viewer application is a web based viewing platform enabling the efficient viewing of video and pavement condition data. It allows multiple concurrent users from different locations to access to all road related information stored in a central repository just by using a web browser (Figures 9 & 10).

The main functionalities of this tool are:

- **Video player button bar** that allows movement sequentially along the collected roads
- **Image views** (right of way and pavement images) with zoom functionality and the ability to overlay detected distresses and cracks on top of pavement images (with identification of cracks attributes)
- Configurable **charts and tables** synchronized to the remaining views with the summary of the collected data (Roughness, Rutting, Pavement Distresses, etc.) at different aggregation levels.
- Synchronized **interactive map** view showing the position of the current location and the current track collected by the different pieces of equipment
- A **dashboard** for overall network comparison of the different parameters obtained for the entire road network.
- A set of **measurement tools** that allow to interactively measure and record areas and lengths of the different elements that can be identified on the right of way images (lane width, barrier height, patch areas, etc.)
- Front, Rear and Pavement images (with overlaid crack maps). Roughness chart and table. Map view.

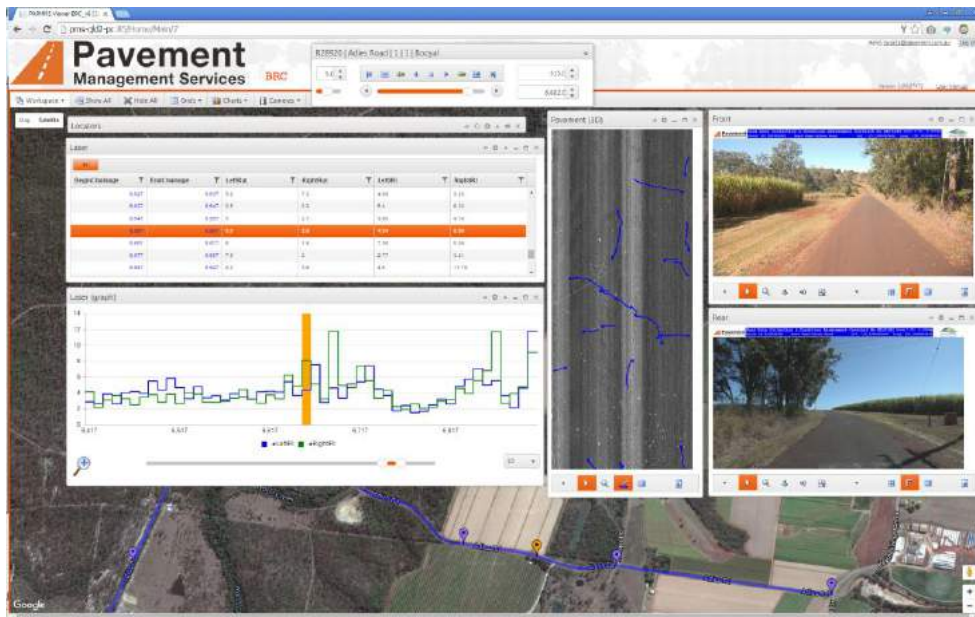


Figure 9. Viewer Application

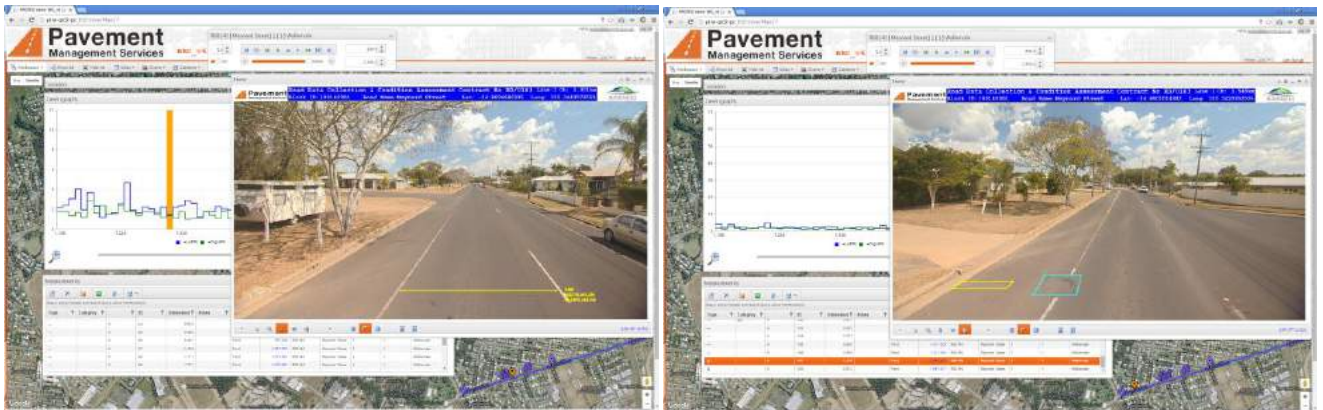


Figure 10. Viewer Application. Width measurement (left). Surface measurement (right)

4.2 RESULTS SUMMARY

In summary, the following data and information has been collected, processed and delivered to BRC for better managing and understanding their sealed and unsealed road networks.

Sealed network:

- Roughness, Rutting, Texture and Pavement Distresses data has been compiled over 1,820 km of sealed roads.
- Over 30,000 asset elements have been identified and geolocated.
- Over 16,000 width measurements have been made on the sealed network.

Unsealed network:

- Pavement distresses have been rated for over 1,150 km of unsealed roads.
- Over 2,700 assets (inventory) elements have been identified and geolocated.
- Over 3,500 width measurements have been made on the unsealed network.

Besides the results being shown in the integrated viewer described above, some of the obtained datasets (mainly the assets/inventory data) has been provided as georeferenced information for using on the BRC assets management tools.

Also some other information that is also shown in the viewer such as surface characteristics and pavement distresses have also been provided in georeferenced tabular format (at 10 meter interval and segment level) for using on the BRC pavements maintenance plans.

5 CONCLUSIONS

The main purpose of this paper was to present the capabilities of the high speed pavement data collection to assist BRC staff to better manage their road network and the decision making process.

It has also been shown how the use of automatic crack detection systems can improve the accuracy and repeatability of the pavement distress ratings that are performed on a road network.

Finally, the paper showed the advantages of using specialized, but at the same time simple to use, viewing software that shows the results of a data collection on a road network, by showing the information in an integrated environment that allows for a better understand the interrelations between different evaluated parameters that coexist at any spot on a local road network.

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PAPER TITLE	The Impacts of Climate Change on Transportation Safety and Their Consequences		
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KEYWORDS:

Road safety, climate change, transportation, pavement materials

ABSTRACT:

The previous researches have shown the contributions of transportation design and construction to climate change. Therefore, it is almost unquestionable that the climate is affected by such activities, and further studies are required in order to observe the consequences of climate change on transportation safety and passenger health. For instance; sudden rain falls, quick icing which are common incidents due to the changes in climate of a particular region may cause crashes, injuries, and increase of mortality more than the planning stage predictions of pavement design. Another example would be unexpected wind flow direction shifts which may risk landing/take off of a passenger aircraft. In overall, this paper interprets the impacts of climate change on transportation safety and hence its economic assessments. The evaluations and discussions of this work are highlighted with the use of case studies and various observation techniques.

The Impacts of Climate Change on Transportation Safety and Their Consequences

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1 INTRODUCTION

It was both at national and international level agreed that some degree of climate change is now unavoidable regardless any future reductions in emissions. This statement was also stated by the Intergovernmental Panel on Climate Change (IPCC) (IPCC 2007). It is clear that the increase of detrimental impacts of global warming and hence all other related climate change phenomena are all related to anthropogenic activities. The usage of fossil fuels causes the release of greenhouse gases. Transportation network is one of the biggest sector which causes the highest greenhouse gas (GHG) emissions. It is apparent that even a slight increase in GHG emissions cause an increase in global warming. An increase of 1.3 °F (0.74 °C) was observed in global average surface temperature since 1850. The researchers have always been focusing on the detrimental effects of materials used or the GHG emissions caused by the Transportation sector. However, a little attention was paid on the fact that global warming and hence climate change also influence the quality, durability and safety of the road designs and hence the materials used. Therefore, a significant decrease in the road safety is expected due to the detrimental impacts of the climate change on the road pavements.

The awareness of the negative effects of climate change on road safety has been increasing for the last few years. Many researchers have been working actively on these effects, their causes and hence consequences (Qiao 2015; Amponsah-Tawiah & Mensah 2016; Taylor & Philp 2015; Rattanachot et al. 2015). It was found by the researchers that the unexpected changes in temperature mostly due to climate change have reduced the long-time performance of flexible pavements. The increase in temperature or seasonal hot/cold extreme temperatures are found to be the main reasons of aggravated flexible pavement mostly at regions where extreme hot/cold climatic conditions are observed (Qiao et al. 2013; Tighe et al. 2008).

This research study is an on-going extensive work which focuses on the road safety which is mainly caused by the changes in climate in Turkey. As it can be seen in Figure 1, major defects such as changes in asphalt flux and major cracks throughout the highways are the main cases that were observed all over the country, mainly caused by unexpected climatic conditions due to changes in climate.



Figure 1: Various pictures taken in Antalya highway and Çanakkale-İzmir interstate highway (En Son Haber, 2015)

The materials and methods used in pavement design and their negative impacts in climate change are discussed as already done by many other researchers in the literature. Not last but not least, the impacts of climate change on road safety and hence its long-term performance will also be evaluated within this work. At first, a specific region of Turkey namely Antalya, which has been experiencing unexpected severe climatic conditions during the last few years, will be investigated and analyzed using a few techniques as discussed within this paper. The cost of the influences of climate change on this region in discussion will also be evaluated and an estimated cost analysis will be undertaken. Recommendations in order to avoid traffic crashes due to the changes in climate and decrease the number of deaths and/or injuries will be given to the national authorities as part of this study.

2 THE IMPACT OF CLIMATE CHANGE ON ROAD SAFETY

Even though over speeding is accepted to be the major cause of road traffic accidents worldwide, it is worthwhile to discuss the detrimental impacts of climate change on road safety. It is clear that climate change influences the quality of road design and hence reduces the safety of roads.

While evaluating the influences of climate change on road safety, transportation safety factors must be taken into account. They are the factors that the roadway designers consider in order to minimize the number of crashes at a particular part of a road network (Reynolds et al. 2009; Oh & Mun 2012; Rzeznikiewicz et al. 2012). For example, vehicle speed limits, distance and straightness of network paths are important considerations (Demir 2013). Transportation safety factors are not only based on the design but also the weather conditions (indirectly the climate change) affects the safety factors at a road section. For example, after or during a precipitation, wet pavement increases the number of crashes at a particular segment of a road (Kou 2005; Caliendo et al. 2007; Pardillo Mayora & Jurado Pina 2009) and the number of fatal crashes increases dramatically if the rate of precipitation increases (Eisenberg 2004; Wang et al. 2011). There are studies for mechanical improvements to reduce the stoppage time in such wet conditions (Mazze et al. 1999); however the increase of the crashes due to a precipitation at a particular highway section or junction is a common result unfortunately. Wet pavements near the road intersections may cause stop violations (i.e. failure to stop a vehicle) (Retting et al. 2003). Stop violations at unsignalized intersections cause a great amount of rear-end crashes. Stop violations ending up with rear-end crashes are also contributed by pavements containing snow and ice accumulation near the intersection (Retting et al. 2003). Moreover, poor drainage systems on a highway segment may lead to crashes related to wet pavement conditions (Wu et al. 2015). Optimizing drainage locations may reduce the number of crashes due to wet pavements. In addition to rear-end collisions at signalized intersections, the other types of collision in traffic may occur due to the sudden change of weather. For example, poor visibility of traffic signals because of sudden and heavy precipitation, and following the leading vehicles on a roadway in such poor visibility conditions may cause crashes too (Broughton et al. 2007; Mitra & Utsav 2011). Especially fog and sudden heavy rain cause the reduction of visibility on a roadway. Also if the sunshine arises suddenly after a rainfall, wet pavement and water accumulations may reflect the sun light like a mirror. This kind of unexpected reflections, especially during sun rise and sunset times, may reduce the visibility and risk the safety of driving (Sun 2010; Churchill et al. 2012).

In addition to the unexpected weather condition changes, according to the researches undertaken pavement cracking and pavement roughness was observed due to the climate change including longitudinal and fatigue cracking (Kim et al. 2005). This additional impact on the performance of pavements brings a need of an additional maintenance for road safety. Changes in solar radiation, air temperature, and dew-point temperature play a significant role for the near-surface temperatures of road pavements (Gui et al. 2007). They not only change the stability of the road materials, but also affect the safety for road users. Paving materials mix design has to be considered while the geometric design of a road pavement is planned (Tighe et al. 2000; Meagher et al. 2012). Thus pavement safety measurements, criteria, and maintenance program and methods could be developed against the climate change of a region. All these will end up an increase in budget to maintain the level of performance at a constant level to provide full safety for the users.

Several studies at national level were undertaken in different countries to measure the cost of the effects of climate change on road infrastructure. For instance, according to Schweikert et al. (2015) research study, the national-level climate cost impact in South Africa was estimated to be in between US\$116.8 million and US\$228.7 million annually by year 2050 if no action is taken (Schweikert et al. 2015).

To mitigate the number of crashes is really vital in order to reduce the costs. The crashes caused by climate change may give rise to loss of many lives and injuries which have monetary costs too. According to the National Safety Council (NSC) in the United States for year 2011 data, the average comprehensive costs

by injury severity are very high (NSC 2013). For example, the cost of a death is \$4,459,000 while an incapacitating injury cost is \$225,100. Further, possible injury cost is \$27,200 and no injury cost is \$2,400 as shown in Table 1.

Table 1. Average comprehensive costs by injury severity

<i>AVERAGE COMPREHENSIVE COSTS BY INJURY SEVERITY</i>	
Injury Severity	Cost (\$)
Death	4,459,000
Incapacitating Injury	225,100
Possible Injury	27,200
No injury	2,400

Compiled from NSC (2013)

Such high amount of values would be considered with an application of Cost Benefit Analyses when deciding on installation of transportation safety measure alternatives related to climate change. Counter-measures, evaluation of safety programs, and allocating resources would be considered with the reality of climate change. Also construction costs, additional maintenance costs, enforcement and emergency management costs would be reevaluated bearing climate change in mind.

On a global scale, shifts in tourism and agricultural productions are also linked to the increase in temperature due to climate change. These unexpected changes in tourism sector and agricultural productions end up some shifts in passenger and freight transport (Koetse & Rietveld 2009). On top of that a focus on road and rail transport infrastructures is needed to see the changes in quality and long-life durability of the transportation systems due to the climate changes and vice versa.

Based on the above mentioned discussions it is clear to understand that the impact of climate change on transportation systems and vice versa requires some cost-benefit analyses to see which method or technique would be appropriate to apply to achieve a less costly, environmental friendly and less impacted systems from climate change.

Therefore, one of the main aim of this research study is to undertake studies at national level to measure the cost of the effects of climate change on road safety and being able to evaluate and compare the national cases with other nations worldwide.

3 FINDINGS AND DISCUSSIONS

3.1 THE USE OF PROPER MATERIALS IN ROAD DESIGN

The main materials used in road pavement design may be listed as asphalt, cement and aggregates. Aggregates are mainly used in road buildings. However, even the extraction of aggregates has a direct contribution into the climate change. Moreover, heavy lorry traffic on unsuitable roads will cause permanent and long-term pavement cracks. The use of finite natural resources in order to produce aggregates also will have direct impact on sustainability. Therefore, the right choice of materials to be used in road designs is required in order to reduce their negative impacts on climate change and vice versa. Luckily, there are a few alternatives available in order to replace virgin aggregates. The impacts of unused materials on climate change may be reduced by replacing them with other alternative materials e.g. recycled aggregates. The impact of road design on climate change and vice versa is linked to the materials used and the way of production. The amount of materials required for road design is extremely high since the paved surfaces can comprise a really high rate of land cover all over the world. For instance, in urban regions of the United States, land cover by paved surfaces may comprise up to 45%. These roads are mostly designed with energy intensive products made up from either Portland cement or petroleum-based asphalt (bitumen). The long-term durability of materials used in the pavement design is quite important in order to preserve the road safety and also to reduce the cost. Therefore, a right type of material should be chosen by considering all factors including the unexpected climatic conditions due to global warming and hence changes in climate.

3.2 EVALUATION OF TECHNIQUES AND STANDARDS

To help avoid or mitigate collisions due to slippery surface and poor visibility, there are suggestions to apply or traffic instruments to install for precaution. In order to decrease the wet pavement area which creates slippery surface overlaying pavement to change friction course, chip seal or slurry seal approaches, grooving pavement surface, providing adequate drainage, drainage locations optimization, setting appropriate speed limit, using slippery when wet sign might be applied (Sun 2010; Chen et al. 2014; Wu et al. 2015). In order to decrease the number of crashes due to poor visibility affected by fog, heavy rain, glare from sun, etc. the following precautions might be applied (Sun 2010). For example, installing and improving warning signs, installing thicker signal lenses allowing more visibility from further distances, installing signal visors, relocating and adding signals, removing sight obstructions, setting appropriate speed limits might be the solutions.

One of the most common applications in order to mitigate the number of crashes on roadways is using Traffic Control Devices (TCDs) (Stamatiadis et al. 1991; Zhang et al. 2014). For example, signs, road markings on pavement and delineators, signals, and temporary traffic controls regulate traffic give warnings and provide guidance to drivers. Standards are needed to set a single form for all drivers, pedestrians, cyclists, and so on. Standards in the United States are publicized in a manual titled Manual on Uniform Traffic Control Devices (MUTCD). The minimum standards of TCD's are set in MUTCD to provide guidance and ensure the uniformity of TCD's (MUTCD 2016). In Turkey, MUTCD standards are used as base to set up a manual for the traffic regulations in the country. The Department of Traffic Safety controlled by the General Directorate for Highways is responsible for this purpose and the most of the MUTCD regulations are used as basement to arrange TCD's for the Turkish highways, arterials, and streets. Since climate change affects the weather also in Turkey, adaptation to this change in terms of transportation safety is also important in this country. The manual might be reviewed and examined according to the new climate conditions. For example, there are parts in the manual about the change in driver behavior to adjust the speed while driving a private vehicle. The article mentions that the traveling speed on the roadway is mostly up to the drivers if congestion effects are neglected (General Directorate for Highways 2014). Drivers prefer their travel speed based on the condition that they feel safe. There are specific conditions affecting on adjusting traveling speed by drivers. These are, for instance, weather conditions, wetness/moisture conditions on pavement amount conditions related to snow, ice, mud, sand on pavement, of the light at the environment, time of the day. As can be seen from the manual publicized by the Department of Traffic Safety, such conditions are extremely related to the weather and climate change.

6 CONCLUSIONS AND FUTURE WORK

As it was already mentioned within the main body of this paper, this is an on-going research project and only a very few observations and investigations were mentioned in this paper. Antalya has been chosen as a pilot study area in Turkey to get some idea about the detrimental impacts of climate change on road safety. When all the required data is completed, it will allow the researchers to have some valid data about the general condition of the road network in Turkey mainly in Antalya. This will allow us to make recommendations on what should be done in transportation sector to reduce the negative impacts of climate change on road safety.

The General Directorate for Highways in Turkey and its subdivision the Department of Traffic Safety would take the changes in climate into account to review the regulations nationwide. As mentioned earlier, the Department of Traffic Safety is responsible for following up the recent upgrades in technology and developments in science to improve the safety in transportation. After confirmations of new applications related to climate change (e.g. improving the visibility of pavement markings, and the quality of pavement design (including the right choice of materials to be used), signalization materials and so on), traffic safety regulations might be changed in the manuals for transportation safety. Finally, the Department of Traffic Safety would implement the new techniques in order to mitigate the number of collisions in traffic.

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PAPER TITLE	Preliminary report for IRI changes after KUMAMOTO earthquake Japan, by using Smartphone roughness measurement		
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KEYWORDS:

IRI, response type measurement, pavement maintenance management, smartphone, accelerometer

ABSTRACT:

The Kumamoto earthquake was occurred in Japan on April 14th 2016. After this earthquake, road damage investigation was done that is roughness condition IRI measurement over 3,000km roads. This preliminary report is written for this investigation results.

As for this IRI measurement, response type measurement was done by using a smartphone built-in an accelerometer / GPS and a commercially available car, instead of the inertial profiler. This method is simple, easy and low-cost. So it can immediately measure IRI of whole disaster areas under the damaged rougher road. Measurement results are drawn on online maps. It includes roughness current status and comparison with past status before an earthquake.

Preliminary report for IRI changes after KUMAMOTO earthquake Japan, by using Smartphone roughness measurement

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1 INTRODUCTION

In Japan, Kumamoto earthquake was occurred on April 14 and 16, 2016, at the west end side of Japan. It was the quite large twin earthquake. In Figure shows the location of Kumamoto prefecture. And Figure 2 shows its seismic intensity map. The first shock occurred at 21:26, April 14 and it recorded moment magnitude Mw6.2. The second main shock was occurred at 01:25, April 16, whose magnitude was Mw7.0 bigger than first one. Of course, there were several aftershocks followed the initial quake. In a series of seismic activities, the first shock is usually the biggest one, but at this earthquake, the second main shock was the biggest, which was quite unusual. In the first main shock, many houses, buildings, bridges and roads were damaged. As you know there are many earthquakes in Japan, for example Kobe Earthquake 1995, Chuetu Earthquake 2004, and the Grate East Japan earthquake 2011. So in Japan, buildings etc. have seismic performance. For this reason, at the first main shock in Kumamoto, many of them have damages to some extent but it was not serious for residents' safe. At that time, I thought it was a large earthquake, but it could be under control. However, after the second shock, many structures had already lost enough seismic performance. Many structures were completely destroyed. As the result, 49 peoples died in Kumamoto earthquake. A at the Grate East Japan earthquake 2011, more than 20,000 people died because of the tsunami attack at some pacific coastal areas, not because the quake. Its wave was higher than 35m. At Kumamoto earthquake, more peoples died by quake damages. That was a large quake. Over 137 thousand houses were destroyed by the quake. Of cause it was affected not only to houses, but also to roads and pavements.

Previously, as for pavement inspection, only the human visual inspection used to be applied. In this case, the result is not quantitative, which is difficult to compare with other inspections. Also the use of an inertial profiler is difficult, because of its high costs and the possibility of the breakdown caused by bad road conditions after an earthquake. On the other hand, the response type roughness measurement by using smartphone is one of the solutions to these problems. It can measure quantitative data by using a car that is tougher than an inertial profiler. After two weeks from the main shock of Kumamoto prefecture roughness measurement was done the whole prefecture. Driving distance was up to 3,100km.

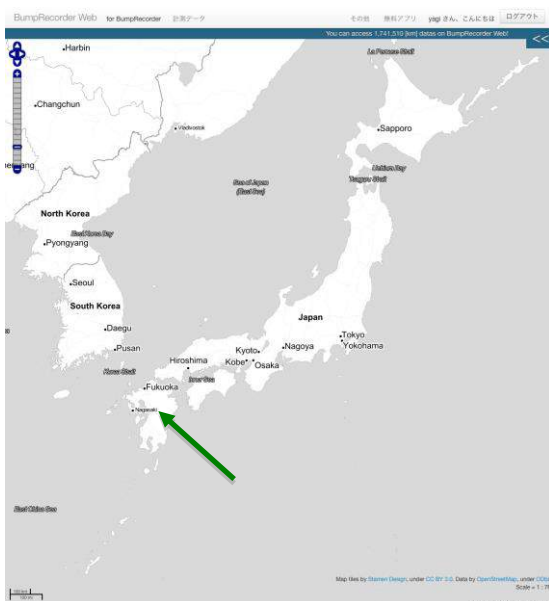


Figure 1. Location of Kumamoto prefecture

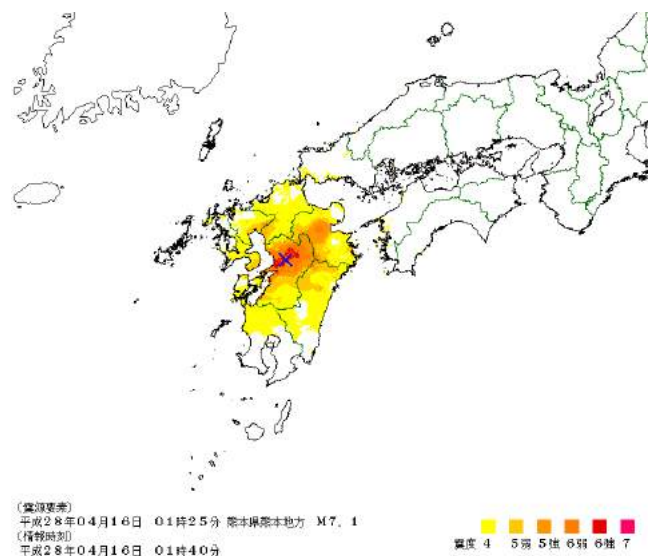


Figure 2. Seismic intensity map

2 SMARTPHONE TYPE ROUGHNESS MEASUREMENT

A smartphone application named BumpRecorder was used for IRI roughness measurement of the road damage inspection from Kumamoto earthquake. The measurement principle was reported at 1st IRF Asia Regional Congress. An essence of this method is as follows.

Smartphone is on the vehicle dashboard. It is located over the vehicle suspension. So a recording acceleration is easily influenced by vehicle model and driving speed. As the result, measurement results are not stable. To increase measurement stability, vibration frequency analysis is applied. Figure 3 is drawing calculation steps.

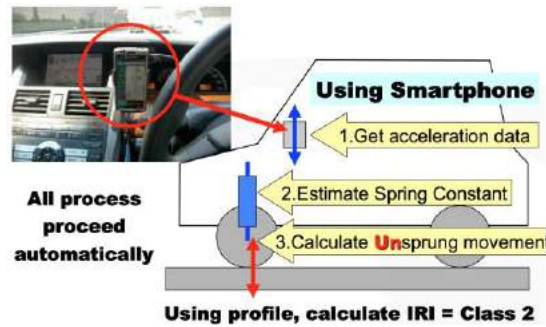


Figure 3. Roughness measurement method

At first, using FFT, vehicle suspensions resonant frequency and damping ratio are estimated. Even if recorded acceleration is including several resonant frequencies, suspensions movement is a main vibration, so it's resonant frequency can be found around 1.5[Hz] easily.

Next, using this spring parameters, the equation of motion of one mass spring model is calculated. This model appears in Figure 4. An equation (1) is an equation of motion for this model. In this equation, Lz is a sprung vertical movement, " u " is an unsprung vertical movement, " ω " is an angular frequency, " h " is a damping ratio. " ω " is defined by equation (2). In this equation, " f " is a resonant frequency. Here, " h " and " f " are used for previous FFT result.

And finally, an unsprung movement " u " is assumed that it is equal to road longitudinal profile, and it is calculating the Quarter Car simulation, then IRI will be calculated.

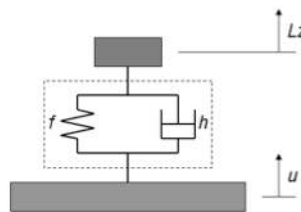


Figure 4. One mass spring model

$$\ddot{Lz} + 2h\omega(\dot{Lz} - \dot{u}) + \omega^2(Lz - u) = 0 \quad (1)$$

$$\omega = 2\pi f \quad (2)$$

Previously, the response type roughness measurement uses correlation equation between IRI and sprung acceleration. That is IRI class 3. And correlation equation is calculated by a test drive result of each vehicle model before measurement drive. On the other hand, BumpRecorder calculates longitudinal profile first, and then QC simulation is applied. That is IRI class 2. And it is not required a previous test drive. That is because calculation parameters of spring constant and damping ratio are estimated form acceleration data of measurement drive.

3 SQUARE MESH SECTION FOR IRI SECTION

To calculate IRI, calculating the section is important. Usually, this section is defined by a road location markers e.g. kilo-post data. But it is not convenience. Because kilo-post data is prepared for a main road, but not for many roads. When not using kilo-post data, the measurement GPS data is separated in a certain distance for example each 100m, which is easy in calculation. But unfortunately, GPS data has an error, when it wants to compare current IRI with previous IRI, it is expected that each IRI section is not same, and a comparison check is difficult.

To improve the situation, "Square Mesh Section" is proposed. It can pick up the same section for same location, anytime, anywhere, only by using GPS data. Square shape mesh is defined by the longitude and the latitude on the earth. When it is driving to cross over one mesh, a section between an entry point and an exit point is used in IRI calculation section. It is drawing in Figure 5.

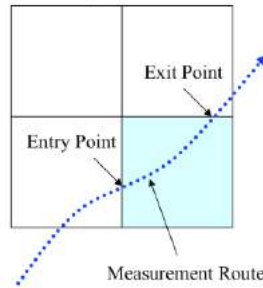


Figure 5. IRI calculation section

In detail, East West length of Square Mesh is defined by $1/8192=2^{-13}$ length for each 1 degree of the longitude. It is about 10m around Japan located around latitude 35 degrees. And North South length is the same width as East West width. For each square shape mesh, Square Mesh Code is a pair code for the longitude and the latitude. An equation (3) defines longitude code of LonCode, and an equation (4) define latitude code of LatCode. LonCode and LatCode are integer.

$$LonCode = \text{int}\left(\frac{longitude}{2^{13}}\right) \quad (3)$$

$$LatCode = \text{int}\left(\log\left(\frac{1 + \sin(latitude)}{1 - \sin(latitude)}\right) \cdot \frac{a}{2}\right) \quad (4)$$

Here, "a" is defined for LatCode is 1 when LonCode is 1. That is as follows equation (5).

$$a = 469,367.1234 \quad (5)$$

In Addition, an expand mesh is defined, because it usually wants to use longer section to calculate IRI. An expanding mesh is defined by 2 times, 4 times, 8 times, 16 times width of original mesh one. Square Mesh code is defined by array of (MeshSize, LatCode, LonCode). It shows in Figure 6.

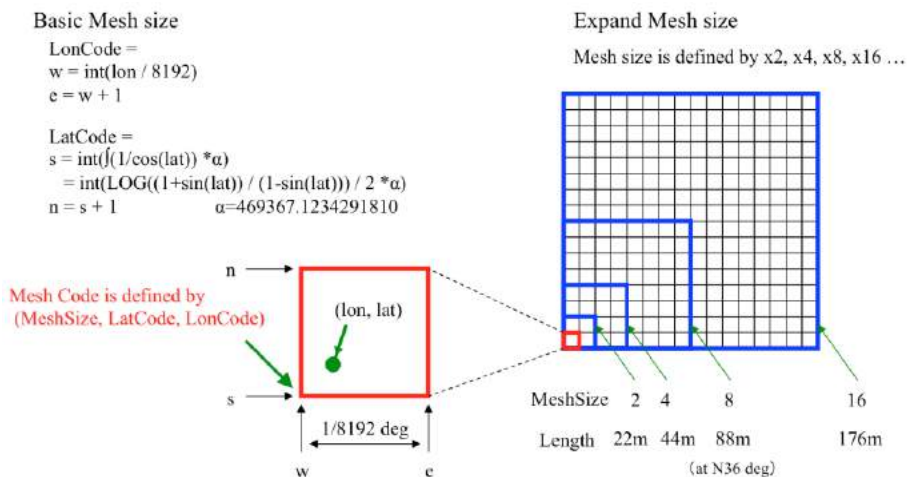


Figure 6. Definition of the Square Mesh Code

4 MEASUREMENT CONDITIONS AND RESULTS

Figure 7 shows roughness measurement roads. It is around Kumamoto prefecture, collected after Kumamoto earthquake. This map has 200km long for south to north, and 260km width for the west to the east. The brown line indicates measurement roads which have over 3,100km long. Measurement roads are located around the epicenter, referred to Figure 2.

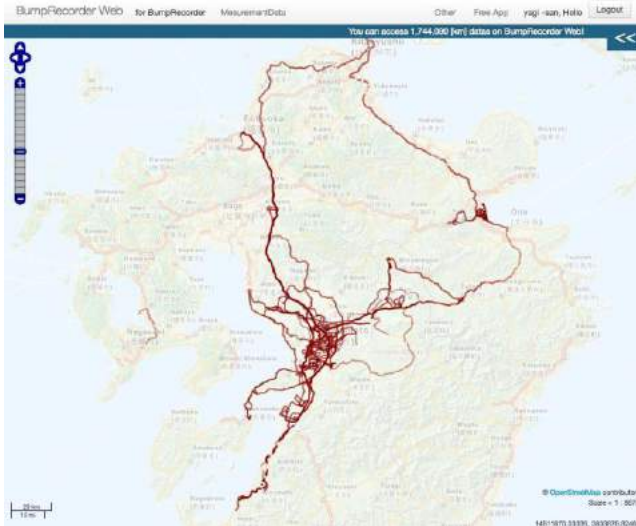


Figure 7. Roughness measurement road around Kumamoto prefecture

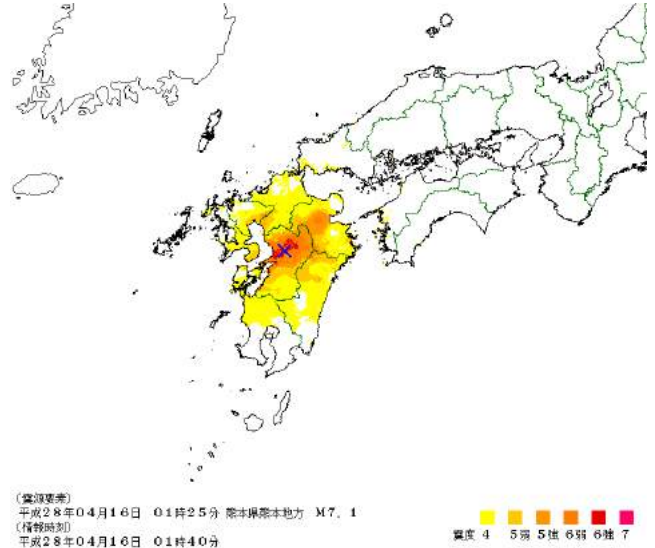


Figure 2. Seismic intensity map (reshown)

Figure 8 and Figure 9 were taken in Mashiki Town during roughness measurement driving. Mashiki Town is located at the Kumamoto earthquake epicenter, and it had biggest damages. Figure 8 indicates a left house originally with two floors, but its ground floor is collapsed down. Figure 9 shows a left house also damaged, its ground floor and wall. that is dangerous to pass its side, so, the left turn lane was closed.

Figure 10 and Figure 11 shows the then Mashiki Town too. Here, pavements are peeled. In Figure 11, on the right hand side of the road, the water drain is floating up, as the result there are bump steps on this intersection.

Figure 12 and Figure 13 shows the then Aso City. It is located at the north east of the epicenter. As for the big bum step of Figure 12, in order to it, the pavement is peeled and graveled. Figure 13 shows a temporally bump step sign for drivers safety, which is continued about 7km long. Driving speed is slowed down.



Figure 8. House collapse at Mashiki Town



Figure 9. House collapse at Mashiki Town



Figure 10. Pavement peeling at Mashiki Town



Figure 11. Pavement peeling at Mashiki Town

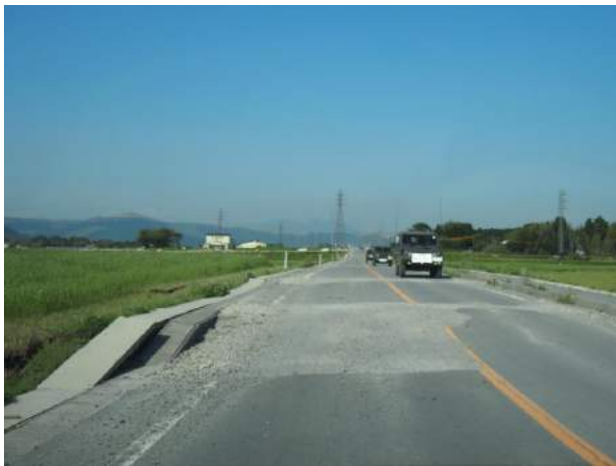


Figure 12. Bump step and pavement peeling at Aso City



Figure 13. A bump step sign at Aso City

Figure 14 indicates the roughness measurement result around Mashiki Town, located near the epicenter. On this map, the line color indicates road roughness, which is the Japanese roughness standard index "Flatness Index" like IRI. light blue lines means most flat conditions, blue, green, brown lines do rougher roads, and red lines mean the most rough roads. In addition, triangular shapes indicate bump steps. Large triangular means large bump steps. This map indicates current roughness situations. However, according to this map, it is a little bit difficult to understand which roughness and/or bump step is affected by the earthquake. That is because some roads have been rough before the earthquake, for example around manholes, or bridges, etc.

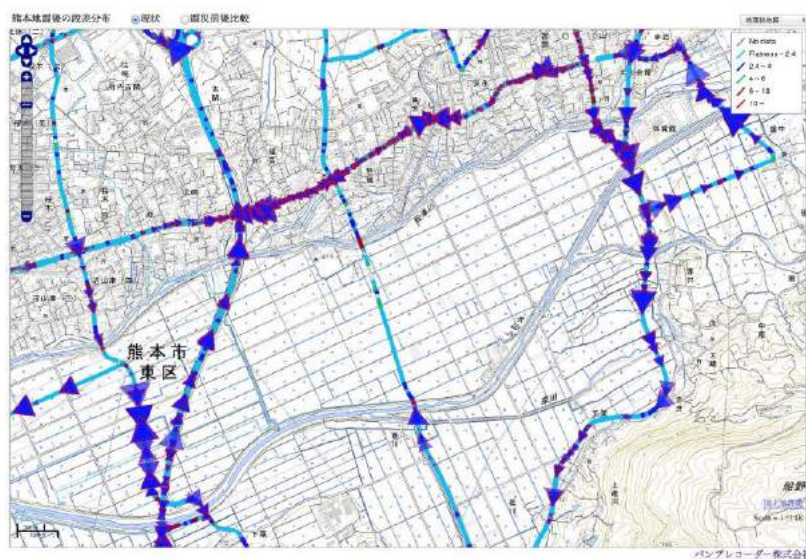


Figure 14. Current roughness and bump step distribution map

5 ROUGHNESS COMPARISON BETWEEN BEFORE AND AFTER EARTHQUAKE

BumpRecorder Co., Ltd is measuring road roughness with the global IRI and the Japanese standard "Flatness" from 2011. Figure 15 indicates the measured roads by BumpRecorder application that has already spread all over Japan. So the roughness data before the Kumamoto earthquake happened was already acquired. Using this data, roughness comparison is done between before and after the Kumamoto earthquake.

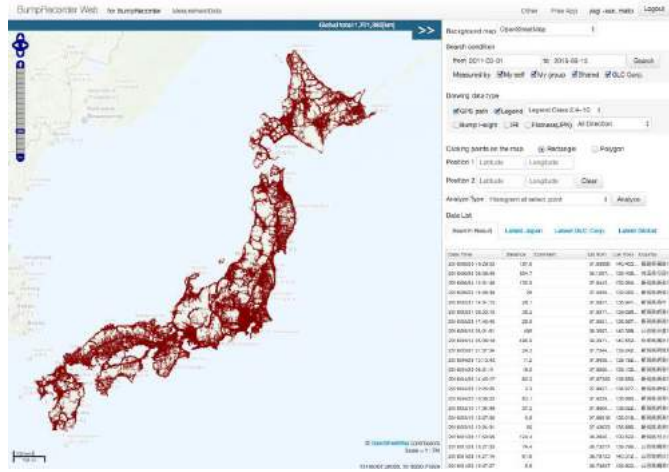


Figure 15. IRI measured road in Japan by BumpRecorder application

Figure 16 shows the roughness comparison results. In this figure, the line width indicates roughness by the Japanese standard index, "Flatness Index" which is like IRI. The bold line indicates rougher road and the thin line does flat roads. Red lines show the data after the earthquake. Blue lines are overlapped on red lines for the data already acquired before the quake. When the roughness situation is the same between before and after the quake, the both lines are the same width. Then only blue lines can be shown. When the road has damages by this earthquake, red lines become wider than blue lines, as the result, the red color can be shown on this map. It is easy to understand the disaster effect on the roads.

Compared with Figure 14, current roughness drawing, Figure 16 of the before and the after comparison is easily understood where the quake has damaged. For example, the green arrow point has a bump step, but it is not the quake affect, which is recognized by Figure 16, but difficult to see by Figure 14.

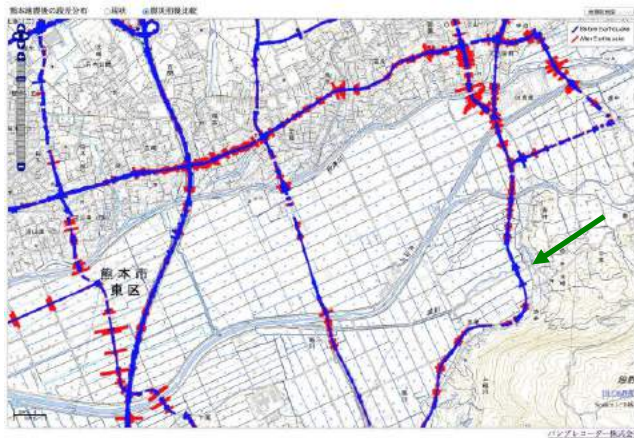


Figure 16. Roughness comparison between before and after earthquake

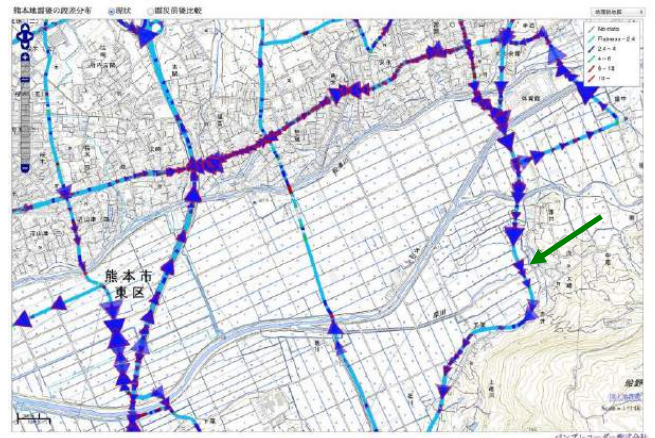


Figure 14. Current roughness (reshown)

Figure 17 and Figure 18, show the roughness comparison result of wider areas around Mashiki Town. Figure 17 has 20km width for the west to the east, and Figure 18 shows 40km width. Look at this figure, road damages are spreading out the south west direction.

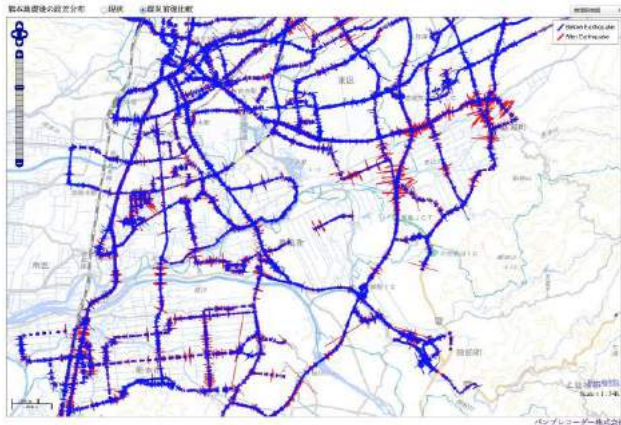


Figure 17. Roughness comparison around Mashiki Town and neighbouring cities and town



Figure 18. Roughness comparison around central Kumamoto

Figure 19 shows the current roughness distribution map of Aso City located at the north east from the epicenter. In this city, after the earthquake, there is measurement data on many main roads. But there is only few data for before the earthquake. For this reason, Figure 20 shows the fact that two roads can only be made roughness comparison map. According to this result, the regular and frequent measurement is very important to understand pavement situations.

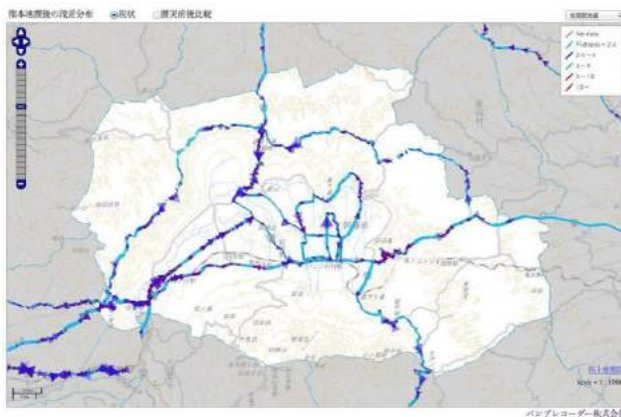


Figure 19. Current roughness distribution at Aso City



Figure 20. Roughness comparison at Aso City

7 CONCLUSIONS

Kumamoto earthquake occurred on April 14 and 16, 2016. The epicenter was in Mashiki Town, and the area strongly damaged by the quake was spreading south west and the north east area from the epicenter. After the earthquake, IRI data was collected from over 3,100km roads in a week. During the inspection, smartphone type roughness measurement application was used. It is easy and low cost to collect roughness data, so it is possible to collect long distance in a short period.

Current roughness data is useful for the pavement maintenance management. However, because the road structure is not sometimes flat, for example manholes and/or abuts of bridge etc., roughness becomes large. That is not damaged by the earthquake. To understand the affect by the earthquake clearly, roughness comparison has been done by using previous measurement data, which enabled us easily to understand where has been damaged. Unfortunately, in some places, for example in Aso City, creating roughness comparison distribution map was impossible because of lacking previous data, which clearly indicates the importance of frequent measurement. It is very important to understand current road conditions. The existing method of inertial profiler is difficult to use because of the high operation cost.

Smartphone type roughness measurement is a powerful tool to collect roughness data. It is easy and low cost. At Kumamoto earthquake, smartphone roughness measurement clearly indicates the possibility to collect roughness data immediately in a short period.

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Energy Usage and Greenhouse Gas Emissions of Pavement Preservation Processes for Asphalt Concrete Pavements

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ABSTRACT

Use of pavement preservation treatments extends the remaining service life of asphalt concrete pavements. These treatments typically include spray applied surface seals, thin overlays, crack treatments, chip seals, slurry seal/micro surfacing, surface recycling and others. Each preservation treatment reduces damaging effects of aging and deterioration of the pavement surface layer and helps protect the integrity of the underlying pavement structure. If proactive preservation treatments are not used, pavements deteriorate more rapidly and require major rehabilitation with structural overlays or reconstruction much earlier.

Every type of pavement strategy requires a series of energy using processes that impacts greenhouse gas emissions. Pavement rehabilitation and reconstruction require large amounts of energy to obtain and process raw materials, transport, mix and apply the final product, while pavement preservation processes require much less energy to apply the final product to the road surface. This paper presents information on energy usage per unit area by comparing pavement life extensions of pavement preservation treatments to typical design lives of reconstruction and rehabilitation techniques. Results show that pavement preservation treatments have significantly reduced energy use and greenhouse gas emissions compared to traditional rehabilitation and reconstruction strategies.

KEYWORDS

Pavement Preservation, Energy Usage, Sustainability, Green House Gasses, Asphalt Concrete Pavement

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INTRODUCTION

Construction, rehabilitation, and maintenance of highway pavements require obtaining, processing, transporting, manufacturing, and placement of large amounts of construction materials. These activities use substantial amounts of energy and generate green house gasses (GHG). Differing philosophies have existed, and still exist, on the proper approach of managing, rehabilitating, and maintaining pavements. Methods range from one extreme of allowing the pavement to deteriorate and then reconstructing; to using preservation treatments to minimize effects of aging and maximize pavement life. Vastly different amounts of energy are consumed with different construction, rehabilitation, and preservation techniques. These various techniques also provide differing amounts of pavement design lives and life extensions. For each preservation treatment the life extension can be compared to the required energy and GHG emissions to determine an annualized energy use and GHG emission level. To minimize energy and GHG emissions over the life of the pavement, treatments can be chosen as having the lowest annualized energy use and GHG emissions.

LITERATURE REVIEW

Energy use and GHG emissions for the construction industry have been receiving increasing attention in recent years. The terms “Green”, “Sustainable Development”, “Environmental Impact”, “Energy Efficiency”, “Global Warming”, “Green House Gases”, and “Eco-efficiency”, are becoming more widely recognized and used.

For buildings, the Leadership in Energy and Environmental Design (LEED) system has been developed to aid in design and construction to minimize environmental impacts. The LEED-ND (for Neighborhood Development) system includes some basic paving considerations in the analysis for multi-unit developments (US

Green Building Council, 2008). The Greenroads system has been developed as a method to assess roadway sustainability. Greenroads enables owners, consultants and contractors to make informed decisions by providing a sustainability performance metric for roadway design and construction. The system defines roadway sustainability attributes, provides a system for evaluation of roadway sustainability, and includes a collection of sustainable design and construction practices. The system includes 11 project requirements, including items ranging from having pavement preservation and environmental maintenance plans to construction quality control and life cycle cost analysis. Credit can be given for several pavement technologies including warm mix asphalt, cool pavements, and quiet pavements, to name a few. Additional voluntary credits are available that can be added to produce a final Greenroads score. The score can be used for tracking and evaluating roadway project and system sustainability (Greenroads, 2009). BASF has developed an Eco-efficiency analysis method that can be applied to many products or systems (Uhlman, 2009). The process considers and evaluates six aspects of a system including raw materials, land use, energy consumption, emissions, toxicity potential, and risk potentials. This procedure has been used to compare eco-efficiency of several paving processes including hot mix overlays, micro-surfacing, and chip seals (Wall, 2004). Cold mix systems, such as micro-surfacing were found to use less energy and to be more eco-efficient than hot-mix asphalt concrete, and emulsion chip seals were found to require less energy and be more eco-efficient than hot-applied chip seals. The publication "Road Rehabilitation Energy Reduction Guide for Canadian Road Builders" (Canadian Construction Association, 2005) was developed to provide information on methods to reduce energy usage during road construction and maintenance operations. Suggestions are provided for reducing energy use during plant operations and construction operations. Chappat and Bilal (2003) reported an in-depth analysis of energy consumption and GHG emissions of over 20 different paving product types by ton of material placed. Their comparisons show that PCC paving materials and processes demand the most energy, followed by hot mix asphalt (HMA) paving. The report also showed that cold in-place recycling (CIR) is the least energy intensive process. Dorchie (2008) reported on software that has been developed to quantify energy use and GHG emissions for various pavement structures based on material types and quantities. Terrel and Hicks (2008) analyzed energy use for hot in-place recycling (HIR) and determined the process utilizes less energy than hot mix asphalt (HMA) paving. Miller and Bahia (2009) in a report on sustainable pavements revealed that proactive maintenance is the least energy intensive process because minimal improvements are made to the pavement structure and surface course. The authors suggest that cold process patching and surface treatments are the most energy efficient.

Extensive analysis of energy use and GHG emissions for the major construction processes was frequently mentioned in the literature review. For preventive maintenance processes, there is limited reporting of energy use and GHG emissions for several treatments with suitable conclusions. However, available reports do not always use the same base data and analysis methods, so comparisons between processes cannot readily be made.

ENERGY USE AND GHG EMISSIONS FOR CONSTRUCTION MATERIALS

When determining energy use and GHG emissions of various preventive maintenance treatments, the first issue is to determine the components of the process to measure. Some comparisons have been reported which only consider parts of the process, such as manufacturing or product placement. These comparisons can lead to misleading conclusions. A more accurate and realistic measure of energy use and GHG emissions of a specific type of work, is to begin with obtaining the raw materials from the earth and adding all the operation steps, such as transport, refining, manufacturing, mixing and placement. Table 1 was compiled by Chappat and Bilal (2003) of energy consumption and GHG emissions for various construction products. The following discussions of energy use for materials and processes are based on information from Table 1.

Table 1
Energy Use and GHG Emissions for Pavement Construction Materials
(Chappat and Bilal, 2003)

Energy consumed and greenhouse gases emitted during the manufacture of one ton of finished product from extraction (quarry, oil deposit, etc.) until the sale at the production unit (refinery, cement plant, etc.)			
Product	Energy (MJ/t)	CO ₂ (kg/t)	Data Source
Bitumen	4,900	285	Eurobitume
Emulsion 60%	3,490	221	Eurobitume
Cement	4,976	980	Athena & IVL
Hydraulic Road Binder	1,244	245	CED
Crushed Aggregates	40	10	Athena & IVL
Pit-Run Aggregates	30	2.5	Athena & IVL
Steel	25,100	3,540	Athena & IVL
Quicklime	9,240	2,500	IVL
Water	10	0.3	IVL
Plastic	7,890	1,100	IVL
Fuel	35	4.0	IVL
Production of Hot Mix Asphalt	275	22	IVL
Production of Warm Mix Asphalt	234	20	IVL
Production of High Modulus Asphalt	289	23	IVL
Production of Cold Mix Plant	14	1.0	IVL
Surface milling of Asphalt for RAP	12	0.8	IVL
In-situ Thermo-Recycling	456	34	Colas MM
In-situ Cold Recycling Stabilization	15	1.13	IVL
In-situ Soil Cement Stabilization	12	0.8	IVL
Laying of Hot Mix Asphalt	9	0.6	IVL
Laying of Cold Mix Materials	6	0.4	IVL
Cement Concrete Road Paving	2.2	0.2	IVL
Lorry Transport (km/t)	0.9	0.06	IVL

Materials

Most materials used in asphalt pavement construction, rehabilitation, and maintenance processes consist of aggregates, of various gradations, and asphalt binders of different performance grades. The total energy used is obtained by starting with the raw material extraction and progressing to transportation and processing/refining.

Aggregates

Energy consumption for aggregate production includes the quarrying, hauling, crushing, and screening. Energy consumption for aggregate production ranges from 25,850 to 34,470 BTU/t (30 to 40 MJ/t), and GHG emissions range from 5 to 20 lb CO₂/t (2.5 to 10 kg CO₂/t).

Asphalt

Energy consumption for asphalt binder production includes crude oil extraction, transport, and refining. Energy consumption for asphalt binders has been determined to be 4.2 mmBTU/t (4900 MJ/t), and GHG emissions are 570 lb CO₂/t (285 kg CO₂/t). For asphalt emulsions, energy consumption is 3.0 mmBTU/t (3490 MJ/t) and GHG emissions are 442 lb CO₂/t (221 kg CO₂/t).

Manufacturing

Manufacturing includes all steps involved with handling, storing, drying, mixing, and preparation of materials for installation. Energy consumed varies depending on the specific material or product type. Typical manufacturing products for highway use include hot mix asphalt (HMA), cold mix, crack sealant, and drying surface dressing aggregate. Production of HMA consumes 237,000 BTU/t (275 MJ/t) and produces 44 lb CO₂/t (22 kg CO₂/t). Warm mix asphalt production, as reported in Table 1, consumes 201,000 BTU/t (234 MJ/t), approximately 15% less than HMA. It is noted that there are several warm mix asphalt processes for which energy use varies depending on required production temperatures. Cold mix asphalt production only requires 12,000 BTU/t (14 MJ/t) because of not needing to heat the aggregate to elevated mixing temperatures.

Transport to Work Site

The produced construction materials must be transported to the work site. Energy consumed on transport varies with the distance and the quantity of material moved. Transport energy has been reported as 1,250 BTU/t-mile (0.9 MJ/km-t) with 0.2 lb CO₂/t-mile (0.06 kg CO₂/km-t).

Placement and Construction

Placement and construction consists of all activities required to install the materials or products. This includes traffic control, site and product preparation, compacting, finishing, clean up, waste disposal, etc. The highest energy consuming process for placement is hot in-place recycling (HIR) at 393,000 BTU/t (456 MJ/t) with 68 lb CO₂/t (34 kg CO₂/t) of GHG. This is due to the required heating to soften and reclaim the existing pavement. Placement of asphalt concrete and cold mixes require between 5,170 and 7,750 BTU/t (6 to 9 MJ/t) with 0.8 to 2.2 lb CO₂/t (0.4 to 1.1 kg CO₂/t) of GHG. Placement energy for PCC is the lowest at 1,900 BTU/t (2.2 MJ/t) with 0.4 lb CO₂/t (0.2 kg CO₂/t) of GHG.

Total Energy Use and GHG Emissions

Tables 2 and 3 are summaries of total energy use and GHG emissions for raw materials, manufacture, transport, and placement of various construction products (Chappat and Bilal, 2003). The data shows that Portland cement concrete pavements use the highest energy consumption at approximately 860,000 BTU/t (1000MJ/t) with the highest energy demand being required for manufacture of the cement. Asphalt concrete utilizes less energy at 586,000 BTU/t (680 MJ/t), with the majority of energy being required for manufacture of the asphalt cement and heating during the hot mix production process. Processes that use unheated aggregate and cold applied binders utilize the least amount of energy per ton.

Table 2

Total Energy Use for Pavement Construction Materials (Chappat and Bilal, 2003)

Energy Consumption (MJ/t) for Each Type of Product						
Product	Binders	Aggregates	Manufacture	Transport	Laying	Total (MJ/t)
Bituminous Concrete	279	38	275	79	9	680
Road Base Asphalt Concrete	196	36	275	75	9	591
High Modulus Asphalt Concrete	284	38	289	79	9	699

Warm Mix Asphalt Concrete	294	38	234	80	9	654
Emulsion Bound Aggregate	227	37	14	81	6	365
Cold Mix Asphalt	314	36	14	86	6	457
Cement-Bound Materials	200	32	14	67	6	319
Cement-Bound Materials & AJ	203	32	14	67	6	323
Aggregate w/Hydraulic Road Binder	50	29	14	61	6	160
Aggregate w/Hydraulic Road Binder & AJ	54	29	14	61	6	164
Cement Concrete Slabs without Dowels	598	40	14	84	2.2	738
Continuous Reinforced Concrete	1,100	29	14	81	2.2	1,226
Untreated Granular Material	0	40	-	68	6	113
Soil Treated In-situ w/Lime + Cement	63	0	-	7	12	81
Thermo-Recycling	98	4	-	12	456	570
Concrete Bituminous w/10% RAP	250	35	275	73	9	642
Road Base Asphalt Concrete w/20% RAP	157	33	275	64	9	538
Road Base Asphalt Concrete w/30% RAP	137	39	275	58	9	510
Road Base Asphalt Concrete w/50% RAP	98	25	275	47	9	454
Emulsion In-situ Recycling	105	4	-	15	15	139

Table 3
Total GHG Emissions for Pavement Construction Materials (Chappat and Bilal, 2003)

Greenhouse Gas Emissions (kg/t) for Each Type of Product						
Product	Binders	Aggregates	Manufacture	Transport	Laying	Total (kg/t)
Bituminous Concrete	16	9.4	22.0	5.3	0.6	54
Road Base Asphalt Concrete	11	7.6	22.0	5.3	0.6	47
High Modulus Asphalt Concrete	17	9.4	23.1	5.0	0.6	55
Warm Mix Asphalt Concrete	17	9.4	20.5	5.3	0.6	53
Emulsion Bound Aggregate	14	9.4	1.0	5.4	0.4	30
Cold Mix Asphalt	20	9.1	1.0	5.7	0.4	36
Cement-Bound Materials	39	5.7	1.0	4.5	0.4	51
Cement-Bound Materials & AJ	40	5.7	1.0	4.5	0.4	51
Aggregate w/Hydraulic Road Binder	10	5.1	1.0	4.1	0.4	20
Aggregate w/Hydraulic Road Binder & AJ	10	5.7	1.0	4.5	0.4	22
Cement Concrete Slabs without Dowels	118	9.6	1.0	5.6	0.2	134
Continuous Reinforced Concrete	188	5.1	1.0	5.4	0.2	200
Untreated Granular Material	0	9.6	-	4.5	0.4	15
Soil Treated In-situ w/Lime + Cement	12	-	-	0.5	1.1	14
Thermo-Recycling	6	1.0	-	0.8	34.2	42
Concrete Bituminous w/10% RAP	15	8.6	22.0	4.9	0.6	51

Road Base Asphalt Concrete w/20% RAP	9	7.8	22.0	4.3	0.6	44
Road Base Asphalt Concrete w/30% RAP	8	7.0	22.0	3.9	0.6	41
Road Base Asphalt Concrete w/50% RAP	6	5.2	22.0	3.1	0.6	37
Emulsion In-situ Recycling	7	1.0	1.1	1.0	0.4	10

ENERGY CONSUMPTION AND GHG EMISSIONS FOR CONSTRUCTION, REHABILITATION, AND PRESERVATION PROCESSES

Different types of pavement construction, rehabilitation, and preservation operations consume different amounts of energy. Energy use and GHG emissions per ton of product provide only a relative comparison of products. The specific pavement structure or work type together with the actual quantities of materials must be evaluated to more accurately compare energy use and GHG emissions for construction, rehabilitation and preservation. Dorchies (2008) performed several comparisons for different structured pavement sections, and determined that for different structures yielding the same structural performance, energy use and GHG emissions can vary as much as 80%.

For some pavement preservation treatments, including thin HMA overlays and HIR, energy use and GHG emissions are available. There have been some specific comparisons performed for various types of chip seals and for micro-surfacing. No references could be found for fog sealing and crack treatments. To provide uniform comparisons, the information developed by Chappat and Bilal (2003), from Tables 1, 2, and 3 was used to calculate energy use and GHG emissions for typical preservation treatments. Energy use and GHG emissions were calculated per unit area of the pavement surface, using typical quantities of raw materials for each treatment. Preservation treatments considered include the HMA overlay, HIR, chip seal, micro-surfacing/slurry seal, crack fill, crack seal and fog seal. For some treatments, several different application rates of the treatment were considered. Table 4 shows calculated energy use and GHG emissions for these pavement preservation treatments. The analysis of energy use and GHG emissions considered the entire process for each treatment including raw materials, transport, processing, mixing and installation as appropriate. Further details on energy determinations are listed in the following discussions for each treatment type. For comparative purposes, Table 5 shows energy and GHG emissions for typical pavement construction and rehabilitation work types.

Table 4

Total Energy Use and GHG Emission for Pavement Preservation Treatments

TREATMENT	DETAILS	ENERGY USE		GHG EMISSIONS	
		BTU/yd ²	MJ/m ²	lb/yd ²	kg/m ²
Hot Mix Asphalt	1.5" (3.8 cm) thickness	46,300	59	9.0	4.9
	2.0" (5.0 cm) thickness	61,500	77	12.3	6.7
Hot In-place Recycling (HIR)	1.5" (3.8 cm) thickness; 50/50 Recycle/New	38,700	49	7.0	3.8
	2.0" (5.0 cm) thickness; 50/50 Recycle/New	51,300	65	9.0	4.9
Chip Seal	0.44 g/yd ² (2.0 L/m ²) Emulsion, 38 lb/yd ² (21 kg/m ²) Aggregate	7030	8.9	0.9	0.5

	0.35 g/yd ² (1.6 L/m ²) Emulsion, 28 lb/yd ² (15 kg/m ²) Aggregate	5130	6.5	0.7	0.4
Slurry Seal / Micro-surfacing	Type III, 12% Emulsion, 24 lb/yd ² (13 kg/m ²)	5130	6.5	0.6	0.3
	Type II, 14% Emulsion, 16 lb/yd ² (8.7 kg/m ²)	3870	4.9	0.4	0.2
Crack Seal	1 lin.ft./ yd ² (0.37m/m ²), 0.25 lb/ft (0.37 kg/m)	870	1.1	0.14	0.08
Crack Fill	2 lin.ft./ yd ² (0.74 m/m ²), 0.50 lb/ft (0.74 kg/m)	1860	2.0	0.25	0.14
Fog Seal	0.05 gal/yd ² (0.23 L/m ²), 50/50 Diluted Emulsion	250	0.4	0.04	0.02
	0.10 gal/yd ² (0.46 L/ m ²), 50/50 Diluted Emulsion	500	0.8	0.07	0.04
	0.15 gal/yd ² (0.69 L/ m ²), 50/50 Diluted Emulsion	750	1.2	0.12	0.07

Table 5

**Energy Use and GHG Emissions for
Asphalt Concrete Pavement Construction and Rehabilitation**

TREATMENT	DETAILS	ENERGY USE		GHG EMISSIONS	
		BTU/yd ²	MJ/m ²	lb/yd ²	kg/m ²
New Construction	4" (100 mm) HMA over 6" (150 mm) Aggregate Base ¹	156,820	198.5	24.1	13.1
Major Rehab Hot Mix Asphalt	4" (100 mm) Overlay ²	112,800	142.8	20.9	11.3
	3" (75 mm) Overlay ²	84,600	107.1	15.6	8.5
Major Rehab Warm Mix Asphalt	4" (100 mm) Overlay ²	108,500	137.3	20.5	11.1
	3" (75 mm) Overlay ²	81,400	103.0	15.3	8.3

¹ Data from Dorchies (2005)

² Data from Chappat and Bilal (2003)

The following are descriptions and findings of the pavement preservation work analyzed:

Hot Mix Asphalt (HMA) Overlay

Thin HMA overlays, placed approximately 1.5 to 2.0 inches (3.8 to 5.0 cm) thick, are commonly used as a pavement preservation treatment. GHG data are calculated based on using a 140 lb/ft³ (2240 kg/m³) in-place density. Results are shown in Table 7 for both a 1.5 and 2.0 inch (3.8-5.0 cm) thickness. The 1.5 inch (3.8 cm) thickness uses 0.079 t/yd² (86 kg/m²) and the 2.0 inch (5.0 cm) thickness uses 0.105 t/yd² (114 kg/m²). The analysis used an energy use of 586,000 BTU/t (680 MJ/t) for the entire process.

Hot In-Place Recycling (HIR)

HIR consists of heating, removing and remixing of one inch of the existing pavement surface followed by installation of a new one inch thick asphalt concrete overlay producing a two inch (5.0 cm) thick treatment. For comparison purposes a 1.5 inch (3.8 cm) total thickness is also shown. Energy use basis is 491,000 BTU/t (570 MJ/t). Data are calculated using a 140 lb/ft³ (2240 kg/m³) in-place density.

Chip Seal

Two chip seal treatment designs were analyzed. First, a high quality design using 0.44 g/yd² (2.0 L/m²) of asphalt emulsion with 38 lb/yd² (21 kg/m²) of aggregate. The second design, a lesser binder application rate of 0.35 g/yd² (1.6 L/m²) with a smaller aggregate gradation of 28 lb/yd² (15 kg/m²). Energy use is calculated including emulsion and aggregate raw materials, transport, and installation.

Slurry Seal/Micro Surfacing

Two slurry seal/micro-surfacing treatment designs were analyzed. First is a typical Type III aggregate, with 12% emulsion and a 24 lb/yd² (13 kg/m²) application rate. The second design is a typical Type II aggregate, with a 14% emulsion and a 16 lb/yd² (8.7 kg/m²) application rate. Energy use is calculated including emulsion and aggregate raw materials, transport, and installation.

Crack Seal

Crack sealing was calculated for a typical pavement cracking density on the basis of one foot of crack sealing per square yard. This density is equivalent to one full length longitudinal crack per lane, and full width transverse cracks spaced at 36 feet (11.0 m). This crack pattern, for a typical lane mile produces 7,040 linear feet (2,146 m) of cracking for the area of 7,040 yd² (5,867 m²) which is one linear ft/ yd² (0.365 m/m²). An installation rate of 5,000 pounds (2268 kg) per day is used. The application yields four linear feet per pound of sealant, producing an installation amount of sealant 0.25 lb/yd² (0.136 kg/m²). Energy use is calculated including raw materials, manufacturing, transport, field heating, reservoir cutting, and installation.

Crack Filling

Crack filling was calculated for a typical pavement cracking density of two feet of crack filling per square yard. This density is equivalent to a crack pattern of two full length longitudinal cracks, and full width transverse cracks spaced at 18 feet (5.5 cm). This crack pattern, for a typical lane mile produces 14,080 linear feet (4,292 m) of cracking for the area of 7,040 yd² (5,867 m²), which is 2 linear ft/ yd² (0.73m/m²). An installation rate of 5,000 pounds (2268 kg) per day is used. The application yields four linear feet per pound of sealant, producing an installation amount of sealant 0.50 lb/yd² (0.272 kg/m²). Energy is calculated including raw materials, manufacturing, transport, field heating, and installation.

Fog Seal

Fog sealing is calculated for three different application rates; 0.05, 0.10, and 0.15 g/yd² (0.23, 0.46, and 0.69 L/m²) of a 50/50 water diluted asphalt emulsion. Energy use is calculated including raw materials, manufacturing, transport, and installation.

New Construction: Hot Mix Asphalt (HMA) Pavement

The structural section for the pavement is 4 inches (100mm) of HMA placed on 6 inches (150mm) of compacted aggregate base course. Energy is calculated including raw materials, heating, mixing, transport, placement, and compaction.

Rehabilitation: Hot Mix Asphalt (HMA) Pavement

Both a 4 inch (100 mm) thick HMA overlay and a 3 inch (75 mm) thick overlay were investigated. Energy is calculated including raw materials, heating, mixing, transport placement, and compaction.

Rehabilitation: Warm Mix Asphalt Pavement

Both a 4 inch (100mm) thick warm mix asphalt overlay and a 3 inch (75mm) thick overlay are examined. Energy is calculated including raw materials, heating, mixing, transport placement, and compaction.

ANNUALIZED ENERGY USE AND GHG EMISSIONS FOR CONSTRUCTION, REHABILITATION AND PRESERVATION PROCESSES

Pavement preservation treatments proactively address the pavement needs and are performed to prolong pavement life. There have been several studies that determined the amount of life extension provided by various pavement preservation treatments. The resulting life extensions have varied widely and are dependent on many factors including environmental factors, timing, treatment design, existing pavement distress, traffic levels, and quality of construction. The range of pavement life extensions for properly design and constructed preservation treatments are shown in Table 6. Pavement life extensions provided by preservation treatments range from one year for fog sealing, up to ten years for thin HMA overlays and HIR. The energy and GHG data must be normalized for the expected pavement life extension to appropriately compare energy use and GHG emissions of preservation treatments. The normalization is accomplished by dividing unit area energy and GHG data from Table 4 by the life extensions in Table 6 to produce annualized results. The annualized results for pavement preservation treatments are shown in Table 7 and for new construction and rehabilitation work types in Table 8. In Table 7. the ranges for energy use and GHG emissions are due to the ranges of life extension times listed in Table 6.

**Table 6
Pavement Life Extensions Provided by Pavement Preservation Treatments**

TREATMENT TYPE	LIFE EXTENSION
Thin HMA Overlay	5 – 10 years
Hot In-Place Recycling	5 – 10 years
Chip Seal	3 – 6 years
Slurry/Micro Surfacing	3 – 5 years
Crack Sealing	1 – 3 years

Crack Filling	1 – 2 years
Fog Sealing	1 year

Table 7
Annualized Total Energy Use and GHG Emission For Pavement Preservation Treatments

Treatment	Details	Pavement Life Extension	Energy Use per Year		GHG Emissions per Year	
			BTU/yd ²	MJ/m ²	lb/yd ²	kg/m ²
Hot Mix Asphalt	1.5" (3.8 cm) thickness	5 – 10 years	4,660 – 9,320	5.9 - 11.8	0.9 - 1.8	0.5 - 1.0
	2.0" (5.0 cm) thickness	5 – 10 years	6,080 - 12,160	7.7 - 15.4	1.2 - 2.4	0.7 - 1.3
Hot In-place Recycling (HIR)	1.5" (3.8 cm) thickness; 50/50 Recycle/New	5 – 10 years	3,870 – 7,740	4.9 - 9.8	0.7 - 1.4	0.4 - 0.8
	2.0" (5.0 cm) thickness; 50/50 Recycle/New	5 – 10 years	5,130 - 10,260	6.5 - 13.0	0.9 - 1.5	0.5 - 1.0
Chip Seal	0.44 g/yd ² (2.0 L/m ²) Emulsion, 38 lb/yd ² (21 kg/m ²) Aggregate	3 – 6 years	1,170 - 2,340	1.5 - 3.0	0.15 - 0.3	0.08 - 0.10
	0.35 g/yd ² (1.6 L/m ²) Emulsion, 28 lb/yd ² (15 kg/m ²) Aggregate	2 – 5 years	1,026 - 2,565	1.3 - 3.3	0.14 - 0.35	0.08 - 0.2
Slurry Seal / Micro-surfacing	Type III, 12% Emulsion, 24 lb/yd ² (13 kg/m ²)	3 – 5 years	1,026 - 1,710	1.3 - 2.2	0.12 - 0.2	0.06 - 0.10
	Type II, 14% Emulsion, 16 lb/yd ² (8.7 kg/m ²)	2 – 4 years	968 - 1,935	1.2 - 2.4	0.10 - 0.20	0.05 - 0.10
Crack Seal	1 lin.ft./ yd ² (0.37m/m ²), 0.25 lb/ft (0.37 kg/m)	1 – 3 years	290 - 870	0.4 - 1.1	0.05 - 0.14	0.03 - 0.08
Crack Fill	2 lin.ft./ yd ² (0.74 m/m ²), 0.50 lb/ft (0.74 kg/m)	1 – 2 years	930 - 1,860	1.0 - 2.0	0.13 - 0.25	0.07 - 0.14
Fog Seal	0.05 gal/yd ² (0.23 L/m ²), 50/50 Diluted Emulsion	1 year	250	0.4	0.04	0.02
	0.10 gal/yd ² (0.46 L/ m ²), 50/50 Diluted Emulsion	1 year	500	0.8	0.07	1.04
	0.15 gal/yd ² (0.69 L/ m ²), 50/50 Diluted Emulsion	1 year	750	1.2	0.12	0.07

Table 8

**Annualized Energy Use and GHG Emissions for
Asphalt Concrete Pavement Construction and Rehabilitation**

Treatment	Details	Design Life	Energy Use per Year		GHG Emissions per Year	
			BTU/yd ²	MJ/m ²	lb/yd ²	kg/m ²
New Construction	4" (100 mm) HMA over 6" (150 mm) Aggregate Base	20 years	7840	9.9	1.2	0.7
	4" (100 mm) Overlay	15 years	7500	9.4	1.3	0.8
Major Rehab Hot Mix Asphalt	3" (75 mm) Overlay	12 years	7050	8.9	1.3	0.7
	4" (100 mm) Overlay	15 years	7210	9.2	1.3	0.8
Major Rehab Warm Mix Asphalt	3" (75 mm) Overlay	17 years	6780	8.5	1.3	0.7

The annualized energy and GHG data for pavement preservation treatments ranges from 250 BTU/yd²-yr (0.4MJ/m²-yr) for a 0.05 g/yd² (0.23 l/m²) fog seal application upwards to 12,160 BTU/yd²-yr (15.4 MJ/m²-yr) for 2.0 inch (5.0 cm) of HMA overlay. Annualized results for the new construction and rehabilitation work types range from 6,780 to 7,840 BTU/yd²-yr (8.5-9.9 MJ/m²-yr). The results group into three categories. The first category includes the thin HMA overlay, HIR, new construction, and rehabilitation, have the highest annualized results ranging from 3,870 to 12,160 BTU/yd²-yr (4.9-15.4 MJ/yr) energy and 0.9 to 2.4 lb/yd²-yr (0.4-1.3 kg/m²-yr) of GHG. The second category includes chip seal, micro-surface, and crack fill at 930 to 2,565 BTU/yd²-yr (1.0-3.3 MJ/yr) energy and 0.13 to 0.35 lb/yd²-yr (0.07-0.20 kg/m²-yr) of GHG. The third and final category includes fog sealing and crack sealing with 250 to 870 BTU/yd²-yr (0.4-1.1 MJ/m²-yr) energy and 0.04 to 0.14 lb/yd²-yr (0.02-0.08 kg/m²-yr) of GHG.

The annualized energy and GHG emission results in Table 7 show that the different pavement preservation treatments provide a year of life extension with differing energy requirements and GHG emissions. Each type of pavement treatment will not always be appropriate for all pavements, distresses, traffic, climate, desired results, etc.

CONCLUSIONS

Comparisons of energy use and GHG emissions for the construction, rehabilitation and preservation of asphalt concrete pavements are calculated and compared. Results show that on an annualized basis, different process types require differing amounts of energy per year of pavement life. New construction, major rehabilitation, thin HMA overlay, and HIR have the highest energy use and range from 5,000 to 10,000 BTU/yd²-yr (6.3-12.6 MJ/m²-yr). Chip seals, slurry seals, micro-surfacing, and crack filling utilize lower amounts of energy per year of extended pavement life and range from 1,000 to 2,500 BTU/yd²-yr (1.3-3.3 MJ/m²-yr). Crack seals and fog seals use the least amount of energy per year of extended pavement life at less than 1,000 BTU/yd²-yr (1.3MJ/m²-yr).

Energy use and GHG emissions for the different products depend primarily on the type and quantity of materials placed per unit area. Products that use lower amounts of asphalt per unit area and products that do not require heating of aggregate use the least amounts of energy. Additionally, products having the lowest quantity of

material applied to the pavement per unit area utilize less energy, simply because not as much material needs to be produced, processed, transported and installed. To minimize energy use and GHG emissions over the life of a pavement, all preservation treatments should be utilized as appropriate to the maximum extent possible for the existing pavement conditions.

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PAPER TITLE	The Effect of Rejuvenators on the Properties of Repetitive Recycled Asphalt		
TRACK			
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KEYWORDS:

Rejuvenator, RAP, Repetitive recycled asphalt, Chemical composition

ABSTRACT:

The technology for recycling usage of reclaimed asphalt pavement (RAP) has been spread widely in 1980's and now, it is authorized in Japan. However, it is not found that the properties of repetitive recycled asphalt, which compared with the property of virgin asphalt mixture, will be effected or not, including the extent of influence. The RAP is estimated to be recycled repetitively and continuously from now on. This study was conducted with aiming to present the suitable rejuvenator, throughout the research of the effect on the properties of recycled asphalt with different types of rejuvenator.

As the results, it is found that the properties of asphalt which is experienced aging and rejuvenation repetitively, would be influenced by the chemical composition of rejuvenator and the ratio of RAP content.

The Effect of Rejuvenators on the Properties of Repetitive Recycled Asphalt

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1 INTRODUCTION

In Japan, the research about recycle use of reclaimed asphalt pavement (RAP) had been started in 1970s. In the 1980s and 1990s, the guidelines and the laws for the recycling technology of the pavement were developed and enforced. After this, full-scale recycling came to be carried out.

By for now, the recycling technology of RAP is widely spread as a general technology. The rate of recycled hot mix asphalt to the total shipments of hot mix asphalt is increasing year by year, in 2013 has reached 75%. In the future, the RAP with experience of recycled repeatedly multiple times will slide into increasing.

In Japan, many studies for the recycling asphalt about the effect on physical and chemical properties in terms of the kinds or the components of rejuvenator have been conducted. It is included recycled asphalt and recycled asphalt mixtures.

However, almost all of these studies were limited on the first recycling of asphalt and the mixture. The physical / chemical properties of repeated recycling asphalt and/or the desirable rejuvenator on the assumption that it is repeatedly reproduced were not clear.

Based on the background above, the series of laboratory tests were conducted aiming to confirm that the effect on the properties of repeated recycling asphalt /asphalt mixture. In this study, the rejuvenator that has large saturation component widely used in Japan was applied. This paper is including the results of these tests.

It should be noted that in this study, the RAP was prepared with accelerated aging by heat in the laboratory after produced with a newly material (hereinafter, the virgin mixtures). Then comparing the properties of recycling asphalt mixtures and recycled asphalts

2 OVERVIEW OF EXPERIMENTS

2.1 Procedure of Experiments

The procedure of experiments in this study is shown in figure 1. In the experiment, repeated aging and recycling of the mixture in the laboratory, physical/chemical properties at the each of aging and recycling asphalt and mechanical properties of recycled asphalt mixture were confirmed.

In the accelerated deterioration, the aging time was set the penetration is reduced to 20 (in the aging procedure, target penetration was 20). In the recycling of asphalt, the rejuvenator was added to make the penetration of aged asphalt to be recovered up to 70 (in the recycling procedure, design penetration was 70).

And also the mixing ratio of RAP in this study was carried out at 60%, taking account for the average of the RAP mixing ratio of each region of Japan to be approximately 30 to 60% (JAMA 2014).

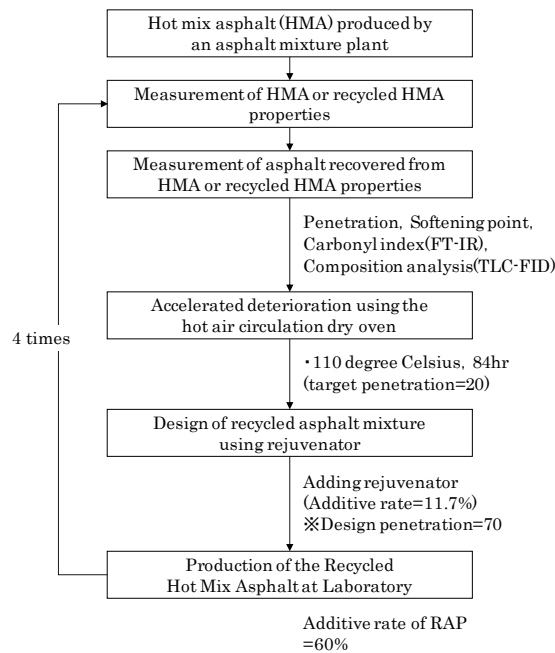


Figure.1 Procedure of experiment

2.2 Materials

The asphalt mixture used in the experiment is dense graded asphalt mixture (the maximum aggregate size is 13mm) with paving petroleum asphalt 60/80 (hereinafter, original asphalt). The optimum asphalt content is 5.3%. The basic physical properties of original asphalt is shown in Table 1.

In addition, the rejuvenator applied in this study is widely using and commercially available in Japan which has a large saturation components. The properties of rejuvenator is shown in Table 2.

Table.1 The properties of original asphalt

Density	Penetration	Softening Point	ductility at 15
g/cm ³	1/10mm	°C	cm
1.038	70	47	100+

Table.2 The properties of rejuvenator

Density	Composition(%)			
g/cm ³	asphaltene	resin	aromatic	saturation
0.966	0	6.3	26.6	67.1

3. DETAILS OF EXPERIMENTS

3. 1 Evaluation of extracted asphalt

The asphalt extracted from the virgin mixture and the recycled mixture (recycling one to four times) were performed physical properties testing and chemical properties tests described below.

It should be noted that extracting asphalt from the mixture, was carried out according to the "G029 asphalt recovery test method (JPI-5S-31-1988)".

3.1.1 Physical Properties Tests

Penetration test and softening test were conducted as physical properties tests

3.1.2 Chemical Properties Tests

a. Asphalt composition analysis test by Thin-Layer Chromatography/Flame Ionization Detector(TLC / FID) method

Asphalt composition components were analyzed by (Ltd.) LSI Medience Co. Iatrosan (MK-6s) thin layer chromatography using a (below, TLC / FID method).

The TLC / FID method, four components separation operation the asphalt in rod-shaped thin adsorption layer on the (TLC), is an analytical method that combines methods (FID) for each of the components is ionized in a hydrogen flame detector. The conditions such as the solvent used are shown in Table 3.

Table.3 TLC/FID Method

Item	Condition
Adsorption layer	Bar-like thin-layer adsorption layer (carrier: silica gel)
Detector	Flame ionization detector (FID detector. The organic component is burnt in a hydrogen flame, detecting the occurrence ion current)
Operation	After spotting the asphalt on the adsorption layer,
	(1) hexane (developing distance 10cm)
	(2) toluene (developing distance 5cm)
	(3) in dichloromethane + methanol mixture (95: 5)
	(Developing distance 2cm)
	Of the deployment in the order, it separated into four components.
	After drying, the respective concentrations were measured by FID detector, determine the concentration ratio.

b. Infrared absorbance measurement

To quantitatively evaluate the progress of the oxidation reaction, oxygen as an index of oxidative degradation of the asphalt by using JASCO Co., Ltd. Co., Ltd. of FT / IR-4000 Fourier transform infrared spectrophotometer (FT-IR) the behavior of the increase-containing functional group (carbonyl group) were measured.

Specifically, by measuring the infrared absorbance of the asphalt, the peak around 1700 cm⁻¹ attributable to the stretching vibration carbonyl group, and the peak around 1600cm⁻¹ attribute to the C = C stretching vibration which is not affected by deterioration were analyzed.

With the ratio of these two peaks, carbonyl index (hereinafter, CI) was determined. Measurement conditions of FT-IR, are shown in Table 4.

Table.4 Measurement conditions of FT-IR

Item	Condition
Absorption spectrometry	On KBr crystal, formed by transmission measurements the thin film of the sample using a solvent
Carbonyl index (CI)	$CI = \frac{\log(I_{o1} / I_1)}{\log(I_{o2} / I_2)}$ <p>I_1, I_2 : transmittance of the peak spectrum in the vicinity of each of 1700cm⁻¹ and 1600cm⁻¹</p> <p>I_{o1}, I_{o2} : each transmittance of the background corresponding to the wave number</p>

c. Molecular weight distribution measurement by gel permeation chromatography

In the measurement of molecular weight distribution, in order to grasp the difference in molecular weight distribution of the asphalt during deterioration, it was measured molecular weight distribution using gel permeation chromatography (GPC). The GPC equipment and operating conditions used are shown in Table 5.

Table.5 Conditions of GPC equipment

Item	Condition
Equipment	LC-10ADvp (Shimadzu Corporation)
Detector	Refractive index detector (Shimadzu RID-6)
Column	TSKgel GMH _{HR} -L(Tosoh)
Mobile phase	tetrahydrofuran (THF)
Mobile phase temperature	20°C
Mobile phase flow rate	1mL/min
Concentration	1.1wt%
The amount of sample	20μL

3.2 Mechanical Properties test of asphalt mixture

Mechanical properties of the virgin mixture and recycled mixture was performed split test (JARA 2010) and the wheel tracking test (JARA 2007). Each tests were conducted each at the virgin mixture, at first time recycled mixture, and at third time recycled mixture.

3.2.1 Accelerate deterioration of asphalt mixture

Accelerated deterioration of the asphalt mixture was carried out using a hot-air circulating drying oven is a device which has become popular in view of the convenience in Japan.

Promote deterioration, asphalt mixture was aged for 84 hours in a hot air circulation drying oven at 110 C. Aging procedure in detail is, break-laying mixture 7.5kg of the state that are not compacted to 36 × 24cm bat, covering the single the surface so as not to be exposed to as much as possible direct hot air in the wide aluminum foil, the edge of the aluminum foil bat as folded.

It should be noted that the curing time of this time (84 hours) is decided so that the penetration of extracted asphalt from the mixture after the deterioration becomes to 20.

3.2.2 Preparing procedure of recycling asphalt mixture

The deteriorated mixture was used as RAP, and reproduced as recycled asphalt mixture, mixing with rejuvenator, new aggregates and new asphalt in a vertical mixer in laboratory.

The reproduce procedure is shown below.

- [1] Curing recycled aggregate (RAP) with the hot air circulation drying oven at 165°C for 3 hours
- [2] Mixing recycled aggregate (RAP) with the vertical-type mixer for 30 seconds
- [3] Mixing after adding a predetermined amount of rejuvenator for 90 seconds
- [4] Mixing after introduce a predetermined amount of new aggregates for 30 seconds
- [5] Mixing after pouring a predetermined amount of put a new asphalt (at 160 °C) for120 seconds
- [6] Discharge the mixture from vertical mixer
- [7] Prepareing the specimen for each tests (for the split test , wheel tracking test) by a suitable compaction machine.

4. RESULTS

4.1 Results of Asphalt

4.1.1 Physical Properties

a. Penetration test

The penetration test results are shown in Figure 2. Asphalt promoted deterioration is recycled to be a design penetration 70 at each stage. Therefore, even when the deterioration and recycling are repeated a plurality of times, the penetration of the asphalt recovered from the mixture after recycled and after accelerated aging are approximately about 20 and 55, respectively.

Therefore the targeted aging and recycling process were seemed to be repeated.

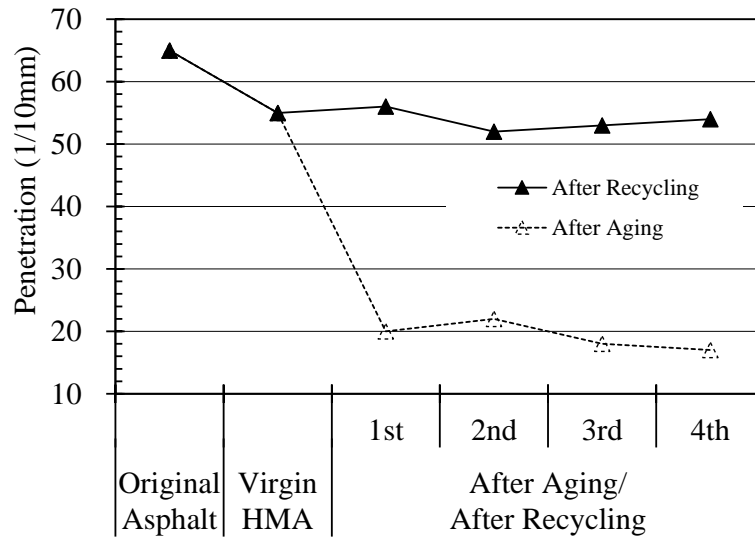


Figure.2 Penetration

b. Softening Point test

The softening point test results are shown in Figure 3. Softening point, aging and recycling tended to be higher in accordance repeated, indicating that the temperature sensitivity of asphalt become dull.

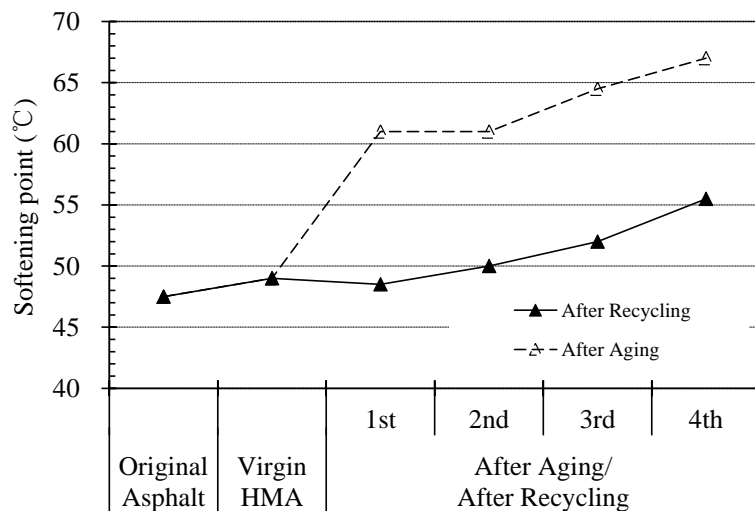


Figure.3 Softening Point

4.1.2 Chemical Properties

a. Asphalt composition analysis test by TLC / FID method

The composition analysis test results of the extracted asphalt from the mixture after the accelerated deterioration are shown in Figure 4. And Figure 5 shows the composition analysis test results of the asphalt, which was extracted from the recycled asphalt mixture. From Figure 4, the composition of the asphalt, which

was extracted from the mixture after the accelerated deterioration, when the accelerated deterioration of times increase, decrease asphaltene and aromatic content, saturated component and resin content tended to increase.

On the other hand, as shown in Figure 5, as in the analysis of the asphalt after accelerated degradation and is repeatedly recycled, decrease asphaltene component and the aromatic content, saturated component and resin content tended to increase.

This means that if the recycled aggregate (RAP) content rate of 60% cannot be compensated by the new asphalt, and consider to be affected by the chemical composition of the rejuvenator.

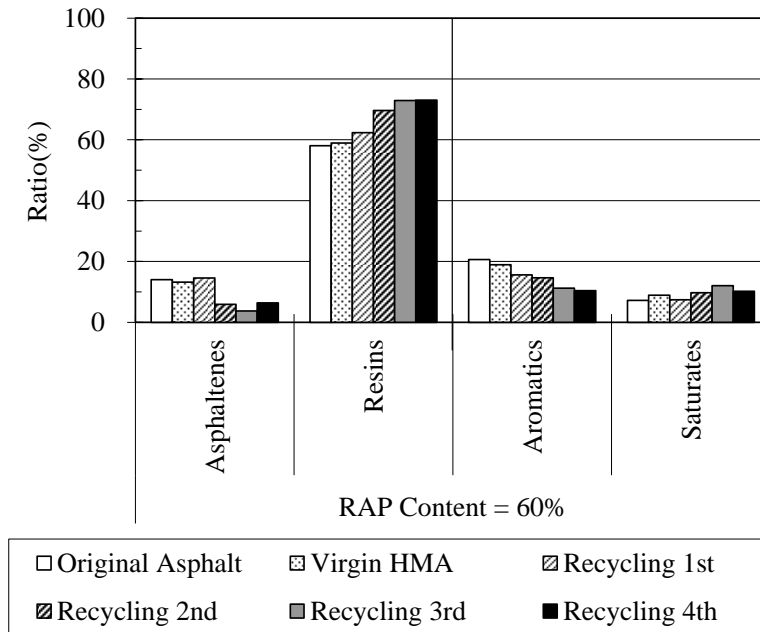


Figure.4 Chemical compositions after accelerated deterioration

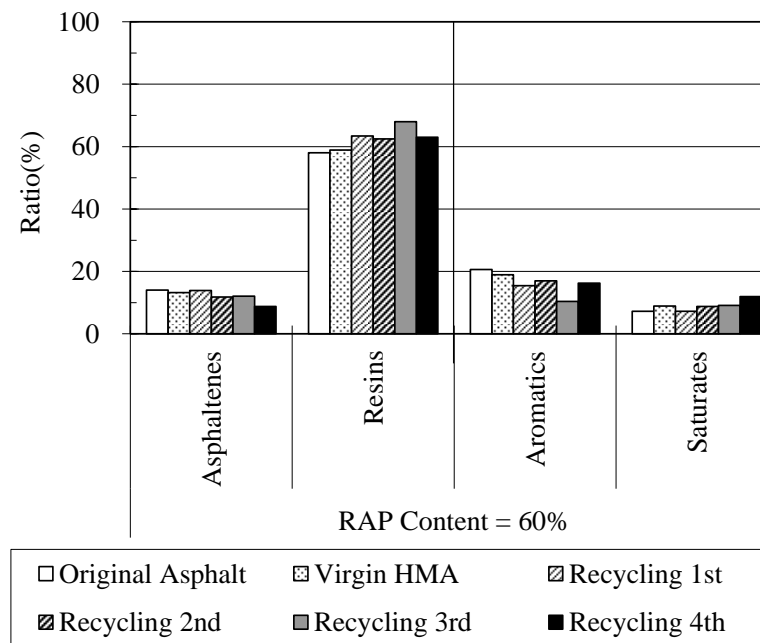


Figure.5 Chemical compositions after recycling

b. Infrared absorbance measurement

The carbonyl index (CI) from the infrared absorbance measurement result of the extracted asphalt from each mixture is shown in Figure 6. The CI of asphalt after accelerated deterioration, has tended to

increase with increasing aging number. Also, the CI of asphalt extracted from recycled mixture has tended to increases with recycling number.

It should be noted that, in studies in Japan (Nitta 2011), CI of asphalt, if recycled aggregate (RAP) blending ratio of about 30%, but small changes even if they are played repeatedly, is a change in accordance with recycled aggregate (RAP) compounded rate increases It indicates that there can be large, but it can be said that the results of this study are consistent with it.

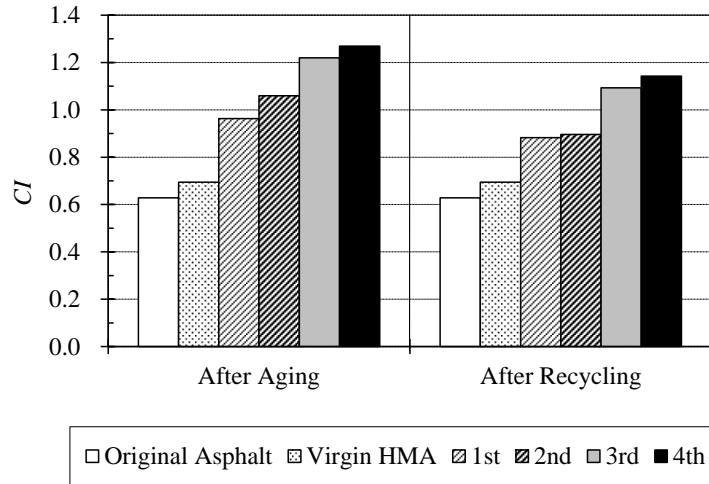


Figure.6 The CI extracted asphalt from aging/recycling

c. Molecular weight distribution measurement by gel permeation chromatography

The molecular weight distribution measurement results of the extracted asphalt from repeated recycling in the recycled aggregate blending ratio of 60% is shown in Figure 7. From results when it becomes much aging number of iterations, a large difference was observed in the peak near a retention time 7.0 ~ 7.2min (molecular weight of about 104), is a tendency that the components of large molecular weight increases were observed.

This trend, since it is the same as the tendency of the CI as shown in Figure 6, the increase in molecular weight in this experiment is considered to have large effect of oxidation. Also, recycled aggregate blending ratio 60% since less is new asphalt to be added at the time of recycling, it is considered a component of increased aging and molecular weight by oxidation asphalt indicating that tends to be accumulated.

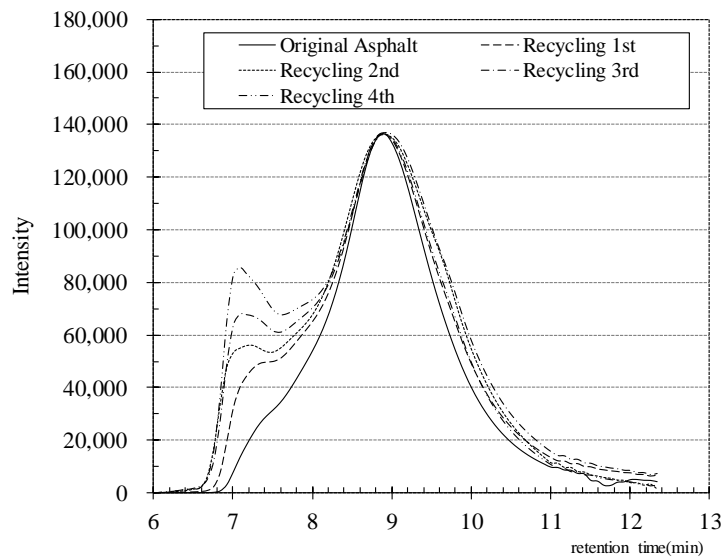


Figure.7 Molecular weight distribution of recycling asphalt

4.2 Results of Asphalt Mixture

a. Split test

Figure.8 shows the result of Split test for virgin mixtures, once, and 3times recycled asphalt mixtures. From the results, split coefficient of 60% recycled aggregate blending ratio tended to increase in accordance with the recycling number.

Therefore, it is considered that when it is recycled repeatedly a plurality of times using rejuvenator contains saturations rich, recycled aggregate blending ratio of 60% recycled mixture would became hard and the fatigue resistance would decrease .

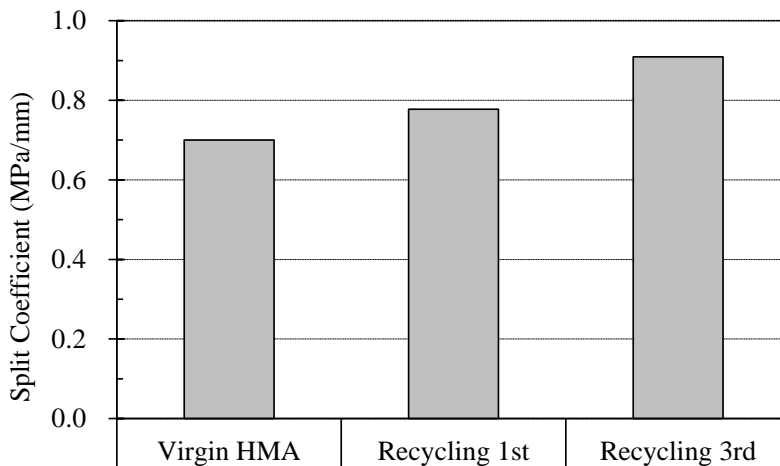


Figure.8 The result of split test

b. Wheel tracking test

Figure 9 shows the result of wheel tracking test for virgin mixtures, once, and 3times recycled asphalt mixtures. From the results, the dynamic stability of 60% recycled aggregate rate became to be larger than the one of virgin mixture. However, the dynamic stability didn't tend to increase in accordance with the recycling number large differences views even increased was observed.

Therefore, recycled aggregate blending rate of 60% of the recycled mixture is considered to tend to become hard even in a relatively high temperature range.

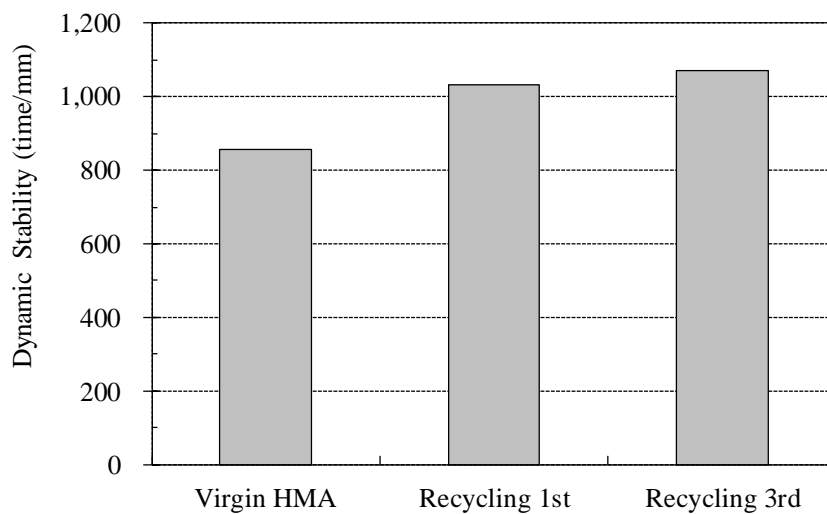


Figure.9 The result of wheel tracking test

5. CONSIDERATION

In this study, it is shown that the physical and chemical properties of the recycled asphalt and recycled asphalt mixture, which is repeated recycling with rejuvenator of saturation rich, behave to change slowly. In the case of the recycled aggregate blending ratio of 60%, since the amount of fresh asphalt being added to recycling mixture is small, physical and chemical properties of recycling asphalt are easy to receive the influence of the properties of the old asphalt and rejuvenator.

Consequently it is considered that the physical and chemical properties of recycled asphalt can become a thing that is different from the original asphalt. Therefore, in recycling the mixture repeatedly with rejuvenator of saturation rich, it will be suitable less recycled aggregate blending ratio.

However this study is as far as the case of using rejuvenator of saturation rich, it is not clear that the effect of rejuvenator component on the properties of repeatedly recycled asphalt or asphalt mixture.

In the future, it is needed to establish the suitable recycling method through comparing the properties of the recycled mixture and recycled asphalt in the case of repeated aging and recycling by using different rejuvenators.

6. CONCLUSION

The conclusion of this study is following.

[1]Repeatedly recycled asphalt has a softening point higher it becomes temperature sensitive resistance tended to decrease.

[2]For the composition of repeating recycled asphalt, the asphaltene and the aromatic tend to decrease according to increase the recycling number.

[3]The CI of repeatedly recycled asphalt tends to increase according to the recycling number.

[4]The molecular weight distribution of asphalt repeated aging and recycling, according to a recycling number count increases, has a tendency that power of a large component (retention time 7.0 ~ 7.2min) to be increase. It is the same as the tendency of CI increases, it is seemed that the increasing in molecular weight in this experiment is considered to have large effect of oxidation.

[5]From the results of split test and wheel tracking test for the repeatedly recycling mixture, it is considered to become "hard",and "low fatigue resistant".

[6]Recycling with the rejuvenator of saturation rich for the recycled aggregate blending ratio 60% , since less is new asphalt to be added at the time of recycling, it is considered a component of increased aging and molecular weight by oxidation asphalt indicating that tends to be accumulated.

[7]In recycling the mixture repeatedly with rejuvenator of saturation rich, it will be suitable less recycled aggregate blending ratio.

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2ND IRF ASIA REGIONAL CONGRESS & EXHIBITION
October 16-20, 2016 – Kuala Lumpur, Malaysia

PAPER TITLE	Long Segment Roads: The Indonesian way to Adopt a Policy of Performance		
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KEYWORDS: Long Segment, Multi Year Contract, PBMC, New Organization			
<p>ABSTRACT: Mid-2015 there was an organizational change in the Directorate General Highways (DGH) from a Territorial to a competency approach, which is the reason for the existence of the Directorate of Road Preservation. This Directorate has the responsibility for Routine Maintenance, Periodic Maintenance, Rehabilitation and Reconstruction of 47,017 km of National Roads in Indonesia. In the same year, the Government also introduced a new Strategic plan 2015-2019 and Key Performance Indices related to road preservation were introduced namely: (1) a reduction in travel time for national roads from 2.6 hours / 100 km, to 2.2 hours / 100 km; (2) increase National Roads in stable condition from 94% to 98%, as no doubt the KPI is directly related to Road Preservation.</p> <p>Target efficiency improvements outlined in the KPI, require that the Directorate of Road Preservation, conduct an internal review of the best practical policy to deliver optimum road preservation to achieve KPI's. The results of the evaluation and review found that as much as 91% of the total road preservation is done by force account while the remainder, only 9%, small-scale contracts rose about 3 to 4 km per contract. An irregularity arose in that road construction is limited to the pavement alone, no consideration being given to drainage facilities. The government noted the failure of Performance Based Maintenance contracts in Java Island conducted several years ago and realized from this that changes especially with respect to preservation, has to be done in stages, rather than only at one time. To achieve KPIs, Long Segments were introduced and within Long Segments changes such as: (1) having long segments of around 50 km to 150 km of road; (2) Multi year Contracts; (3) comprehensive Road Standards, were included. The transition period commences this year 2016, and will continue up to the end of the Strategic Plan in year 2019. The paper will examine the preparation and implementation of Long Segments with particular reference to a Performance Based Maintenance Program.</p>			

SLAMET MULYONO & MAX ANTAMENG

1. Introduction

Implementation of Performance based Contract through a pilot project at 5 places not too successful, because the Government through regulations that there is still not adapting best practices such as those written in documents or document World Bank Latin America. The handling pilot project even though his contract was already reaching out in terms of the length of the road into the scope of the km's contract, but the contract still adapting attention focused only on roughness only not as an output contract. Based on World Bank, Gunter Zacs review found the following:

1. The existence of the fact that the implementation of the pilot project for the Performance Based Contract fail to adapt the international best practices

2. The existence of a common perception that already in place, That the current focus to construct road will be only pave, no consideration to take into account another three important components of the road such as Drainage, Shoulder, and other road equipment
3. During preservation implementation in 2015, it was found that out of 91% of the length of the national road carried out a preservation with using force account. Only 9% of the total length of the national road which was done by means of a contract.

There is a need to improve this current practices, and the most eligible short time solution will be to eliminate both no. 2 and 3 above to approach the objectives on no. 1.

The Development Framework of institutional side can be drawn as follows:

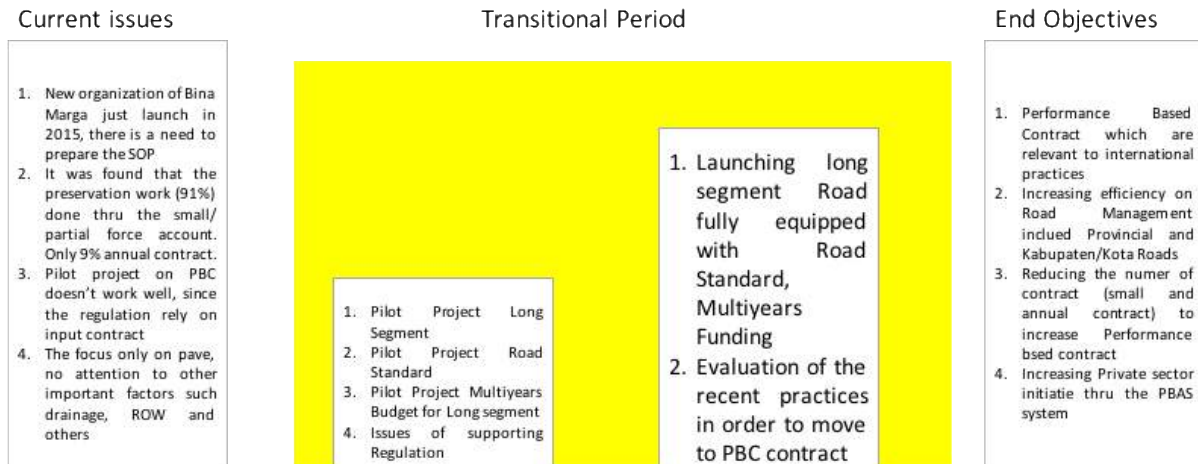


Diagram 1 Current Issues, transitional period and End objectives, source: Writer (2016)

Below diagram shown us effort has been taken by the Government of Indonesia to achieved Performance Based Contract thru several stages:

Long Segment as an Indonesian Way to achieved Performance Based Contract

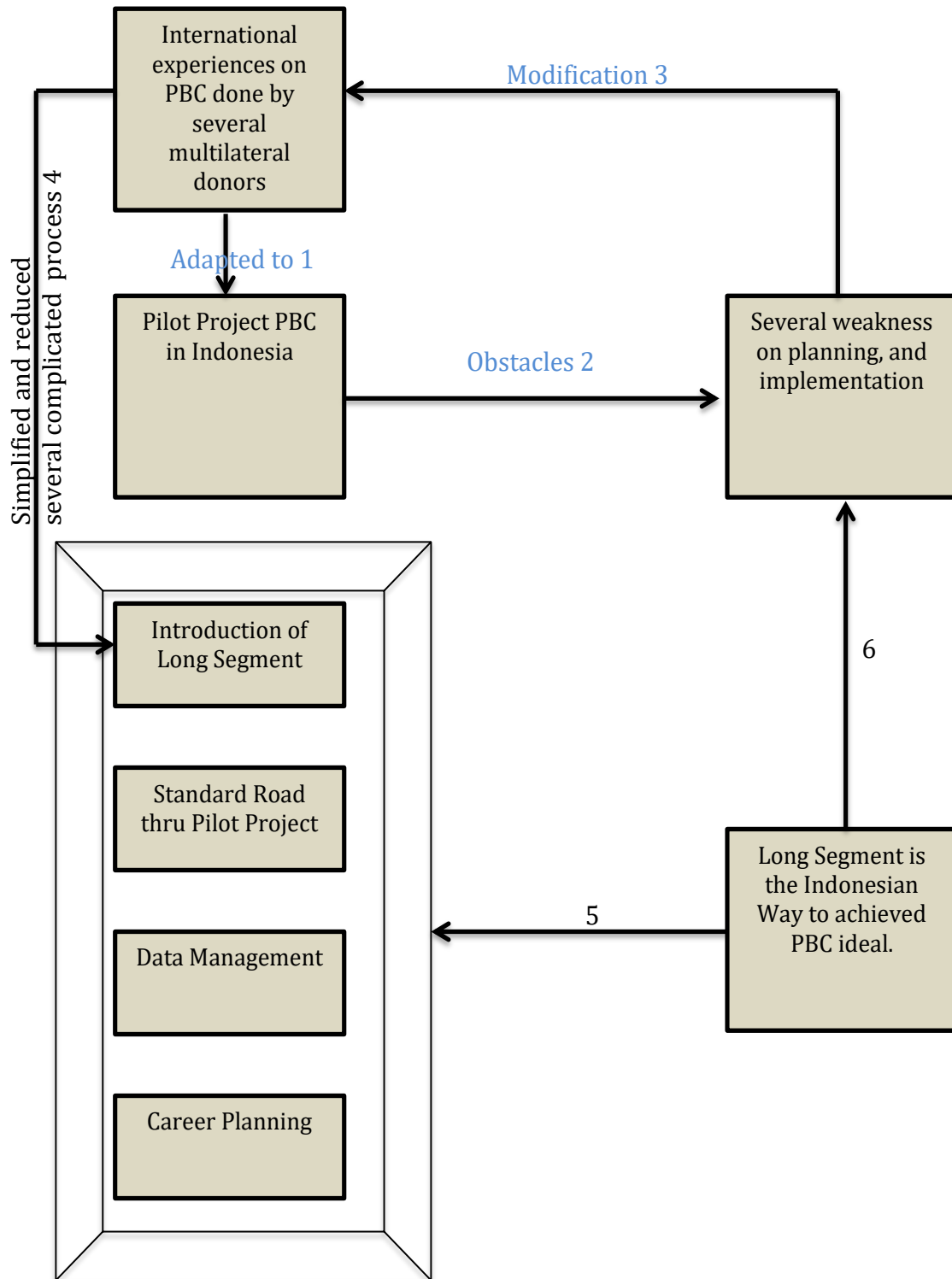


Diagram 2 Long segment as an Indonesian way to achieved PBC, source: writer

2. KPI's of National Road Sector at the end of 2019

Discussion on Performance-Based need to discuss opportunity, and the opportunity was there since 2015 is the beginning of the national strategic plan for the Ministry and also medium-term development plan 2015-2019. The Key Performance Index set by Government on Road Sectors as follows (existing road 47.017 km length):

- National road stability from 94% to be (good + Fair) 98%
- Regional Road Stability (good + fair) from 601% to 70%
- Average travel time in main corridor from 2.6 hours/100 km reduce to 2.2 hours/100 km
- 3,057 km Widening
- 15 km Fly Over / Under pass
- 2,650 km new roads
- 1,000 km toll road
- 500 km support for regional road

From the above KPI's, that are directly related to the preservation of the road i.e. national road stability (good + fair) to 98%. All efforts have to fulfil the target which is already mentioned in the Strategic Plan. Since there is also finance constraints, needs an effective approach, as well as a systematic attempt to hit KPI's by implementing Performance Based Contract to increase level of efficiency, thru the transition period policy with Long Segment, Road Standard, and multiyear budget.

3. Existing Condition of National Road Sector 2015

Existing National Road condition can be seen as follows:

Table 1 National Road Condition in Indonesia, sources: DG Highways 2015

Islands	Road Length (Km)	Pave		Not Paved	
		Length (Km)	%	Length (Km)	%
SUMATERA	13,709.85	13,451.19	98.11%	258.65	1.89%
JAWA	6,534.62	6,534.62	100.00%	0.00	0.00%
KALIMANTAN	7,620.01	6,656.42	87.35%	963.59	12.65%
SULAWESI	8,792.82	8,297.53	94.37%	495.29	5.63%
PAPUA	3,963.11	2,206.27	55.67%	1,756.84	44.33%
BALI, NTB, NTT MALUKU DAN MALUT	6,396.87	5,478.57	85.64%	918.30	14.36%
TOTAL INDONESIA	47,017.27	42,624.60	90.66%	4,392.67	9.34%

As can be seen from table 1 that islands outside Java, part of the road still unpaved, even in Papua, the number of roads that unpaved reached almost half of the total length of the road. This happened, since Government policy on road is always related to the number of traffic pass thru that road. If the number of traffic or AADT below the standard, then imposed Minimum Service Standards, like in Papua, where in this case the number of AADT, is stil far below the spesification.

Table 2 National Road Condition in Indonesia, sources: DG Highways, 2015

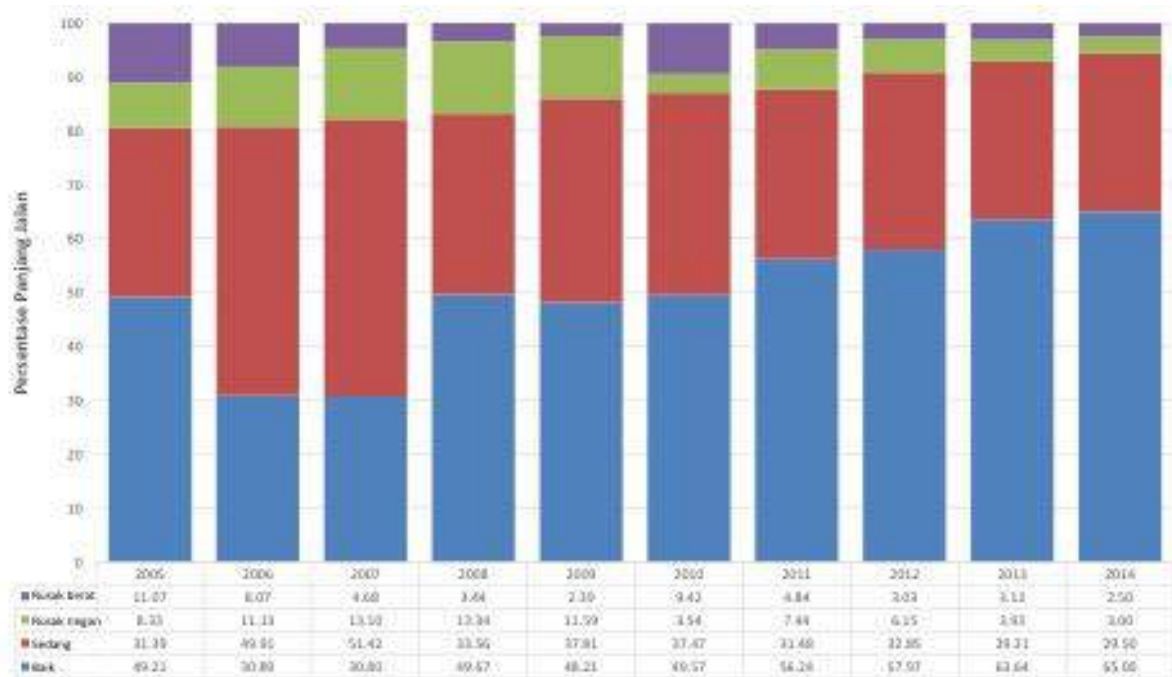


Table 2 shows the trend of National Road Condition from 2005 up to 2014, Blue color show the “Good” road, Red color refer to “Fair” Road, while green color is the light damage and purple means “heavy damage”

Due to budget constraints, in the past, the Government has to focus only on Pave alone at the time and then prioritize on the length of the road or length of road course. Minimum service standards that are on the Road sector, due to the accessibility aspects, leading to areas like Papua, still much in the way that there is no cover. At a time when the volume of traffic had already stepped forward on certain limitations, then the mobility aspect of becoming a benchmark in the development of the road, the road becomes such standards established by Act No. 38/2004, on the road. It's just that, in fact, the situation, build roads just focus on asphalt cover and limited to the road course, it seems to be true for all national roads, few are aware that the road is not just a road but including drainage and other road equipment.

4. Unsuccess story on Pilot Project on Performance Based Maintenance System

DG Highways Indonesia has conducted trials Performance Based contract since 2000's by the Directorate General of Bina Marga, in early 2009, established that the concept of the PBC can be started as pilot project in 2010 in two sections in Java: Ciasem-Pamanukan and Demak - Trengguli. By 2012, another 3 road project established: (1)Semarang-Bawen, (2)Bojonegoro-Padangan, Padangan-Ngawi, and (3) Sei Hanyu – Tb Lahung (Central Kalimantan).

Unfortunately, the result of the review found that the implementation of the pilot project is still undeniably contract input, not PBMC, whereas the government still have to face the risk, while on the best practices, other way around, the private sectors has the responsibility of initiation of PBC. There are several issues related to the PBC, such as:

- Difficulties of Hybrid Performance Based Contract
- Regulatory issues
- Quality management
- Unpredictable maintenance
- Measurement and payment of hybrid

In implementing the pilot project performance based contract, the main one is the quantity or length of the project. Refer to the South Africa, one contract was reached 100 km, then in Indonesia should be around 50 s/d 100 km/contract. Another important point on construct PBC, is the launching of Standard Road, which is including pavement, (2) Drainage; (3) Shoulder and (4) other road equipment.

Almost all ASEAN countries have switched on the Long Term Contract including Performance Based Contract, Countries like Malaysia, Singapore, and Thailand have introduced long term comprehensive contract, which we call as Performance Based Contract. While at the same time we still deal with input and small contracts. To do the reform from far left to far right, has to be done in several stages, the unsucess story of PBC, approved that.

5. Transition Period to achieved Performance Based Contract thru long segment included Standard Road

Each country has a different approach in carrying out the organization of the road, as well s Indonesia, in terms of conducting Performance Based Contract. Although the model of Each country has a different approach in carrying out the Organization of the road, as well as Indonesia, in terms of conducting of Performance Based Contract. Although PBC model documents available everywhere (World Bank, ADB, Latin American experiences). Learning from the result of pilot project, need to achieve the objectives thru several stages. Below are the things that are done as a policy transition towards performance based contract: Standard road, Long Segment

a. Standard Road

Awareness of standard road need to be socialized as standard which must be met on road construction. Based on the Regulation (PP No. 34/2006) has referred to those parts of the road, which should always exist in the planning. Standard means: not only Pavement alone, it has to be in place: Shoulder, Drainage, and other Road Equipment. From the economic point of view, ignoring Drainage, which cost around 7 up to 10% of the total road construction, will cause the early rate deterioration of Road. This is **due** to the number and duration of rain that pooled the asphalt. The Pilot Project of Road Standard implemented in 2015, implemented in each Preservation Project Manager as long as 1 km, per Project Manager. Project Manager will decide which part of their road will be as a pilot project. Once that particular road become pilot project, it has to be fulfil with at least Drainage, Shoulder, and detail of road equipment as mentioned on the regulation

b. Long Segment

Concept of long segment launch in mid-2015 is based on the recent findings, that 91% of preservation funds was done thru force account while 9% use annual small contract. Transition policy is needed, thru the introduction of long segment as an effort to eliminate/minimize force account. The Rationale of applying and introduction of "Long Segment" based upon:

- a. Awareness of the good road maintenance needs
- b. Approximately 98% of national roads need maintenance
- c. Inadequate funding to carry out the maintenance of optimum path resulting in national road conditions vary in the short-short segments
- d. Effectiveness (administer) vs. efficiency (best value)

The Definition of Long Segment as follows: The Long Segment is a way to handle road preservation within the boundaries of one long segment which is can be more than one road section(Sources: Min of PUPR decree No. 19/PRT/M/2011) which was implemented with the aim to obtain a uniform road condition. In Organizing the Long Segment, include the following activities: Widening, Reconstruction, Rehabilitation and Road Maintenance.

In the year 2016, the introduction of Long Pavement is still limited and is done thru the technical aspects which is condition that is available. One example: to design a long segment of the job picture is done with the incorporation of the roads will be DED merged so the length of the segment, and is done with a focus on maintenance and assigned based on quantity approach techniques associated with the condition that is available. One example: to design a long segment of the job picture is done with the incorporation of the roads will be DED merged so the length of the segment, and is done with a focus on maintenance and assigned based on quantity.

Whereas in fiscal year 2017, scheduled so that each province is putting together a one (1) contract of long-term preservation (Long Term Preservation contract), that would later form the multiyear long program over a period of 3 years budget

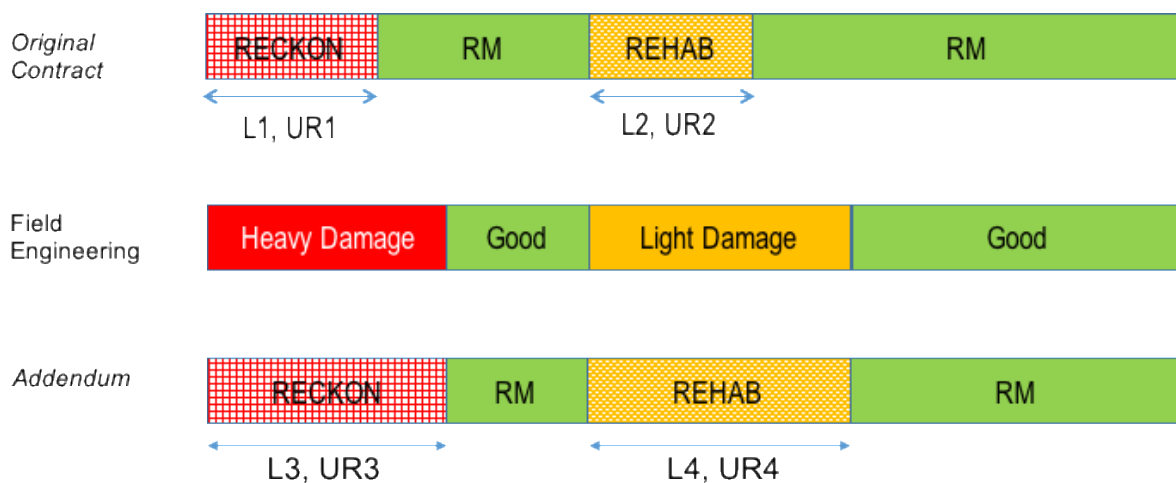
Long Segment definition include the following:

- (1) Road package with the length from 100 to 200 km
- (2) Within multiyear budget
- (3) Combination of three program: Preventive Maintenance, Rehabilitation and Reconstruction

Contractors and consultants involved in long segment, it must be prepared and understand the root of the problems of structural damage in order for selected preservation technology on time. One contract for 100 up to 200 km is not new in the world, there are a lot of country already doing so. This long segment insist the contractors to change from “executor” into “Road Manager”, means that work very carefully

The objective of the implementation of long segment on preservation of the road is to achieved uniform standard with stable condition and fulfil road standard with the following detail:

1. Minimized and eliminate road with a severe damaged condition
2. To Prioritize Road with bad condition until all the long segment addressed by optimizing capacity as long as the budget available
3. The First priority is a very bad road with reconstruction treatment and have the technical life around 5 to 10 years, while took another section (due to budget constraint) in holding treatment.
4. The second priority will be to handled light damage road with rehabilitation
5. Widening will be done simultaneously with pavement (rehabilitation and reconstruction)











- After field engineering revision of Design to extend implementation target (L3 > L1, L4 > L2) with reducing life cycle plan (UR3 < UR1, UR4 < UR2)
- Forexample for reconstruction UR 20 years to 10 years/5 years
- Forexample for Rehabilitation UR from 15/10 years to 10/5 years.

Diagram 3 Program of Handling Long Segment, sources: DG Highways 2015

Some provisions in the implementation of road preservation with the long segment concepts are:

1. The contract System is a contract unit price with most of the work: lump sum at the end of the period (after it refund conditions) for the segment of road maintained
2. No Guarantee for the Routine maintenance Work
3. Reconstruction and rehabilitation work guarantee around 2 years.

No	Scope of Work	Implementation Period	Maintenance Period
1	Widening		
2	Reconstruction		
3	Rehabilitation		
4	Maintenance		

PHO

FHO






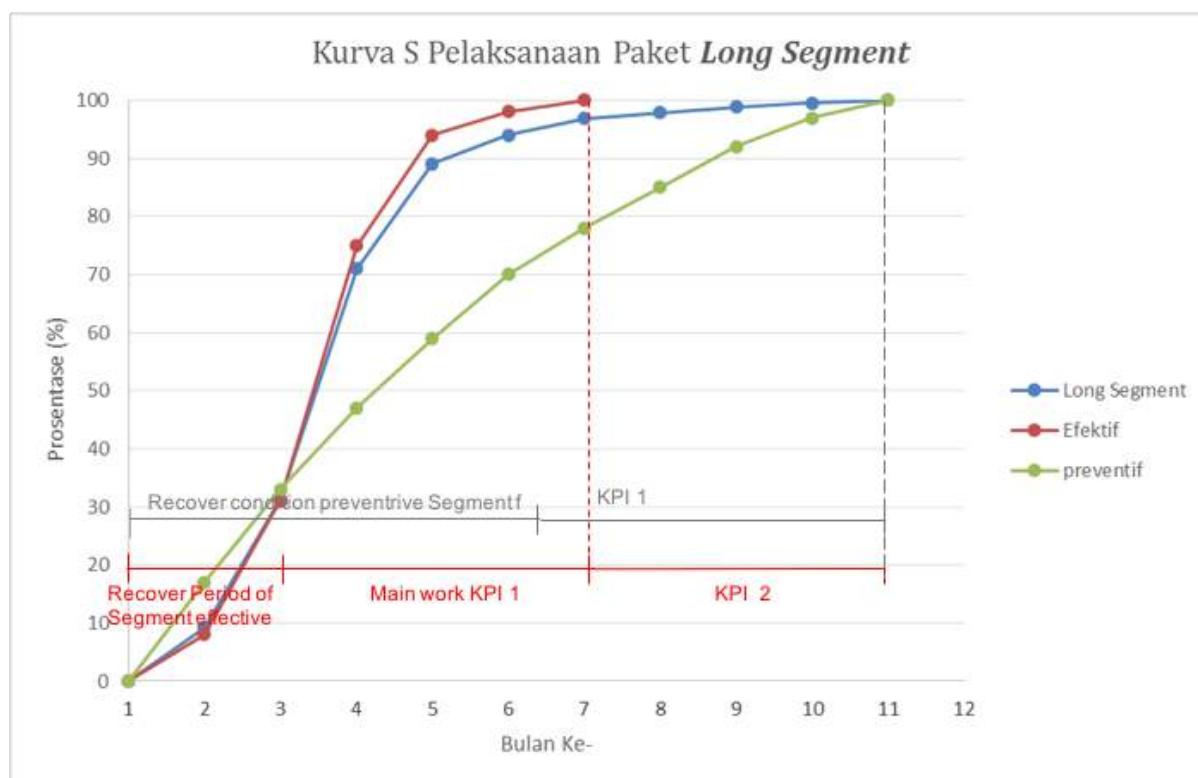
-  : Recover Period (Construction work at the pavement pay based on Volume)
-  : Implementation Period (Construction work at pavement will be paid by volume based)
-  : Period after recovery up to PHO (Routine work will be paid in Lumpsum)
-  : Completion of work period up to PHO
-  : Warranty Periode

Diagram 4 KPI 1 and 2, source: Writer



6. Conclusion

Firstly, starting from 2016, again to remind all the stakeholder of preservation to do a fundamental changes, where for building road, it has to refer to Ministerial PUPR decree No. 19/PRT/M/2011 in connection with Road Standard and should be implemented as a guidelines to build road in Indonesia. The concept also lead to introduce new motto: Apparatus Care Standard (APS) and become part of the Standard Operational Procedures in implementing preservation of National Road. To implement this, by 2016, it will be done pilot project, 1 PPK/1 km.

Secondly, start from 2016, introduced and implemented the Long Segment concept, with around 100 up to 200 km road length per contract package, and this contract will be consists of comprehensive procurement procedures. By introducing this policy, government try to eliminate and minimized the force account and also implementing combination of treatment between routine, periodic, rehabilitation and improvement within a contract. In 2016 still in transition, but in 2017 will be done systematically.

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KEYWORDS:

motorcycle safety, exclusive motorcycle lane, access, gap, gap acceptance

ABSTRACT:

Motorcycle accidents contribute up to 60% of the traffic fatalities in Malaysia. To increase motorcycle safety, exclusive motorcycle lanes (EMCL) have been built with the aim to segregate them from other (larger) vehicles. Egress (path of exiting) and ingress (path of entering) are hazardous locations on EMCL where motorcyclists are required to make critical decision whether to join or leave a traffic stream. The important factor they will need to consider is the availability of a gap between two vehicles that, in the motorcyclist's judgement, is adequate for them to complete the maneuver. Poor gap acceptance decisions will increase likelihood of accident to occur. Findings from the study reveal that gap acceptance for motorcyclist is lower than 1 sec. This paper presents findings in regards to a better understanding of the gap acceptance of motorcyclists and guides the way for safer design solutions for EMCL.

Motorcycles Gap Acceptance At Merging Point Of Egress And Ingress Of Exclusive Motorcycle Lane In Malaysia

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1 INTRODUCTION

Access into and out from the exclusive motorcycle lane (EMCL) is provided through egress and ingress section. Egress is a path of exiting from the motorcycle lane whilst ingress is a path of entering into the motorcycle lane. Similar to intersection, interaction area (merging and diverging) of egress and ingress is expected to be one of the risky locations on both EMCL and adjacent main carriageway. Here, motorcyclists are required to make multiple decisions such as to leave or join the traffic stream, slow down or stop to observe for oncoming traffic and many more. Knowing the fact that motorcyclist will lose stability at speed below 5 km/h, most of them refuse to stop to complete the maneuver.

The most important factor a motorcyclist will need to consider when joining the traffic stream is the availability of a gap between two vehicles. Acceptance gap is the gap available that in the motorcyclist's judgement, is adequate for them to complete the maneuver. Acceptance or rejection of available gap between two vehicles on the intended traffic stream has serious consequences on safety. Poor gap acceptance decisions will increase the likelihood of accident to happen.

The size of gap in traffic stream is one of the main parameter in capacity analysis. Capacity is important to determine the performance of the road network. Generally, capacity at access point predominates the overall capacity of the road network (Amin & Maurya, 2015). Significant volumes of joining or leaving traffic at access point will cause interruptions and capacity reduction. Therefore, incorrect estimation of the capacity at the design stage will lead to inappropriate design decision. If large gap is assumed, the road will be over-designed causing negative value of infrastructure investment. It also will reduce facility efficiency and adversely affecting the road system. On the other hand, if small gap is assumed, it can force motorcyclists to make gap acceptance decisions in a dangerous manner. Hence, the study on gap acceptance decision among motorcyclists will contribute to the new knowledge for EMCL design.

Gap acceptance parameter forms a crucial part in many microscopic traffic simulations. Even though most of the simulation software has provided default value for gap acceptance, it still allows the modeller to adjust to more relevant values to suit onsite motorist behaviour. In addition, most of the default value provided in simulation software is for car and it is expected that the default value will not simulate onsite condition accurately for motorcycles behaviour. Sensitivity testing of the simulation model of six ways roundabout in Birmingham reveals that the observed gap acceptance value showed a better level of fit to onsite condition compared to default values (Kay, Ahuja, & Cheng, 2006). Kay et al added in their findings that gaps accepted by heavy vehicles differ from light vehicles. Therefore, findings from this study can contribute to more motorcycle research and the application of microscopic simulation in the future.

At present, to the best knowledge of the author, no published works examining gap acceptance for motorcycles has been found in the literature. Based on the above point of view, a need exists to adopt greater understanding of motorcyclists' gap acceptance behaviour. The aim of the study is to determine gap acceptance value for motorcyclists at merging point of egress and ingress of EMCL. Findings obtained from this study will help fill in the gap in existing motorcycle research and will improve the safety of Malaysian motorcyclists.

2 SCOPE AND LIMITATION

The study started in July 2014 and ended in March 2016. Data collection was conducted in August and September 2015. The study was conducted at six (6) egress locations and six (6) ingress locations along the main arterial road, Federal Road 2. Three (3) main types of data collected for the study are; gap acceptance time, classified vehicle speed and classified vehicle volume. All data were collected during peak and off-peak period on normal weekday (Tuesday, Wednesday and Thursday) where no abnormal traffic composition was observed.

The limitation to the study is that the data collection was restricted to only merging point of egress and ingress. Other external effects that may occur on the traffic exiting/entering the exclusive motorcycle lane were not included in the study.

3 LITERATURE REVIEW

3.1 Motorcycle Safety

Motorcyclists are categorized as one of the vulnerable road users due to the low ability to sustain severe injuries in the event of a traffic collision. Abdul Manan & Várhelyi, (2012) in their study highlights that Malaysia has the highest road fatality risk (per 100,000 population) among the ASEAN countries and more than 50% of the road accident fatalities involve motorcyclists. Motorcyclists are prone to injuries due to the fact that helmets and type of clothing worn are the only possible protection gear for them.

Number of registered motorcycle in Malaysia shows an increasing pattern since it was first introduced to the local market in mid of 1970's (Road Transport Department Malaysia, 2014). Malaysians choose to ride motorcycles due to several factors including it being the cheapest and fastest mode of traveling that these small vehicles allow for. Less petrol and less space requirement for parking make it more favorable for Malaysians working in the city center like Kuala Lumpur (Industry Research Department, 2015).

The increase of motorcycle volume consequently leads to the increase of road exposure, one of the possible reasons for the high injuries and fatalities on road. This is supported by crash statistic from year 2002 to year 2012, where motorcycle fatalities rose approximately 22% from 3,429 to 4,178 fatalities (Royal Malaysia Police, 2013). Motorcycle accident in Malaysia is an epidemic issue because it cuts out more than half of road accident fatalities.

3.2 Motorcycle Facility

There are two (2) types of motorcycle lane in Malaysia i.e. exclusive motorcycle lane (EMCL) and non-exclusive motorcycle lane (NEMCL). EMCL differ from NEMCL where it was built separately from the traffic carriageway. The NEMCL on the other hand make use of road marking to separate motorcycle from other vehicle types. Both type of motorcycle lane has shown benefit in reducing the number of motorcycle crashes. As reported by Radin Umar et. al., (1995), the first EMCL constructed on Federal Road 2 was proven to reduce approximately 34% of motorcycles fatalities excluding accident occurring on the main carriageway.

Radin Umar & Barton, (1997) proved that the preliminary Benefit to Cost Ratio (BCR) of providing an EMCL ranges from 3.3 to 5.2 depending on assumptions used in calculating the motorcycle accident costs and the capacity of the exclusive motorcycle lane. Their study indicated that even though the cost for provision of exclusive motorcycle lane is high, the benefit is at least three times higher than the construction cost whereby resulting in a cost effective approach to tackle motorcycle safety problems in Malaysia.

Carriageway in Malaysia is based on a mixed traffic system where motorcyclists have to share the road with larger vehicles resulting in a differential cruising speed and mixed flow conflicts. Previous study showed that vehicles travelling with large speed variance were more likely to be involved in an accident (Hauer, 1971). If motorcycles were segregated from other larger vehicle, the mixed traffic flow risk can be managed efficiently. However, to date, the total length of motorcycle lane provided on Malaysian road are still low.

3.3 Gap Acceptance

Highway Capacity Manual (HCM, 2000) defined gap as the space between the vehicles on the major road at an unsignalised intersection while gap acceptance describes the completion of a vehicle’s movement on minor road into a gap. In addition, HCM defined the critical gap as the minimum time interval between the front bumpers of the second of two successive vehicles in the major road that will allow the entry of one minor road vehicle. The capacity of minor road of an unsignalised intersection rely on the possibilities to have enough gap between vehicle on the major road to cross the conflict space safely (Amin & Maurya, 2015). HCM (2000) also explained that gaps sizes required by the minor road depends on the major road traffic stream, available sight distance, the length of time the minor road vehicle has been waiting at intersection, and the driver characteristics (driver’s eyesight, reaction time, age, etc.).

Laberge et. al., (2006) highlights that in observing the gap size, drivers must have a sense of how fast the approaching vehicle is traveling, how far away the vehicle is and how soon it may arrive at the intersection. Drivers may accept gaps that are less safe when they overestimate the arrival time of approaching vehicles. Previous study found out that the gap acceptance has significant correlation with driver’s age. Older drivers seem to accept greater median gap and take longer time to cross the intersection than younger drivers (Alexander, Barham, & Black, 2002). HCM, (2000) outlined critical gaps ranging from 5.0 seconds for major road to 6.5 seconds for minor road and AASHTO, (2001) recommends a minimum threshold of 7.5 seconds for a left-turn maneuver. However, this is based on the passenger car’s behavior and no study has been found on the motorcyclist’s behavior towards gap acceptance.

4 METHODOLOGY

This section discusses the methodology applied in conducting the study. Field data collection was conducted in order to obtain data needed to evaluate and quantify the gap acceptance for motorcyclists.

4.1 Identification of Study Location

Site reconnaissance was conducted on the exclusive motorcycle lanes along the Federal Highway (F2) in the state of Selangor (30km per direction) and the Shah Alam Highway (KESAS) (20km per direction). Drive-through video recording was conducted on the EMCL and on the main carriageway. The video recording was used to assess the road environment and to simulate a motorcyclist’s/motorist’s riding/driving experience. The video was also used to help in identifying suitable locations to conduct the study.

Egress and ingress were grouped into the five (5) types based on the category sets in previous study conducted by Norfaizah, Abdul Manan, & Nusayba, (2015). The five (5) groups of typical types of egress and ingress are listed in Table 1. Two (2) locations for each type of egress and ingress was selected for study locations as summarised in Table 2. The Type 1 (Type E1 and Type I1) and Type 5 (Type E5 and Type I5) egress and ingress was excluded from this study due to insufficient number of location. This left twelve (12) locations i.e. six (6) egress and six (6) ingress selected for this study as shown in Table 2.

Table 1. The five (5) types of egress and ingress identified from previous study

Type	Egress	Ingress
Type E1 or Type I1 Entry angle 90 degree, length of slip lane < 15m		









Type E2 or Type I2 Presence of auxiliary lane for acceleration and deceleration on the motorcycle lane		
Type E3 or Type I3 Entry angle <90 degree, length of slip lane <15m		
Type E4 or Type I4 Entry angle <90 degree, length of slip lane >15m		
Type E5 or Type I5 Skewed access that gives impression of a seamless connection between motorcycle lane and the main carriageway.		

Table 2. Selected study locations

TYPE	LOCATION	ID
Egress Type 1 (E1)	KESAS – Subang Jaya	E1-01
	KESAS – Taman OUG	E1-02
Egress Type 3 (E3)	Padang Jawa	E3-01
	Subang Airport	E3-02
Egress Type 4 (E4)	Amcorp Mall	E4-01
	Ke Bulatan Kayangan	E4-02
Ingress Type 1 (I1)	Susur Batu Tiga	I1-01
	Petronas Shah Alam	I1-02
Ingress Type 3 (I3)	Seksyen 7	I3-01
	Setia Jaya	I3-02
Ingress Type 4 (I4)	Susur NPE	I4-01
	Dari Bulatan Kayangan	I4-02

4.2 Data Collection

Accurate and sufficient data play a critical role in the success of any conducted study. Field data collection was conducted at twelve (12) locations, comprising of six (6) locations at egress and another six (6) locations at ingress. To collect the required data for this study, a video camera was used. The video camera was mounted on a special pole at the height of 3 m and was positioned to capture the motorcyclist exiting or entering the exclusive motorcycle lane as well as the oncoming traffic, simultaneously. Figure 1 depicts the position of mounted video camera.

Video recording was conducted for two (2) hours of morning peak (7.00 am – 9.00 am) and two (2) hours of morning off-peak (10.00 am – 12.00 am). A total of 48 hours of video playback is obtained for data extraction and analysis. Data collection was conducted during weekdays (Tuesday, Wednesday and Thursday). Data collection was not conducted during bad weather or during unusual traffic conditions (e.g.

traffic congestion due to roadway maintenance, crashes incident, police enforcement activity or public holiday).

Three types of main data collected are gap acceptance time together with the details (as listed in Section 4.3), classified vehicle speed, and classified vehicle volume. Gap acceptance time and classified vehicle volume data was retrieved from the video recordings while vehicle speed data was collected manually on site using speed radar gun. The placement of speed observer officer is illustrated in Figure 1 and four (4) types of speed data collected are:

- i. Speed on EMCL at 100 m to 200 m away from the egress/ingress merging point
- ii. Speed on EMCL at the egress/ingress point
- iii. Speed on main carriageway at 100 m to 200 m away from the egress/ingress merging point (speed for different types of vehicles on slow lane only)
- iv. Speed on main carriageway at the egress/ingress point (speed for different types of vehicles on slow lane only)

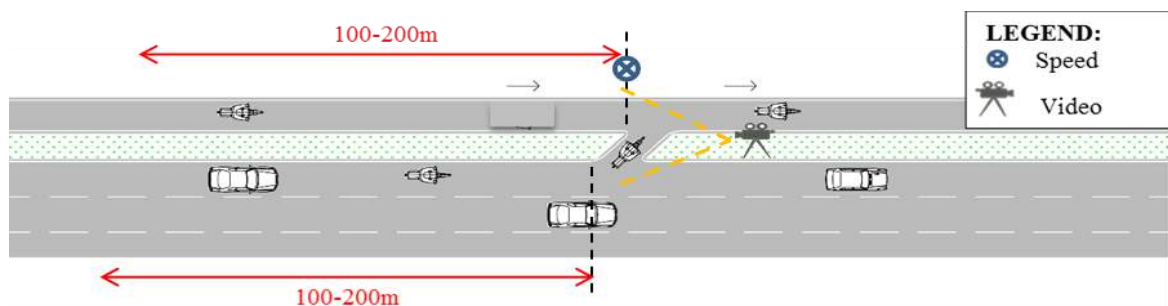


Figure 1. The illustration on the location for data collection

4.3 Data Retrieval

The video recording was analysed in the office. The types of data were retrieved from the video are:

- i. Motorcycles arrival and departure time at egress and ingress
- ii. The time when conflicting vehicle crosses the point where it conflicts with the entering/exiting motorcycles.
- iii. Accepted/rejected gap
- iv. Motorcycles entering or exiting exclusive motorcycle lane from stand still position or floating.

A customised video play program used to retrieve data from the video was created using Language C#. This is to increase the accuracy of the data. With the program, research assistant was asked to press specific keys on computer when vehicles reached designated locations. The program generated a unique timestamp marking the position of the vehicles.

4.4 Data Analysis

The analysis of this study was done in two parts, i.e. analysis of gap acceptance decision and speed study. It is expected to disseminate the findings in terms of revealing the motorcycle gap acceptance, speed distribution, cross tabulation of gap acceptance based on egress and ingress type and travel period (peak or off peak). Microsoft Excel, SPSS, XLSTAT and Wizard software was used to help in the data analysis.

5 RESULT AND DISCUSSION

5.1 Gap Acceptance by Location

Table 3. Gap acceptance and rejection characteristic by location

ID	ACCEPTANCE			REJECTION		
	SAMPLE SIZE	MIN TIME	MAX TIME	SAMPLE SIZE	MAX TIME	MAX TIME
E1-01	378 (98.95%)	0.000	57.310	4 (1.05%)	0.662	30.187
E1-02	120 (96.77%)	0.000	40.538	4 (3.23%)	0.786	2.836
E3-01	515 (95.72%)	0.000	43.854	23 (4.28%)	0.000	14.323
E3-02	617 (99.68%)	0.000	32.687	2 (0.32%)	0.270	3.463
E4-01	865 (100%)	0.000	23.142	-	-	-
E4-02	567 (93.87%)	0.000	33.613	37 (6.13%)	0.207	22.462
I1-01	22 (84.62%)	0.210	12.455	4 (15.38%)	0.361	1.559
I1-02	212 (97.25%)	0.078	45.402	6 (2.75%)	0.283	4.608
I3-01	968 (88.97%)	0.110	46.661	120 (11.03%)	0.158	3.232
I3-02	1045 (95.78%)	0.060	36.359	46 (4.22%)	0.154	36.430
I4-01	2065 (99.85%)	0.112	40.160	3 (0.15%)	0.548	1.508
I4-02	148 (99.33%)	0.014	33.203	1 (0.67%)	0.266	0.266

Table 3 lists the sample size, minimum and maximum time of gap acceptance and rejection collected at 12 locations of study area. The percentage of rejection gaps amongst motorcyclists is apparently low. The high percentage of gap rejection was observed at ingress type of I1-01 (15.38%) and I3-01 (11.03%). Based on the site observation, ingress I1-01 and I3-01 located just after a curve section where the observation on the oncoming motorcyclist is limited to the entering motorcyclist at ingress. This was seen to trigger motorcyclists to stop and reject small gaps. In addition, there is no shoulder provided on the EMCL to allow for comfortable lateral clearance.

It was perceived that motorcyclist accept minimum gaps time of zero (0) seconds at egress locations most probably due to availability of sufficient lateral clearance between edge line and vehicle on the intended lane (main carriageway). The narrow paved shoulder provided on the main carriageway also offers additional space for motorcyclists to merge into the main traffic.

Cumulative frequency distribution of gap acceptance and rejection type was illustrated in Figure 2. It was found that most motorcyclists tend to reject gap smaller than 1.75 seconds (45%). However, it was interesting to note that the percentage of motorcyclist accepting gap below than 0.25 seconds was 7.06% (572 samples).

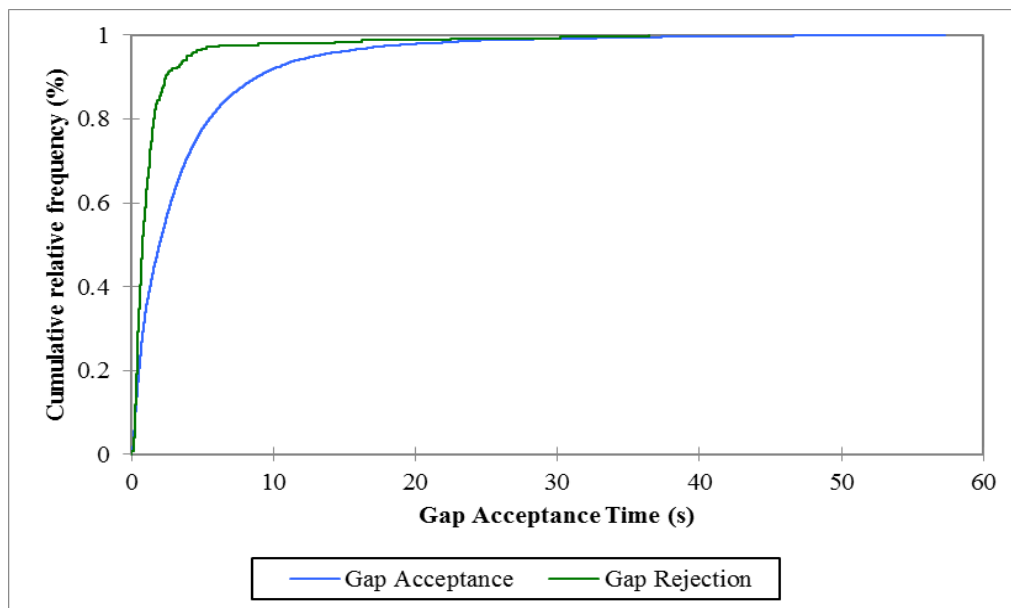


Figure 2. Gap acceptance and rejection time cumulative relative frequency distribution

The gap acceptance cumulative relative frequency distribution was plotted by egress and ingress type as shown in Figure 3 (egress) and Figure 4 (ingress). Generally, the graph indicates that gap acceptance at each location was unique and not correlated with the egress and ingress design. As explained in section 4.1, for each egress and ingress design, two locations were selected for data collection. The similar gap acceptance trend was only observed for egress type E3 and ingress type I3.

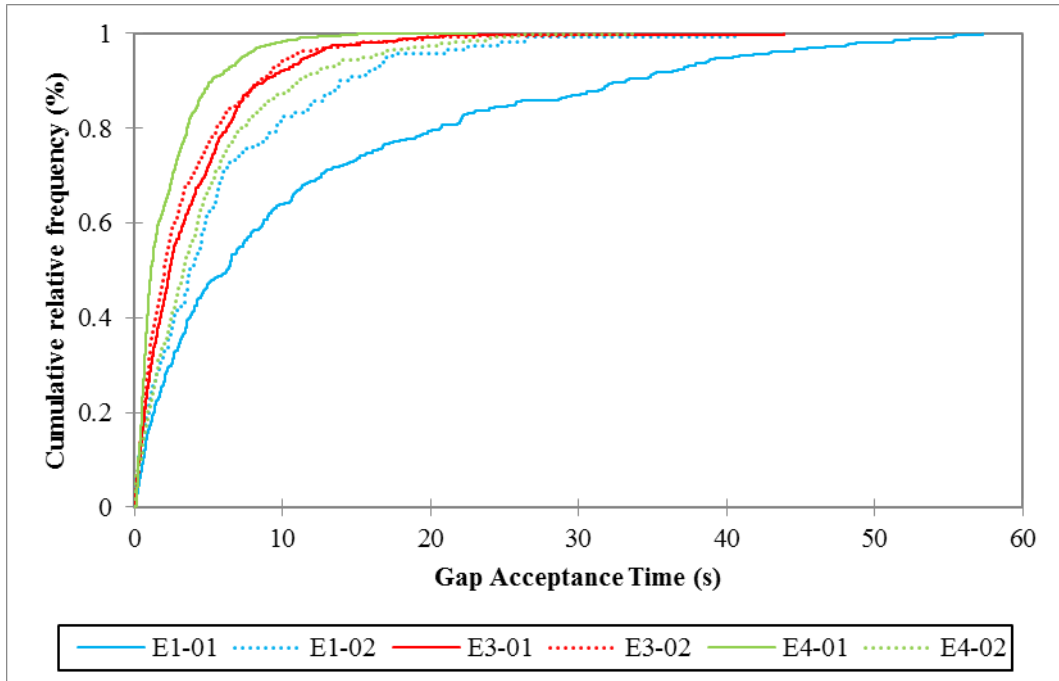


Figure 3. Gap acceptance cumulative relative frequency distribution by egress location

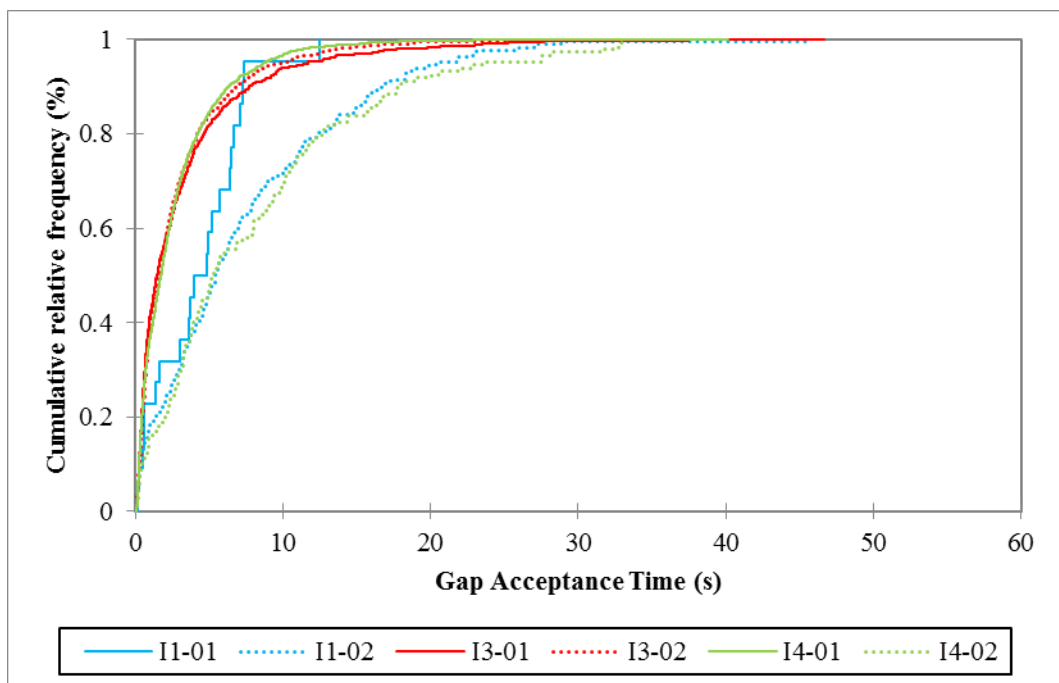


Figure 4. Gap acceptance cumulative relative frequency distribution by ingress location

5.2 Gap acceptance by time period

The gap acceptance was also tabulated to see the effect of time period on gap acceptance behavior as illustrated in Figure 5. It was found that motorcyclist accept smaller gap time during peak period as compared to off peak period. This was parallel with findings by Kyte, (1994), where sites with higher traffic volumes are likely to put pressure on drivers to accept shorter gaps.

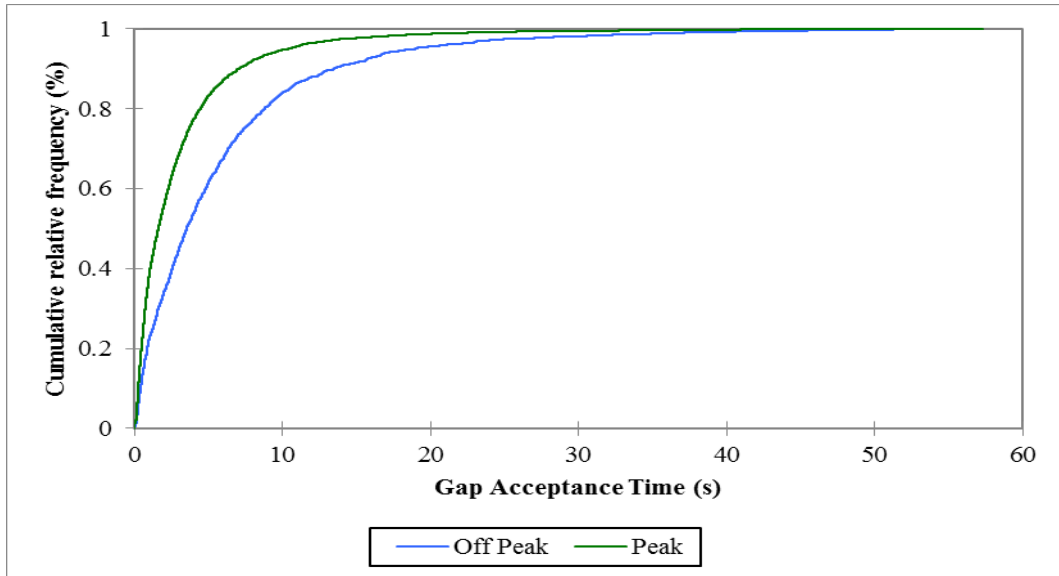


Figure 5. Gap acceptance time cumulative relative frequency distribution by time period

5.3 Speed study

In addition to the gap acceptance analysis, this study also included the effect of speed on the existence of egress and ingress section on EMCL and the main carriageway (MC). The analysis was translated into graph as shown in Figure 6 (EMCL) and Figure 7 (MC) showing mean approaching speed (speed at 100-200 m from the egress or ingress) and mean merging speed (speed at merging area).

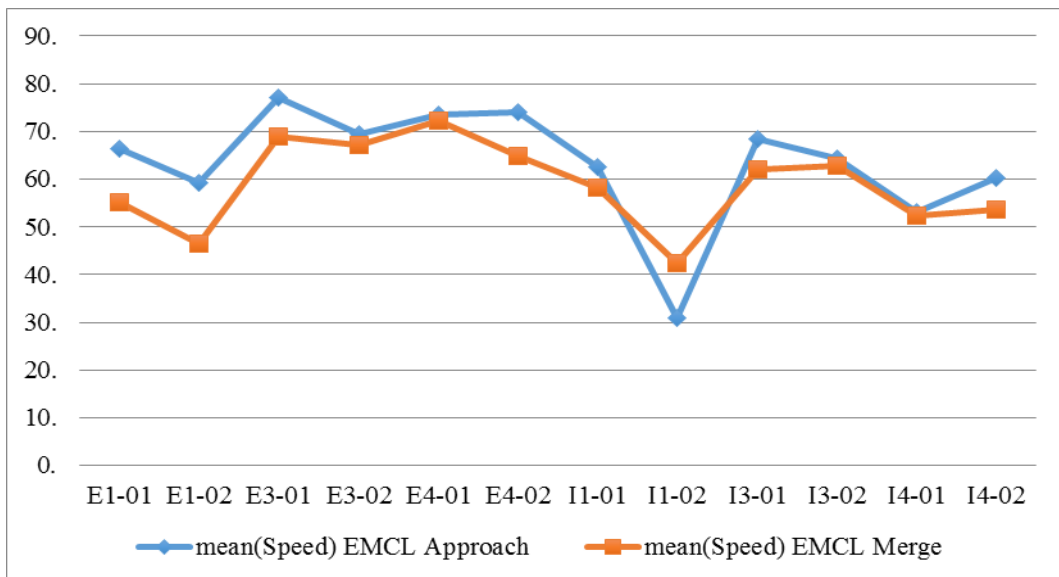


Figure 6. Mean speed on EMCL at approaching of egress and ingress and at the merge area

Based on the data analysis on EMCL, approaching speed is higher compared to at merging area at all study location except at location I1-02. Based on observation, ingress location of I1-02 is near the curve area and thus a lower mean speed was observed mainly because motorcyclist has reduced the speed to maneuver the curve safely. At location E3-02, E4-01, I3-01 and I4-01, no significant speed reduction was observed at merging area.

Whilst on the main carriageway, it was observed that vehicles do not reduce the speed at merging area for location I3-01 and I3-02 and no significant reduction was observed at location I1-01 and I1-02. The speed at merging area of MC at location E1-01 was unable to be collected due the presence of enforcement activity during data collection. Speed of vehicles on MC were mostly not affected or interrupted at ingress section, compared to egress section.

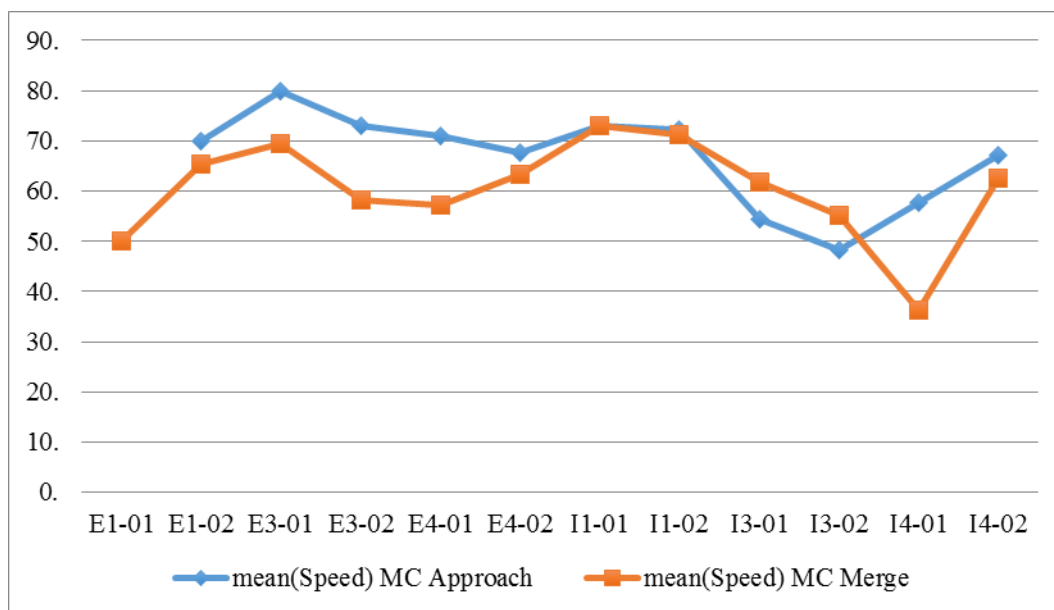


Figure 7. Mean speed on MC at approaching of egress and ingress and at the merge area

6 CONCLUSIONS

The research in gap acceptance decision among motorcyclists at merging point of egress and ingress of EMCL is long overdue. It is significant to improve the infrastructure for motorcycles and increase the level of safety. By knowing the gap acceptance, not only can the design parameter for EMCL be known, but it also opens up opportunities to further explore research on motorcycle infrastructure and behavior. This in long term can help to reduce the number of motorcycle fatalities due to road accidents.

These conclusions can be drawn from this study:

- i. Motorcyclists accept minimum gaps time of zero (0) second at egress location due to the availability of sufficient lateral clearance between edge line and vehicle on the intended lane;
- ii. The percentage of rejection gap time for motorcyclists was low (3.46%);
- iii. Most motorcyclists tend to reject gap time smaller than 1.75 seconds (45%);
- iv. Gap acceptance was unique at each locations and not correlated with the design type;
- v. Motorcyclist accept smaller gap time during peak period as compared to off peak period; and
- vi. Significant of speed reduction at merging area of egress and ingress can be observed on both EMCL and MC at most locations.

For safety reason, it is suggested that the design of egress and ingress should consider the provision of taper lane or auxiliary lane so that entering or exiting motorcycles have ample time to adjust their speed with the

speed of intended traffic. This is due to the reason that motorcycle have high probability to accept small gaps up to zero (0) seconds which may increase the likelihood of accident.

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PAPER TITLE	Cracking performance evaluation of asphalt mixtures using semicircular bending test: Experimental evaluation and Box-Behnken optimization		
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KEYWORDS:

Cracking, asphalt mixture, fracture resistance, semicircular bending test, performance optimization

ABSTRACT:

Owing to the increasing popularity of asphalt rubber gap-graded (AR-Gap) mixtures, a long-term cracking performance of AR-Gap demands a fundamental fracture characteristics evaluation. Since the working principle of the gap-graded mixture is distinct from the conventional dense-graded asphalt mixture, it is important to understand the fracture mechanism of the AR-Gap mixtures. In addition, a better performance of AR-Gap mixture in terms of cracking necessitates an optimized design with a combination of mixture-variants; and understanding the effects of potential contributing factors to the fracture characteristics. Thus, the objective of this study was to understand the cracking performance mechanism of the different AR-Gap mixtures using mechanical fracture tests, that included determination of fracture resistance (K_{IC}) using static semicircular bending (SCB) test at varying mixture variants and temperature combinations. A total of eight AR-Gap mixtures were prepared to comprehensively assess the effect of three factors: temperature, asphalt content, and air voids. Further, the experimental data was statistically analyzed to understand the effect of predictor variables on K_{IC} using analysis of variance (ANOVA) test. It was found that temperature and asphalt content significantly influenced the fracture resistance of the AR-Gap mixtures. An optimization study was carried out using Box-Behnken technique to assess the level of the three factors; to capture which of them was a better cracking performance indicator. The optimization solution indicated that low level of temperature and air voids, in association with intermediate level of asphalt content resulted in optimized cracking performance evaluation. Furthermore, the fracture mechanism of AR-Gap mixture was analyzed using fundamental load-time relationship and compared to the conventional dense-graded asphalt mixture. It was noticed that occurrence of cracking in the AR-Gap mixture took approximately twice that of the time taken for the dense-graded one. Overall, this study helped understand the fracture characteristics of AR-Gap based on fracture mechanics principles, and developed a rational methodology to evaluate the cracking performance both reliably and rationally.

Cracking performance evaluation of asphalt mixtures using semicircular bending test: Experimental evaluation and Box-Behnken optimization

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1 INTRODUCTION

Cracking is one of most fundamental failure modes in the flexible asphalt pavements. Although the cracking constitutes a wide variation of distresses in the flexible pavement, a categorical classification of cracking includes a load-associated and thermal-derived failure in the mainstream cracking performance analysis. In general, the load-associated fracture behavior is closely pertinent to the fatigue performance; while the thermal-derived fracture is associated with the study of the low-temperature thermal performance of the asphalt mixture. Thus, a long-term cracking performance of the asphalt mixtures demands a fundamental evaluation of fracture behavior using fracture mechanics principles.

In the last few decades, the use of asphalt rubber gap-graded (AR-Gap) mixtures has been very popular due to its better performance over conventional dense-graded (DG) asphalt mixtures. A wealth of literature can be found on the cracking performance evaluation of AR-Gap mixture on the premise of conventional mechanistic approach, where the analysis pivots the stiffness and strain parameters to sketch the cracking performance. It is noteworthy that the stone-to-stone contact between the aggregates is the key factor that demarcates the working principle of DG asphalt and AR-Gap mixtures. With this background, it can be understood the AR-Gap is designed with higher air voids and asphalt content in comparison with the DG asphalt mixes. Thus, the cracking mechanism of AR-Gap employs a distinct evaluation approach to understand the mechanism of the cracking performance. Furthermore, it is also important to understand the level of asphalt content and air voids to obtain the optimized cracking performance of the AR-Gap mixture.

Although there are several fracture parameters available, fracture toughness (K_{IC}) provides simple but a comprehensive understanding of the fracture behavior of the asphalt mixture. In recent times, Semi-Circular Bending (SCB) test has been utilized by many searchers (Molenaar et al. 2003, Birgisson et al. 2008, Li and Marasteanu 2010, Minhajuddin et al. 2015, Saha and Biligiri 2015, Saha and Biligiri 2016) successfully to evaluate fracture characteristics of asphalt mixtures. This objective of this study was to evaluate the cracking performance using a statistical optimization: Box-Behnken Design (BBD) and investigate the fracture mechanism of different AR-Gap mixtures on the basis of K_{IC} using the SCB test. The scope of the study includes (Figure 1):

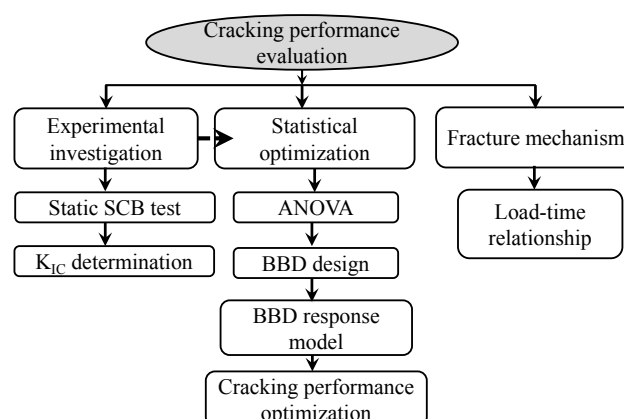


Figure 1. Research framework

- Determination of fracture toughness of various AR-Gap mixtures at varying temperature using static SCB test method
- Formulation of BBD orthogonal matrix and perform an optimization analysis
- Comparative analysis of cracking performance of various AR-Gap mixtures
- Recommend an optimized level of variables to obtain the best cracking performance of AR-Gap mixtures
- Investigation of fracture mechanism of AR-Gap in contrast with DG asphalt mixes

2 BACKGROUND TO THE BBD OPTIMIZATION

BBD is a second-order response surface approach to optimize the response based on incomplete factorial design. The formulation of BBD employs consideration of treatment level combination within the central-zone instead of extreme ranges. Since the operational principle of BBD is distinctive from the full factorial design, and response surface approach; it is more efficient due to the less number of experimental runs as well as the elimination of extreme levels of treatment combinations. A detail on the BBD can be found in the literature (Ferreira et al. 2007). In this study, a three-factor BBD was formulated, which encompassed 12 base-experimental runs with two replicates totaling 24 experimental runs. The orthogonal matrix of the BBD design is presented in Table 1. Note that -1, 0, and 1 indicate the low, medium, and high level of the factors, respectively.

Table 1. Orthogonal matrix of BBD for three factors

Experiment runs	Factors		
	X	Y	Z
1	-1	-1	0
2	1	-1	0
3	-1	1	0
4	1	1	0
5	-1	0	-1
6	1	0	-1
7	-1	0	1
8	1	0	1
9	0	-1	-1
10	0	1	-1
11	0	-1	1
12	0	1	1

3 MATERIALS AND EXPERIMENT PROGRAM

A total of eight AR-Gap mixtures were prepared using crumb rubber modified binder: CRMB-60 and Arizona gap-graded aggregate prescribed in (Way et al. 2012). Superpave gyratory specimens, (height 120 mm and diameter 150 mm) were prepared using varying asphalt contents (AC) and gyration levels to achieve the different air voids (*AV*). The gyratory specimens were sliced into two discs of height 50±2 mm and then, each disc was cut centrally to obtain two semi-circular discs. Subsequently, a central notch of 15 mm was introduced at the mid-point of the base along the thickness direction. A total of 24 semi-circular specimens were prepared with two replicates to suit with the orthogonal matrix shown in Table 1. As a next step, an SCB test was conducted on 24 specimens at three temperatures, namely 5, 15, and 25 °C using Universal Testing Machine (UTM) in accordance with AASHTO TP 105, and K_{IC} was calculated using Equation (1) as summarized in Table 2. The test specimens were conditioned for four hours prior to the testing. A notched-SCB sample and the experimental setup along with the specimen are shown in Figure 2.

$$K_{IC} = \sigma_o \sqrt{\pi a} \times Y_I \quad (1)$$

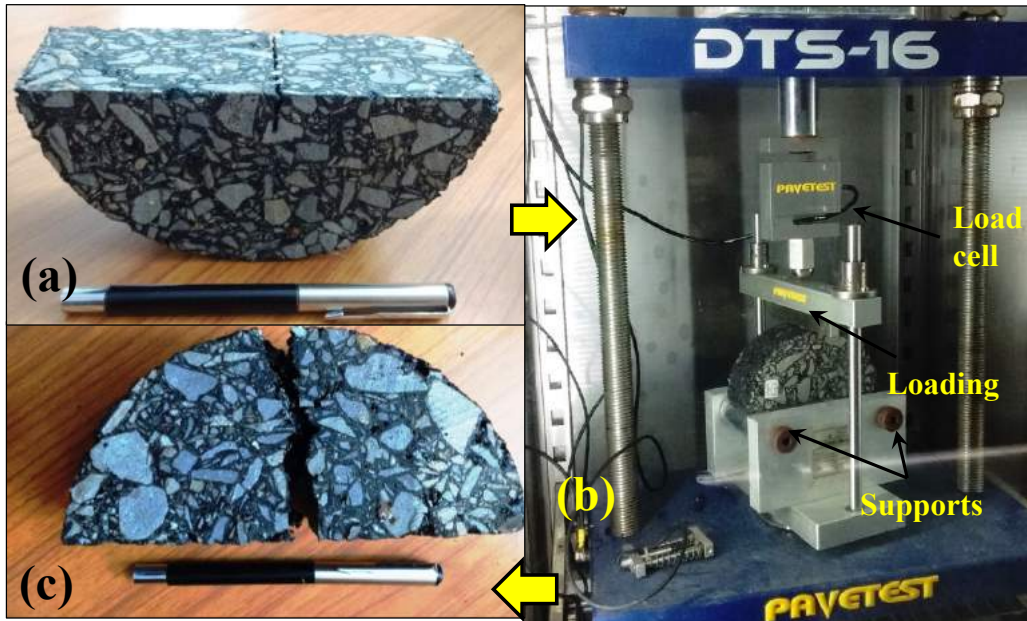


Figure 1. (a) Notched- SCB specimen, (b) experimental set-up of SCB test, and (c) cracked specimen after the SCB test

Table 2. Experimental matrix

T (°C)	AC (%)	AV (%)	K_{IC} (MPa. \sqrt{m})	
			Rep_1	Rep_2
5	6	8	0.601	0.643
25	6	8	0.143	0.133
5	8	8	0.529	0.567
25	8	8	0.103	0.108
5	7	7	0.626	0.644
25	7	7	0.099	0.125
5	7	9	0.509	0.617
25	7	9	0.139	0.135
15	6	7	0.303	0.390
15	8	7	0.268	0.321
15	6	9	0.340	0.287
15	8	9	0.329	0.322

Where:

K_{IC} = fracture toughness, MPa. \sqrt{m}

σ_o = $P/(D*L)$

P = failure load (MN)

D = Specimen diameter (m)

L = Specimen thickness (m)

a = Notch length (m)

Y_I = geometric factor = $4.782 + 1.219 \left(\frac{a}{r}\right) + 0.063 e^{\left\{7.045\left(\frac{a}{r}\right)\right\}}$

4 RESULTS AND ANALYSES

The BBD experimental matrix and statistical analysis were conducted using MINITAB® 17.0. As mentioned earlier, three factors: temperature (T), asphalt content (AC), and air voids (AV) were varied at three levels with two

replicates per combination. The BBD was considered to investigate the difference in the fracture properties of various AR-mixtures at 95% confidence interval. The statistical analysis of BBD is explained next.

4.1 ANOVA RESULTS

As the first step of BBD, ANOVA test was performed to understand the primary effect of the factors on the fracture properties of AR-Gap mixtures. The results obtained from the ANOVA are presented in Table 3. The null and alternative hypotheses considered in this process were as follows:

$$H_0 : \mu_T = \mu_{AC} = \mu_{AV}$$

$$H_1 : \text{At least one of the mean not equal}$$

If the P -value of the factor shows a value less than 0.05, the null hypothesis can be rejected. As observed in Table 3, T and AC produced P -value less than 0.05, which indicated that the null hypothesis could be rejected. Hence, it can be concluded that T and AV had a significant effect on the K_{IC} of the AR-Gap mixtures. In contrast, AV did not show any significant impact on fracture performance in the range under consideration. Furthermore, the interaction effects were also checked to assess the behavior of the response variables with respect to the interactive factors. It was found that the interaction and quadratic effects did not produce any significant impact on the K_{IC} of the AR-Gap mixtures. However, it can be concluded that AC and T showed a significant effect on the K_{IC} in the range under consideration.

Table 3. ANOVA results

Factors	DF	Adj SS	Adj SS	T -stat	P -value	Model decision
T	1	0.8799	0.8799	809.48	< 0.05	Significant
AC	1	0.0053	0.0053	4.89	< 0.05	Significant
AV	1	0.0006	0.0006	0.57	0.461	Insignificant
T^2	1	0.0044	0.0044	4.05	0.062	Insignificant
AC^2	1	0.0003	0.0003	0.27	0.611	Insignificant
$T*AC$	1	0.0009	0.0009	0.79	0.389	Insignificant
$T*AV$	1	0.0048	0.0048	4.37	0.054	Insignificant
Total	23	0.91751				

4.2 EFFECT ANALYSES

As the next step, it was deemed important to analyze and understand the results obtained from the ANOVA test. Since there was no significant interaction effect, the only main effect would provide a complete understanding towards the K_{IC} of AR-Gap mixtures. Essentially, the main effect explains the effect of an independent predictor variable averaging across all its levels on the dependent variables. Figure 2 shows the main effect of T , AC , and AV on the K_{IC} .

As observed in Figure 2, K_{IC} decreased with increasing temperature. This phenomenon can be explained by the viscoelastic properties of the AR-Gap mixtures. An increase in the temperature reduced the stiffness of the mixtures, which in turn reducing the cracking resistance. As a result, the mixture at higher temperature failed under a lower magnitude of load due to the reduced viscosity of the mixtures. It is also important to observe the linear reduction of K_{IC} with respect to the temperature. It can be expected that the BBD model (discussed in Section 4.3) established in this study would predict the K_{IC} with respect to temperature without any quadratic and interaction with any other variables.

Concurrently, K_{IC} was initially found to be not varying with increasing asphalt content (AC : 6-7%) and then, it reduced with further increment in the asphalt content (AC : 7-8%). The initial stable plateau of K_{IC} indicated the K_{IC} in the range of 6-7% of AC was not sensitive to the change in the AC . It can be understood that an increase in the AC increased the adhesion between the aggregates in the mixture-matrix. But after reaching a certain level (here: at 7% of AC), an increase in AC did not help improve the adhesion, rather it provided a lubrication effect along the aggregate boundaries. Hence, K_{IC} dropped with further increment in AC . On the other hand, AV produced an insensitive change in AC . Similar findings were also noticed from the ANOVA results discussed in Section 4.1. Although it is an established point that air voids affect the mixture performance significantly, the impact of AV was not clear in the analysis. It may be because of the fact that the potential effect of AV on the cracking performance of AR-Gap mixtures based on the K_{IC} was lower than the other variables and their associated levels and thus, the cracking performance was governed by the other factors. However, a reduction in K_{IC} was noticed with increasing AV . Since an increase in AV creates bigger air

void pockets in the mixture-matrix, the stress concentration at the tip increases. As a consequence, fracture resistance decreased with higher AV in the AR-Gap mixtures.

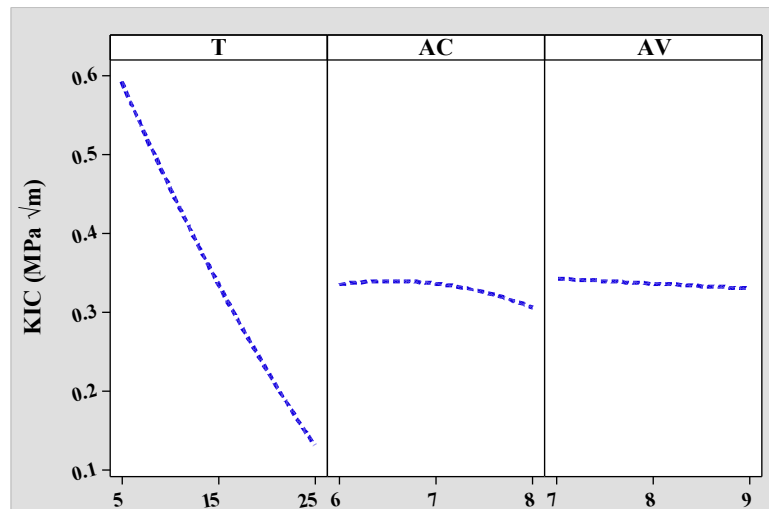


Figure 2. Main effect on K_{IC}

4.3 BBD RESPONSE MODEL

Development of a BBD response model is the crucial step in the process of statistical optimization. The model was constructed using three variables with three factors based on the general regression concept. The model presents the K_{IC} as the output variable as a function of T , AC , and AV including the main and interaction effects. It is noteworthy that though the interaction and quadratic effects were not found to be significantly contributing to the fracture resistance of the AR-Gap mixtures, the model included these effects to achieve better accuracy. The model accuracy was measured by the goodness of fit, $R^2 = 0.9545$ indicated that a good agreement between the measured and the predicted values.

$$K_{IC} = 1.80 - 0.0602 T - 0.041 AC - 0.1541 AV + 0.000332 T^2 - 0.0086 AC^2 + 0.00103 T*AC + 0.00244 T*AV + 0.0159 AC*AV \quad (2)$$

Where:

- K_{IC} = Fracture toughness, MPa. \sqrt{m}
- T = Temperature, $^{\circ}C$
- AC = Asphalt content, %
- AV = Air voids, %

4.4 K_{IC} optimization

As a final step to optimize the response of K_{IC} , BBD optimization was carried out on the basis of the BBD model. It is important to understand the relationship of the fracture performance of the asphalt mixtures with the parameter used for the analysis prior to the optimization process. Since K_{IC} indicates the fracture toughness of the mixtures, it is expected that higher fracture toughness augments the fracture resistance of the AR-Gap mixtures. With this background, an optimization goal was set to maximize the K_{IC} in order to optimize the cracking resistance of the mixtures. In this process, all the variables were kept to an initial setting to find the optimized solution to achieve the optimization goal. The optimization summary is shown in Figure 3.

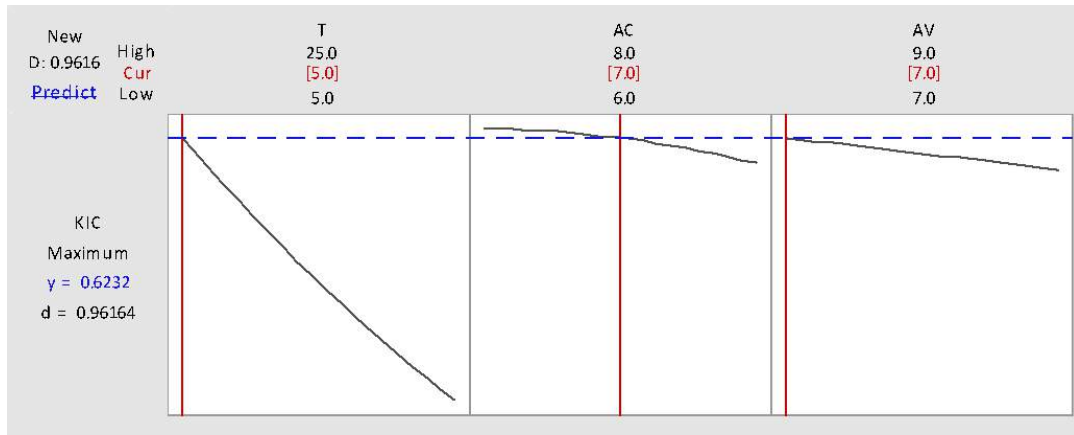


Figure 3. Optimization plot of K_{IC}

It can be observed that K_{IC} was found to have maximized at a lower level of temperature (T), and air voids (AV) and intermediate level of asphalt content (AC). The lower temperature increased the fracture resistance as explained in Section 4.2. Also, 7% of AC produced the maximum response that supported the findings obtained from ANOVA results. The optimization process adopted in the study can be utilized to optimize the fracture resistance of various types of asphalt mixtures including different asphalt binders and aggregate gradations.

5 FRACTURE MECHANISMS

As mentioned earlier, one of the tasks of the study was to investigate the fracture mechanism of the AR-Gap mixtures. To understand the difference in the fracture mechanism, the load-time history was recorded from the data acquisition system of UTM and then, plotted with a comparison with a DG mixture as shown in Figure 4. A conventional DG mixture was prepared with a view to compare and contrast the cracking mechanism of the two different mixture types. One DG gyratory specimen was prepared, and SCB sample geometry was prepared using the aforementioned procedure discussed in Section 3. Note that AR-Gap and DG mixtures, which were used for comparison purpose, were prepared with same air voids, 7%, and with AC of 5 and 7%.

As observed, the DG mixture showed a rapid increment in load within a short span of time, whereas the AR-Gap mixture slowly increased the load magnitude. It was also interesting to note that AR-Gap mixture resulted in failure load, under which the sample sustained some time. On the other hand, the DG mixture showed a sudden drop in the load after reaching the peak load. It revealed important information regarding the distinct fracture pattern of the two mixtures. A hold in peak load magnitude explains that although AR-Gap mixture failure took place at the peak load, it still offers a resistance against the crack propagation. The AR-Gap mixture continued to take the additional load, which worked against the residual crack resistance. It is possibly due to the fact that the presence of the crumb rubber modifiers in the AR-gap mixture endured the viscoelastic characteristics, which in turn rendered a viscoelastic bonding between the asphalt binder and the aggregates. Since the DG mixture was prepared using a virgin binder (viscosity graded VG-30); there was no endured elasticity observed during the specimen fracture. Furthermore, the failure of the specimen after the peak was also found to be distinctive where AR-Gap mixture took a longer time to completely fail as compared to the DG mixture. It provides additional information regarding the complete cracking of the two types of asphalt mixtures. DG mix completely cracked with a rapid reduction in load within a very short span of time whereas the AR-gap mixture showed a prolonged cracking performance with a slow reduction rate of load. Overall, it can be concluded that though the AR-gap mixture would initiate the crack at a comparatively lower load than the DG-mix, AR-Gap mixture would provide a better cracking performance than the DG asphalt mix due to endured viscoelastic effect of the AR-Gap mixture.

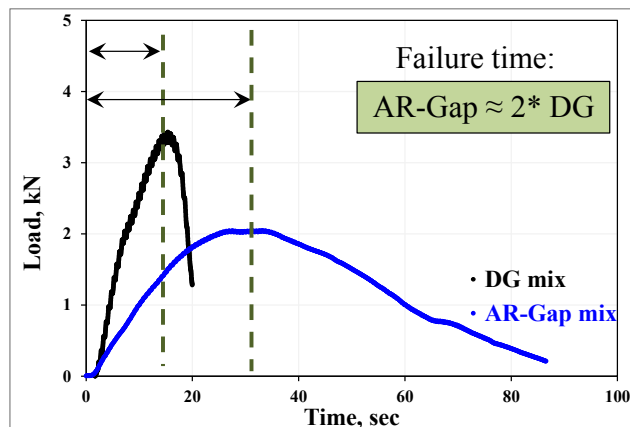


Figure 4. Load-time relationship and fracture mechanism

6 CONCLUSIONS

The objective of this study was to optimize the cracking performance and understand the fracture mechanism of the AR-Gap mixtures. A total of eight AR-Gap mixtures were used to investigate in respect of their cracking characteristics with three factors: temperature, asphalt content, and air voids at three levels. A significant contribution of the study was to formulate a statistical algorithm: Box-Behnken design to assess the effects of variables on cracking performance. The major findings of the study are as follows:

- *Fracture toughness determination*: the study determined the fracture toughness of eight AR-gap mixtures at three temperatures using SCB test method. Then, the test results were statistically analyzed using ANOVA. The statistical findings showed that temperature and asphalt content had a significant impact on cracking performance. Overall, it was found that the SCB test procedure was found to be very promising candidate test to distinguish fracture behavior of AR-Gap mixtures even for a minor change in the mix-composition, such as: asphalt content.
- *Box-Behnken optimization*: A response model was developed as a part of the study to perform the statistical optimization. The accuracy of the model was excellent ($R^2 > 0.95$) indicating an excellent prediction of the response variable: K_{IC} . Further, the optimization results concluded that higher fracture resistance could be achieved with lower temperature, intermediate asphalt content, and low air void levels.
- *Fracture mechanism*: the study discussed the fracture mechanism of AR-Gap and demarcated the mechanical differences in cracking process of the DG asphalt mix. It was found that the occurrence of failure load for AR-Gap took approximately twice the time required for the DG-Gap mix. A sustained peak-load hold was also reported for AR-Gap mixes, which revealed a residual cracking resistance after the crack initiation has taken place.

Although the study considered fracture toughness to evaluate the cracking performance of AR-Gap mixtures, a similar methodology can be utilized using other fracture parameters as assessors. However, more research is definitely needed on this subject to extend the understanding for the other types of materials, including various asphalt binders and aggregate gradations.

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PAPER TITLE	STATISTICAL EVALUATION OF CRUMB RUBBER MODIFICATION PROCEDURE IN ASPHALT BINDER MODIFICATION		
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KEYWORDS:

Asphalt binder, Crumb rubber, Binder modification, Statistical analysis, Factorial design

ABSTRACT:

The objective of the study was to investigate the effect of various mixing/blending factors in the modification procedure of the asphalt binders. The scope of the study included crumb rubber modification of three base binders at various levels of crumb rubber dosages and digestion times. The evaluation of various factors was conducted on the basis of advanced rheological characterization using the Dynamic Shear Rheometer (DSR). The statistical evaluation was performed at various levels of factors using 3³ factorial design on two asphalt binder performance assessors: Strategic Highway Research Program (SHRP) rutting parameter, $G^*/\sin\delta$; and energy dissipation parameter, $\tan\delta$. Further, the main and interaction effects of the factors were also examined to comprehensively understand the improvement in performance of asphalt binder towards high pavement temperature rutting distress. The ANOVA and factorial design results indicated that all the three factors including, base binder, digestion time, and CR dosage significantly influence the performance of the crumb rubber modified (CRM) binder. It was also found that the interaction effect of all three factors was significant in developing a better performing CRM binder with enhanced rutting resistance at higher temperatures. Higher levels of base binder viscosity, digestion time, and CR dosage produced the CRM binders with higher $G^*/\sin\delta$ and lower $\tan\delta$ indicative of improved rutting resistance of the material. It is envisioned that this study will provide a necessary understanding towards binder modification procedure for achieving a superior performing asphalt binder.

STATISTICAL EVALUATION OF CRUMB RUBBER MODIFICATION PROCEDURE IN ASPHALT BINDER MODIFICATION

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1 INTRODUCTION

The innovations in sustainable development have encouraged researchers to consider several recycling techniques to develop better performing pavement materials and design practices. In this context, the use of crumb rubber (CR) from truck/car scrap tires has received positive response across the world due to its superior pavement performance characteristics. The addition of CR improves the binder resistance at various climatic conditions and high traffic. Thus, the advantages of crumb rubber modified (CRM) binders are multifold, which includes usage of waste material leading to a sustainable environment and better long-term pavement performance. In the last few decades, research studies have shown that the addition of CR derived from scrap tires blended into a virgin binder resulted in improved resistance to rutting, fatigue cracking, and low-temperature thermal cracking (Neto *et al.* 2003, Way 2003), and thereby reducing the thickness of the asphalt overlays and reflective cracking potential (Amirkhanian 2003).

The performance of CRM binders highly depends on the production parameters and raw material properties. The production parameters include mixing speed, temperature, and time (also known as digestion time); also, raw material properties including base binder characteristics as well as CR properties and dosages. Researchers have studied the interaction of CR with bitumen (Abdelrahman and Carpenter 1999, Airey *et al.* 2004, Jeong *et al.* 2010); the effect and significance of digestion time while blending (Neto *et al.* 2006, Presti and Airey 2013); influence of CR properties (Peralta *et al.* 2010, Celauro *et al.* 2012, Kovařáková *et al.* 2013, Guillaumot *et al.* 2013); and effect of mixing temperature (Subhy *et al.* 2015).

The improvement in the properties of CRM binders is chiefly due to the swelling of CR particles in binders, which absorb oils and resins to form a viscous compound similar to asphaltenes and thereby, increasing the viscosity of the CRM binders. Thus, the performance of CRM binder depends on the amount of CR particles (CR dosage), the presence of oils and resins (base binder), and interaction time between CR particles and binder (digestion time). In this direction, several studies investigated the effect of binder modification. But, the significant influence of each of the mixing parameters and their interaction effects on the improvement of the asphalt binder's strength and viscoelastic properties imparted by the modifiers with the compatible dosages at their interfacial level plays a decisive role towards the overall rutting performance characteristics of the binders. Thus, it is deemed important to understand the effect of each mixing parameter in developing a better performing CRM binder with improved strength and resilient properties.

2 RESEARCH OBJECTIVE AND SCOPE

The objective of this research study was to statistically evaluate the effect of various mixing parameters on the performance of CRM binders. A total of eighteen laboratory binders (three base binders \times three CR dosages \times three digestion times) were used in the study. The scope of the effort included:

- Laboratory preparation of CRM binders at three dosages (10, 20, and 30%) and three digestion time intervals (30, 60, and 90 minutes) using three base viscosity-graded binders (VG10, VG30, and VG40)
- Evaluation of the binder rheological characteristics using temperature-frequency oscillation test
- Formulation of factorial design on Strategic Highway Research Program (SHRP) parameter, $G^*/\sin\delta$ and dissipation energy parameter, $\tan\delta$
- Statistical evaluation of mixing parameters: base binder, CR dosage, and digestion time

3 MATERIALS AND EXPERIMENTAL PROGRAM

3.1 Materials

A blender-stirrer combination was employed to produce the eighteen laboratory blended CRM binders with VG10, VG30, and VG40 as base binders and with CRM dosages of 10, 20, and 30% by weight of each of the virgin

asphalt binders. The CR gradation used was Type-B gradation as specified by the Arizona Department of Transportation (ADOT) and shown in Table 1 (Way *et al.* 2012). CR was added to the base binder, which was maintained at 170-180 °C, and then blended for 30, 60, and 90 minutes at 2000 rpm to achieve a homogeneous CRM binder matrix. The designations of all the eighteen CRM asphalt binders used in the study are given in Table 2.

Table 1. Crumb rubber gradation (Way *et al.* 2012)

SIEVE SIZE, mm	CUMULATIVE PERCENT PASSING, %
2.36	-
2.00	100
1.18	65-100
0.600	20-100
0.425	-
0.300	0-45
0.150	-
0.075	0-5

Table 2. CRM binder designations

BASE BINDER	CRUMB RUBBER DOSAGE, %	DIGESTION TIME , min	DESIGNATION
VG10	10	30	3V1R1
VG10	20	30	3V1R2
VG10	30	30	3V1R3
VG30	10	30	3V3R1
VG30	20	30	3V3R2
VG30	30	30	3V3R3
VG40	10	30	3V4R1
VG40	20	30	3V4R2
VG40	30	30	3V4R3
VG10	10	60	6V1R1
VG10	20	60	6V1R2
VG10	30	60	6V1R3
VG30	10	60	6V3R1
VG30	20	60	6V3R2
VG30	30	60	6V3R3
VG40	10	60	6V4R1
VG40	20	60	6V4R2
VG40	30	60	6V4R3
VG10	10	90	9V1R1
VG10	20	90	9V1R2
VG10	30	90	9V1R3
VG30	10	90	9V3R1
VG30	20	90	9V3R2
VG30	30	90	9V3R3
VG40	10	90	9V4R1
VG40	20	90	9V4R2
VG40	30	90	9V4R3

3.2 Experimental Program

For rheological evaluation, all the eighteen CRM binders were subjected to temperature-frequency oscillation test using a Dynamic Shear Rheometer (DSR) as shown in Figure 1. The oscillation test was conducted on unaged and short-term aged binders over 10 – 0.1 Hz (a total of 21 frequencies) and for a range of 46 – 82 °C (with 6 °C increments totaling seven temperatures). The tests were performed at 12 and 10% controlled strain rate for unaged and short-term aged binders, respectively. Subsequently, the results from the oscillation test were used to estimate the upper Performance Grade (PG) temperatures for both the unaged and short-term aged asphalt binders as per ASTM D7175-05 (2005). In this process, the SHRP rutting parameter: $G^*/\sin\delta$ was used to identify the PG upper temperature in accordance with ASTM D7175-05 (2005).



Figure 1: Dynamic Shear Rheometer (Kinexus DSR+)

4 RESULTS AND ANALYSES

4.1 Oscillation Test

The oscillation test on all the asphalts provided complex shear modulus (G^*), and phase angle (δ) at all temperature-frequency combinations. G^* is the ratio of peak stress to peak recoverable strain, and δ is the lag between stress and strain pulse in a viscoelastic material. From the oscillation test results, $G^*/\sin\delta$ were estimated. $G^*/\sin\delta$ of all the asphalt binders was considered to classify the binders based on the PG at upper temperatures. SP-1 guideline (1997) defines PG upper temperature as the highest temperature at which the magnitude of $G^*/\sin\delta > 1.0$ and 2.2 kPa for unaged and short-term aged binders, respectively. Table 3 summarizes the estimated PG upper temperature range of the various asphalt binders used in the study.

Table 3. Range of PG upper temperatures

DESIGNATION	PG UPPER TEMPERATURE, °C		$\tan\delta$ @ 1.59 Hz & 64 °C
	BASE BINDER	MODIFIED BINDER	
VG10	58 – 64	-	58.53
VG30	58 – 64	-	25.56
VG40	64 – 70	-	9.95
3V1R1	-	58 – 64	19.20
3V1R2	-	64 – 70	8.38
3V1R3	-	64 – 70	4.91
3V3R1	-	58 – 64	18.06
3V3R2	-	64 – 70	5.29
3V3R3	-	64 – 70	3.94
3V4R1	-	64 – 70	8.07
3V4R2	-	70 – 76	4.72
3V4R3	-	70 – 76	2.43
6V1R1	-	58 – 64	7.91
6V1R2	-	64 – 70	5.19
6V1R3	-	64 – 70	3.55
6V3R1	-	64 – 70	7.19
6V3R2	-	64 – 70	3.21
6V3R3	-	64 – 70	2.56
6V4R1	-	70 – 76	3.92
6V4R2	-	70 – 76	4.36
6V4R3	-	76 – 82	2.43
9V1R1	-	58 – 64	6.41
9V1R2	-	64 – 70	4.51
9V1R3	-	64 – 70	3.38
9V3R1	-	64 – 70	6.14
9V3R2	-	64 – 70	2.57
9V3R3	-	70 – 76	2.42
9V4R1	-	70 – 76	3.73
9V4R2	-	76 – 82	2.85
9V4R3	-	76 – 82	2.36

It can be observed that the inclusion of CR escalated the PG upper temperature of CRM binder with respect to the base binders. With the addition of 20 and 30% CR into the base binder, the PG upper temperature increased by two levels (with an increment of 6 °C per level) for all the base binders. The increase in digestion time increased the PG upper temperature indicative of an increase in the binder strength with more digestion time. The influence of the base binder properties on PG upper temperature of CRM binders can also be observed from the oscillation test results. CRM binders with stiffer VG40 base binder produced higher PG upper temperature followed by CRM binders with softer VG30 and VG10 as base binders. The highest PG upper temperature range was observed for 9V4R2 and 9V4R3 binders.

The oscillation test results indicated that the rutting performance of the asphalt binders improved when it was blended with CR. Since the PG upper temperatures were measured by the SHRP parameter: $G^*/\sin\delta$, which explains the stiffness and phase lag simultaneously, it can be also inferred that the CR inclusion in the virgin binders augmented the stiffness of the CRM binders significantly (approximately 1.6 – 1.8 times the stiffness of the virgin binders).

Table 3 also provides the dissipation energy parameter, $\tan\delta$ for all the eighteen binders. $\tan\delta$ aids in evaluating the rutting resistance of the asphalt binders on the basis of the energy concept. As observed, all the CRM binders produced very low magnitude of $\tan\delta$ in comparison with the virgin binders. It is important to note that higher magnitude of $\tan\delta$ indicates more relative dissipation energy, and thereby there is a very high possibility that the asphalt binder will dissipate more energy during a loading cycle. Hence, lower the $\tan\delta$, better is the rutting resistance of the asphalt binder. $\tan\delta$ of the virgin binders was approximately 50-100 times more than that of the CRM binders indicative of the fact that virgin binders showed higher dissipation energy, and in turn would be more prone to rutting. Further, an increase in the digestion time and CR dosages also resulted in lower $\tan\delta$ indicative of improved rutting resistance.

From the $G^*/\sin\delta$ and $\tan\delta$ results, it can also be inferred that all the three parameters including the base binder characteristics, digestion time, and CR dosages affect the performance of the CRM binders. A statistical analysis was carried out on the oscillation test results, namely $G^*/\sin\delta$ and $\tan\delta$ to appreciate the significance of each parameter on the CRM binder performance as discussed next.

4.2 Statistical analysis: Factorial design

In essence, the analyses of $G^*/\sin\delta$ and $\tan\delta$ encompassed three factors: base binder, digestion time, and CR dosage. All factors consisted of three levels: base binder – VG10, VG30, and VG40; digestion time – 30, 60, and 90 minutes; and CR dosage – 10, 20, and 30%. A 3^3 factorial analysis was designed to investigate the main and interaction effect of each factor with the knowledge gained from (Montgomery, 2013). The interaction effect with second degree was considered in the analyses and the higher order interactions were neglected for simplification purposes. The factors and level details of the factorial design are summarized in Table 4.

Table 4. 3^3 factorial design

FACTORS	LEVELS		
	Base binder	VG10	VG30
Digestion time, min	30	60	90
CR dosage, %	10	20	30
Replicates = 2			
Total samples = $3 \times 3 \times 3 \times 2 = 36$			

Factorial analysis was conducted using the statistical software MINITAB® and the ANOVA results are presented in Tables 5 and 6. A two-tailed ANOVA was done at a confidence interval of 95%. The p -value for all the main and interaction effects was less than 0.025 indicative of the significant impact of the factor on the binder performance parameters, $G^*/\sin\delta$ and $\tan\delta$. As observed in Tables 5 and 6, all the main effects and interaction effects of all the treatments (base binder \times dosage; base binder \times digestion time; and dosage \times digestion time) produced a p -value < 0.01 , thus indicating the significant contribution of all the factors on the output parameters. The details of the significant effects of the main and interaction factors are explained later in Section 5.

Table 5. ANOVA for $G^*/\sin\delta$

Source	DF	Adj. SS	Adj. MS	F-value	p-value	Model findings
Base binder	2	179700360	89850180	2525.21	<0.01	Significant
Digestion time	2	10813088	5406544	151.95	<0.01	Significant
CR Dosage	2	87844797	43922398	1234.43	<0.01	Significant
Base binder \times Digestion time	4	3713724	928431	26.09	<0.01	Significant
Base binder \times CR Dosage	4	52796304	13199076	370.96	<0.01	Significant
Digestion time \times CR Dosage	4	2745121	686280	19.29	<0.01	Significant
Error	35	1245344	35581			
Total	53	338858737				

Table 6. ANOVA for $\tan\delta$

Source	DF	Adj. SS	Adj. MS	F-value	p-value	Model findings
Base binder	2	68.01	34	24.55	<0.01	Significant
Digestion time	2	155.38	77.69	56.08	<0.01	Significant
CR Dosage	2	239.57	119.78	86.46	<0.01	Significant
Base binder × Digestion time	4	23.32	5.83	4.21	<0.01	Significant
Base binder × CR Dosage	4	40.87	10.21	7.37	<0.01	Significant
Digestion time × CR Dosage	4	103.55	25.88	18.69	<0.01	Significant
Error	35	48.49	1.385			
Total	53	679.2				

5 EFFECTS OF VARIABLES

As discussed in the earlier Section, the ANOVA results provided an idea of the significant treatments of the experiment. As a next step, the practical understanding of this analysis was evaluated by understanding the individual factor effects (also known as main effects), and the interaction effects of the treatments. The main and interaction effects with regard to $G^*/\sin\delta$ and $\tan\delta$ are shown in Figures 2 and 3, respectively.

5.1 Main effects

The main effect of a variable is basically the influence of the independent input variable on the dependent variables. The effect of each variable is evaluated by considering the difference between the levels of the variable of all the significant treatments with respect to the response variables. The pattern of response variables was further interpreted to understand the in-between difference in the factors.

As observed in Figure 2(a), $G^*/\sin\delta$ increased with stiffer base binder. The viscosity grading of binders indicates that VG40 has higher viscosity followed by VG30 and VG10. Thus, VG40 can be considered and is the stiffer binder out of all the three base binders. As observed, $G^*/\sin\delta$ increased from softer VG10 base binder to harder VG40 base binder. The base binder significantly influenced the stiffness of the CRM binder. The main effect of the base binder was steep with a similar slope indicative of the significant influence of the base binder on $G^*/\sin\delta$ in both segments from VG10 to VG30 and VG30 to VG40.

Figure 2(b) presents the main effect relationship of $\tan\delta$ with all the three factors. It can be observed that $\tan\delta$ decreased with stiffening of the base binders. The magnitude of $\tan\delta$ decreased from softer VG10 base binder to harder VG40 base binder. The innate viscoelastic properties of the base binder significantly influenced the performance improvement in CRM binders. The slope of the main effect plot of the base binder with $\tan\delta$ was observed to be steeper and thereby indicating the significant influence of the base binder on the dissipation energy parameter, $\tan\delta$ of the CRM binders.

Figure 2(a) also presents the main effect of digestion time on $G^*/\sin\delta$. The $G^*/\sin\delta$ magnitude increased with increase in the digestion time. Increased time for CR and binder interaction results in homogeneously blended CRM binders. A steeper slope was observed when the digestion time was varied from 30 to 60 minutes and comparatively flatter slope was seen for $G^*/\sin\delta$ when the digestion time was varied from 60 to 90 minutes. With the increase in digestion time, the rate of improvement in binder performance reduced as indicated by the slope of the plot. Figure 2(b) shows the main effect of digestion time on the dissipation energy parameter, $\tan\delta$. $\tan\delta$ decreased with an increase in the digestion time indicative of better rutting resistance of the CRM binders at higher digestion time. A homogeneous CRM binder obtained at higher digestion time provides sufficient stiffness and elasticity that is not achieved at lower digestion time. It can also be observed the slope of the main effect segment from 60 to 90 minutes was flatter than 30 to 60 min segment, indicative of a reduction in the rate of improvement in $\tan\delta$.

The main effect of the third factor, CR dosage on the output dependent variables, $G^*/\sin\delta$ and $\tan\delta$ are also presented in Figures 2(a) and (b), respectively. From Figure 2(a) it can be observed that $G^*/\sin\delta$ increased with increase in the CR dosage. The addition of CR particles in asphalt binder triggers the swelling of CR particles by absorption of the asphalt binder components and aiding in the improvement of stiffness and viscoelastic properties of the binder. The increase in the concentration of CR particles increased the proportion of the swelled CR particles in the binder leading to a stiffer CRM binder with improved elastic behavior. Similarly, increase in CR dosage resulted in a decrease of $\tan\delta$ magnitude as observed in Figure 2(b). The addition of CR improved the elastic property of the CRM binder, consequently reducing the phase angle of the binder. Phase angle reduction led to a decline in dissipation energy and increase in binder's resistance to rutting.

Overall, it was observed that $G^*/\sin\delta$ increased with an increase in the base binder viscosity, digestion time, and CR dosages. Further, it was also observed that $\tan\delta$ magnitude decreased with increase in the base binder viscosity, digestion time, and CR dosages. It was also found that all the three factors significantly affect the binder parameters, $G^*/\sin\delta$ and $\tan\delta$. Although the individual consideration of the different variables provided a preliminary understanding regarding the dependency of the various treatments with response variables, the analysis of the interaction effects of all the treatments was necessary for better understanding of the CRM characteristics, which are discussed next.

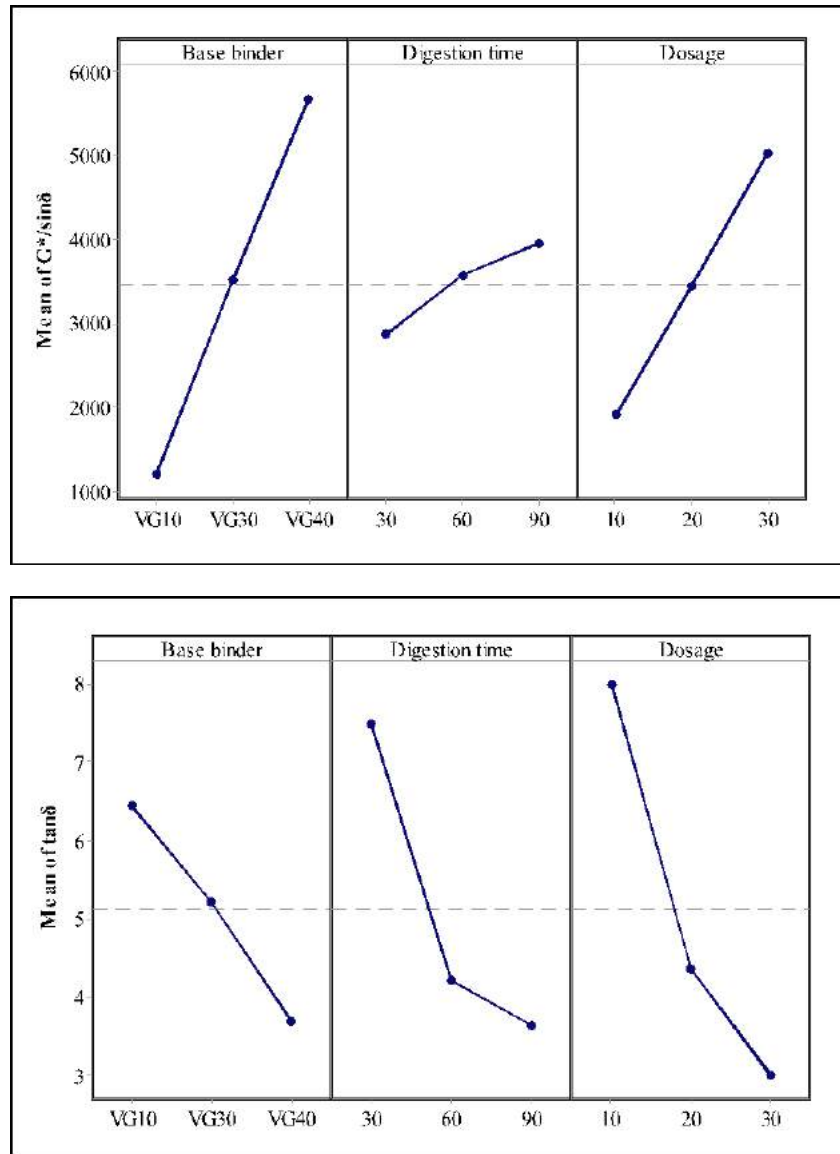


Figure 2. Main effect of variables on (a) $G^*/\sin\delta$ and (b) $\tan\delta$

5.2 Interaction effects

As discussed in Section 4.2, all the interaction effects produced a significant effect on $G^*/\sin\delta$ and $\tan\delta$. A brief discussion on the interaction effects of two-degree factorial is provided here. Figures 3(a) and (b) depict the interaction effects of the variables on $G^*/\sin\delta$ and $\tan\delta$. As observed, Figure 3(a) presents the interaction effect of the base binder \times digestion time on $G^*/\sin\delta$. As shown, base binder \times digestion time interaction significantly affected the $G^*/\sin\delta$ magnitude. For all digestion times, the $G^*/\sin\delta$ increased with base binder viscosity. For VG10, the variation in $G^*/\sin\delta$ at varying digestion times was insignificant compared to the VG30 and VG40 base binders. Further, for VG30 and VG40, a significant difference was found between 30 and 60 min digestion time and comparatively reduced the difference between 60 and 90 min digestion time. Figure 3(b) shows the interaction effect of base binder \times digestion time on $\tan\delta$. A drastic variation in $\tan\delta$ was found between 30 and 60 min digestion time for all the three base binders. In contrast, $\tan\delta$ variation in a lower degree was observed between the digestion times of 60 and 90 min. The results showed that after 60 min, the improvement in the parameters for all the base binders was negligible.

Figure 3(a) also presents the interaction effect between base binder \times CR dosage on $G^*/\sin\delta$. It was observed that $G^*/\sin\delta$ increased with increase in the CR dosage for all the base binders. At 10% of CR dosage, it can be observed that there was very less variation in the $G^*/\sin\delta$ of CRM binders with VG30 and VG40 as base binders. With the increase in CR dosage, a significant variation in $G^*/\sin\delta$ was observed for all the base binders indicative of the improvement in binder rutting resistance and concurrently with an increase in the CR dosage and base binder viscosity. A similar outcome was observed for the interaction effect of $\tan\delta$ between base binder \times CR dosage as shown in Figure 3(b). $\tan\delta$ magnitude decreased with the increase of both base binder viscosity and CR dosage. The variation in $\tan\delta$ was higher between 10 and 20% CR dosage for CRM binders with VG10 and VG30 when compared with CRM binders

with VG40 base binder. Note that VG10 and VG30 contain a higher proportion of low molecular weight binder components, and thus CR swelling will be more in CRM binders with VG10 and VG30 base binders.

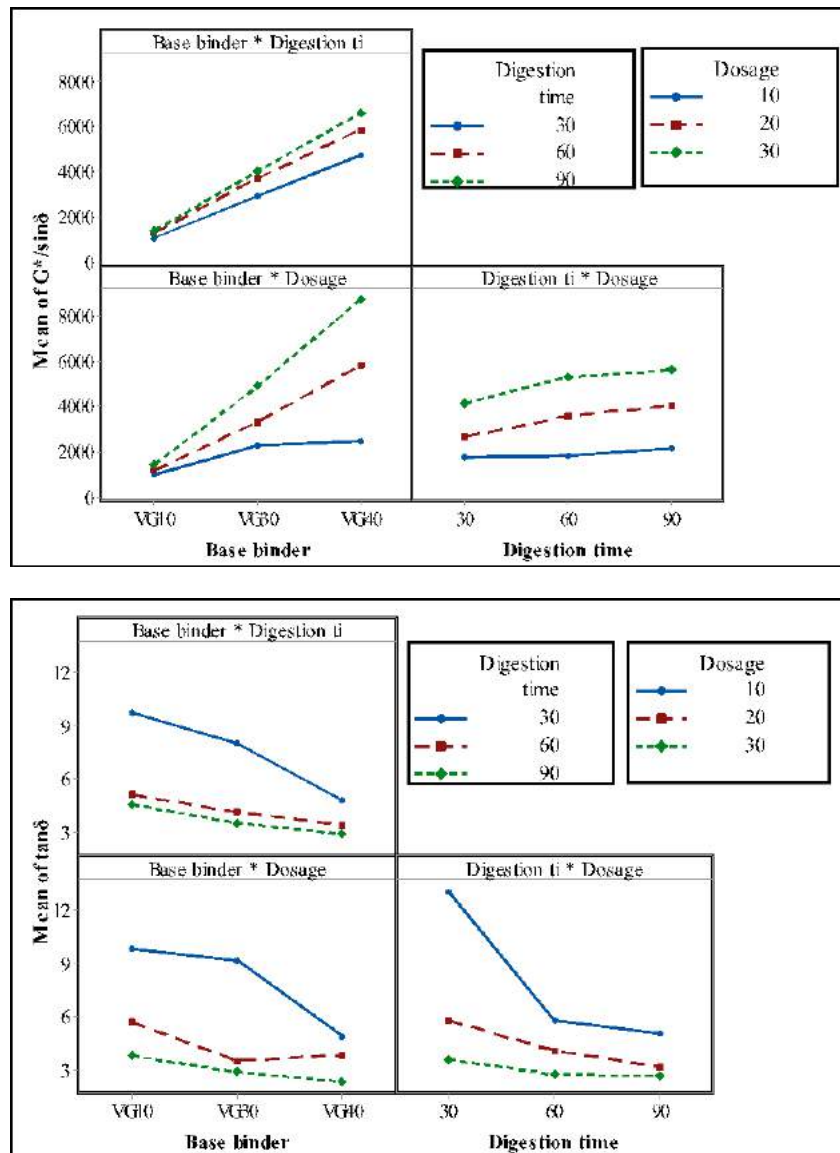


Figure 3. Interaction effects of variables on (a) $G^*/\sin\delta$ and (b) $\tan\delta$

The interaction effect between digestion time \times CR dosage on $G^*/\sin\delta$ is shown in Figure 3(a). Very less variation of $G^*/\sin\delta$ with digestion time was observed at 10% CR dosage, indicative of the presence of fewer CR particles in the binder. At 20 and 30% CR dosage, an increase in $G^*/\sin\delta$ was observed with increase in the digestion time. Higher time intervals of CR and binder interaction lead to more CR particle swelling, and thus producing CRM binder with improved rut resistance. From Figure 3(b), it can be observed that both digestion time and CR dosage significantly influenced the $\tan\delta$ parameter. It was observed at 20 and 30% CR dosages, the reduction in $\tan\delta$ was comparatively less at a digestion time of 90 min, indicative of CR swelling saturation. At digestion times of 30 and 60 min, a significant variation in $\tan\delta$ was observed at all CR dosages.

6 CONCLUSIONS

The objective of the study was to investigate the effect of various mixing parameters on the performance of CRM binders in respect of the binder rutting parameter $G^*/\sin\delta$ and dissipation energy parameter $\tan\delta$. The three mixing parameters considered in the study were: base binder, digestion time, and CR dosage. Further, all the three factors consisted of three levels. Thus, a total of eighteen CRM binders were employed for the investigation.

From the factorial design and ANOVA, it was found that all three factors significantly affected both binder performance parameters, $G^*/\sin\delta$ and $\tan\delta$. As the factors individually and simultaneously affect the binder's performance, main effects and also the interaction effects were studied. Main effects of factors and interaction effects between the factors indicated the variation of output parameters with change in the levels of each factor. The increase in

all the three factors increased the CRM binder's $G^*/\sin\delta$ and decreased the magnitude of $\tan\delta$. Though the performance of binder improved with digestion time, it was found that the difference in performance between 60 and 90 min was less indicative for obtaining saturation threshold. Thus, it can be concluded that stiffer base binder with higher CR dosage mixed at higher digestion time will result in a homogenous CRM binder with enhanced rutting resistance.

Even though this study investigated the influence of mixing parameters on the rutting resistance of asphalt binders modified with crumb rubber, it is deemed important to understand the effect of mixing parameters on the fatigue and low temperature thermal cracking resistance of asphalts due to crumb rubber modification. Also, it is necessary to understand the effect of different crumb rubber gradations on the viscoelastic behavior of the CRM binders. Therefore, more research is definitely needed in this direction to characterize the CRM binders and other modifiers such as polymers, and fibers, etc.

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STRATEGIES FOR ROAD SAFETY IN THE CARIBBEAN – EXPLOITING THE LOWEST HANGING FRUIT

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Challenges have been experienced worldwide in the effective and successful implementation of the five E's of Road Safety. One of the major challenges is related to National Financial considerations. This problem has been a persistent one in the Member Countries of the Caribbean Community CARICOM. It is easy to conclude that the lowest hanging fruit are associated with Pillar 4 – Road User Behavior – along with the E's associated with Education, Enforcement, Encouragement and Evaluation. The Pillar of infrastructure and the E of Engineering are universally known to attract the highest capital costs. This has indeed been the experience of most CARICOM Territories. The paper will attempt to analyze the experiences in this regard from selected CARICOM Territories, confirm or otherwise the benefits to be derived by exploiting the low hanging fruit and recommend modified strategies that could be adopted by all Caribbean Territories and Latin America.

KEYWORDS

Road Safety Economics Education Fatalities

1 INTRODUCTION AND BACKGROUND

1.1 A History of the Caribbean Association of Roads

The Caribbean Association of Roads was launched in the Caribbean in 2008 in Tobago. Its membership is open mainly to Road and Highway Engineers and other Practitioners in the fourteen Territories of the Caribbean Community – CARICOM. Its focus in the Caribbean continues to be

- Road Safety
- Road Maintenance
- The innovative financing of Road Projects

1.2 Background

Road safety must be the priority of every road user. Regard for human life should be the unspoken code that guides every driver, cyclist and pedestrian. Our task is to find innovative ways to have this conversation with the travelling public in as many ways as are needed to drive the message home. This paper will attempt to analyze effective methods that have worked in the Caribbean context. It will explore effective low cost ways for engaging road users in the discussion. This dissertation will explore old methods that worked in the past with new eyes and new ideas for the future. If we wish to change road user behavior in an affordable Engineering Environment, we must start with Education, continue with Encouragement, carry out Evaluations, and when all else fails deter errant behavior with Enforcement.

1.3 The UN Decade of Road Safety

In 2011 the UN Decade of Road Safety was launched. Most of the countries of the United Nations body committed themselves to inter alia, influencing the reduction of road fatality rate at the end of the decade to 50 percent of what it was at the commencement.

In early 2016, the new United Nations Sustainable Development Goals (SDG) were approved. They included a new goal to reduce the 2015 level of fatalities by 50% in 2020. This is far more challenging than the original Decade of Action goal mentioned previously.

1.4 Caribbean Road Fatality Trends

Table I below shows the annual trend in road fatalities and road fatality rates per 100,000 of population for eight selected territories in the CARICOM Caribbean. From the numbers presented, it is noted that both Trinidad and Tobago, and Suriname have demonstrated encouraging downward trends over the first half of this decade. Barbados has maintained its position of being the only country so far to sustain its single digit status. Hopefully some of the others will get there in another year or two.

	2011	2012	2013	2014	2015	2016 -Projected
Jamaica	304	264	306	296	390	350
Jamaica	11.7	10.1	11.8	11.4	14.0	12.5
Trinidad	191	193	151	140	150	150
Trinidad	14.7	14.8	11.6	10.0	11.0	11.0
Guyana	115	110	108	146	125	135
Guyana	15.3	14.7	14.4	19.5	16.7	18.0
Suriname	85	80	76	75	58	55
Suriname	17.0	16.0	15.2	15.0	11.6	11.0
Barbados	23	28	23	20	22	20
Barbados	8.4	10.2	8.4	7.3	8.0	7.3
Saint Vincent	4	6	7	9	25	15
Saint Vincent	3.7	5.5	6.5	8.2	22.7	14.0
Belize	57	64	51	60	87	85
Belize	16.7	18.5	14.6	10.0	25.1	24.9

TABLE I

Trends in annual road fatalities and road fatality rates – 2011 to present

Note: Road Fatality Rates expressed in deaths per 100,000 of population

1.5 Development Banks and Economic losses to a country due to Road Fatalities

1.5.1 The Inter American Development Bank

Various formulae have been developed in the past to assess the negative economic impacts to a country as a result of road fatalities. A recent IDB developed formula postulates that the economic loss to a country resulting from a single road fatality is approximately -

70 TIMES THE PER CAPITA GROSS DOMESTIC PRODUCT FOR THE COUNTRY

For a country with a per capita GDP of US\$ 10,000, this equates to a loss of US\$ 700,000 per fatality.

It should be noted that the current IDB Road Safety Strategy is aligned under the five pillars of the “Decade of Action” with the intention of implementing concrete actions – and measureable results – that contribute towards meeting the target set by the United Nations of reducing the number of road accident fatalities by 50% by 2020. Specifically, the Bank seeks to:

- A. Incorporate road safety components into all transportation operations;
- B. Promote transport operations exclusively focused on improving the countries’ road safety;
- C. Facilitate regional and cross-regional dialogue among the governments; and
- D. Place road safety as a priority in the political agenda of the region’s governments; and
- E. Create a culture of change and civil responsibility in the subject of road safety by raising awareness among the population of the negative impact they will suffer while irresponsible traffic attitudes and behaviors persist in society.

1.5.2 The Caribbean Development Bank

In March 2012, the Caribbean Development Bank responded to a request from the Government of Belize which resulted in the CDB currently assisting in the financing of a multi-sectoral Road Safety intervention that includes infrastructure improvements and capacity building for agencies involved in road safety management. The budgetary cost of this program is approximately US\$5M.

More recently, the Government of Guyana benefitted from a modest sum of US\$200,000 for a School Road Safety Education Program which formed a small component of a US\$35M Highway Project.

3 DISCUSSION

3.1 The Highest Hanging Fruit

We can consider some of the highest hanging fruit on the road and highway safety tree to be

- Divided Highways
- Anti Collision Devices
- Roundabouts and Overpasses

3.2 The Higher Hanging Fruit

Some examples of higher hanging fruit not incurring capital costs as high as the highest hanging fruit would be

- Highway Lighting
- Traffic Signing
- Road Marking

3.3 The Lowest Hanging Fruit

3.3.1 Road Safety Education from Kindergarten to University

- Kindergarten

The Guyana Red Cross Society has designed and is promoting a coloring book for preschool children. The book graphically depicts the do's and don't's of Road Safety and is full of illustrations for the kids to color.

- Primary

School Traffic Patrols should play an integral part in all Primary Schools in CARICOM Territories.

- Secondary

Secondary School Pupils should be encouraged to take part in Radio and Television Quizzes on Road Safety.

- Tertiary

Final Year Projects should be developed based inter alia on statistical analyses of traffic accident black spots.

- Defensive Driving

CARIBBEAN ASSOCIATION OF ROADS



DEFENSIVE DRIVING TIPS

DO A DEFENSIVE DRIVING COURSE

DRINK RESPONSIBLY

DRIVE WITHIN THE SPEED LIMIT

AVOID DISTRACTIONS

DON'T DRIVE WHEN DROWSY

WEAR YOUR SEAT BELT

TAKE EXTRA CARE IN BAD WEATHER

DON'T FOLLOW TOO CLOSELY

LOOK OUT FOR THE OTHER GUY

CHECK YOUR VEHICLE REGULARLY

PRACTISE DEFENSIVE DRIVING

3.3.2 The Stakeholders of the Lowest Hanging Fruit.

Many effective stakeholders of Road Safety Public Awareness already exist in our society. The secret is to employ these stakeholders in a smart way. Some of these stakeholders are briefly discussed below.

- National Road Safety Councils

Currently there are four NRSCs actively in operation in the CARICOM Caribbean. These are located in Guyana, Trinidad and Tobago, Barbados and Jamaica. The oldest and most dynamic of the four exists in Jamaica. (Public Awareness role of NRSCs + Umbrella Body). (Initiatives for CARICOM to get its Health Desk to include Road Safety on its Agenda)

- Youth Groups for Road Safety

Young people all over the Caribbean are involved as the perpetrators and victims of a substantial number of serious and fatal road accidents. Within the last year or two, the territory of Belize has introduced an entity Belize Youths for Road Safety (BYOURS) which has dynamically interacted with the Belizean Society in organizing a number public awareness related road safety programs.

- Insurance Companies

Members of CAR have engaged Insurance Companies of several territories of the Caribbean who have spasmodically shown interest in investing in curbing road casualties. Profits of Insurance Companies are reduced as Road Accident and Fatality Rates increase.

- Safety Companies

JARIC – is a Trinidad based company providing Environmental, Safety and Health Services in the Southern Caribbean. including, inter alia, fleet training in Defensive Driving, First Aid & CPR. In the current decade they have successfully organized several Road Safety Conferences in the Southern Caribbean.

- Chambers of Commerce

In Guyana, the Georgetown Chamber of Commerce has partnered with the Police Traffic Department and donated a number of Speed Guns in their interest in making the roads safer.

- Newspapers

The two Excerpts shown below are examples of reporting which effectively stimulate heightened road user public awareness.

Hit-and-run results in seventh road fatality of the year

BY: *LOOP NEWS*
15:38, February 23, 2016

Eyewitnesses said that Alimoenadi was run over by either a truck or tanker after his moped had fallen on the road. They said that the driver who struck Alimoenadi continued on without stopping. He has yet to report to the police.

By the time police were on the scene, Alimoenadi had already died. He marks the seventh road fatality of this year.

An autopsy will be conducted and Richelieu police will continue to investigate the case.

Excerpt I

TRINIDAD AND TOBAGO

Carapichaima man dies in accident

By ALEXANDER BRUZUAL Friday, September 17 2010

***THE ROAD fatality rate climbed once again on Wednesday night when a 50-year-old Carapichaima man was killed in an accident, bringing the number of road deaths for the year to 144, 12 less than the figure of 156, for the corresponding period last year.**

Excerpt II

- Billboards

The Suriname population is informed on a weekly basis by several suitably placed Billboards with updated Fatality Statistics. These have proven over the years to be a very effective public awareness tool.

- Radio and Television Stations

A Radio and Television Station in Guyana is preparing to host a monthly Road Safety program highlighting road fatality statistics, speed gun demonstrations, and defensive driving techniques.

- Social Media

Facebook and other Social Media are gaining in popularity as effective safety tools of mass communication among road users .

- Funding Agency Contributions and Experiences

It is encouraging to note the modest investments of the Regional Banks in Road Safety as described in paragraph 1.5. Once other stakeholders continue to become more aware of the impressive magnitude of economic losses to the country resulting from Road Accidents, we can expect to see an increase in the level of investments focused on the soft solutions.

- Enforcement

Table II presents the relationship between numbers of Traffic Enforcement Personnel and Populations in eight CARICOM Territories. Although a detailed scientific analysis is possible at this time, the numbers suggest that there should be the recruitment of additional personnel in several countries. The cost of the additional personnel could be expected to be covered by the reduction in economic losses resulting from a smaller number of accidents.

		Population	'Rate' per 100000
GUYANA	220	750000	25
SURINAME	85	500000	17
TRINIDAD AND TOBAGO	500	1400000	35
BARBADOS	40?	275000	15
SAINT LUCIA	20	160000	13
SAINT VINCENT	20	120000	17
JAMAICA	300	2800000	12
BELIZE	60?	330000	18

TABLE II

Relationship between numbers of Traffic Enforcement Personnel and Populations in 8 CARICOM Territories.

5 CONCLUSIONS

5.1 Development Banks

Both the Inter American Development Bank and the Caribbean Development Bank are currently actively involved in investing in Road Safety initiatives as part of their Highway Investment portfolio in the Caribbean. This laudable initiative needs to be further pursued in the future.

5.2 Trinidad and Tobago and Suriname are two territories of the CARICOM Caribbean which have been exhibiting continuing encouraging downward trends in Road Fatality Rates in recent years.

5.3 The traditional media have been playing an important role in heightening Public Awareness about Road Safety matters in some Caribbean Territories.

5.4 The Social Media can play a major role in stimulating Public Awareness in Road Safety matters.

6 RECOMMENDATIONS

- 6.1 Public Relations activities on Road Safety via Radio Stations TV Stations, the printed Media, Facebook and other Social media should be substantially increased.
- 6.2 The concept of dynamic Youth Groups for Road Safety should be embraced and effectively implemented throughout the Caribbean.
- 6.3 Defensive Driving Programs in the CARICOM Caribbean should be extensively proliferated.

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PAPER TITLE	EVALUATION OF AUTHORISED LEFT (ALT) TRAFFIC CONTROL: A CASE STUDY IN KUALA LUMPUR		
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KEYWORDS:

Include up to 5 keywords

LTOR, Authorised Left Turn, Efficiency, Safety

ABSTRACT:

The abstract should be written in English, readily understandable to most readers and may contain up to maximum of 500 words. Authors are invited to use Times New Roman Size 10 and the full type width of the page (single column).

Different forms of Left Turn on Red (LTOR) traffic control have been operated in Malaysia for several years. However, the efficiency of LTOR in improving traffic flow at junctions has yet to be proven as no study has been conducted thus far. Due to demand for its implementation nationwide, this practice was gazetted under the Malaysian Road Transport Act and was reintroduced as Authorised Left Turn (ALT). Due to some safety concerns, MIROS had produced a guideline for the installation of this traffic control for use by the local authorities in 2012. This study was undertaken to assess the ALT from the efficiency perspective such as, violation rates and vehicle delay (LOS). Data were collected at three junctions, before and after ALT installation. The results showed that ALT slightly improved the capacity. Most of the left turn on red movements violated the ALT rule of which motorists need to come to a complete stop before performing ALT. Multiple Regression Model suggests that types of vehicles, conflicting vehicles and total approach volumes significantly affect the ALT violation rates. Based on the findings, it can conclude that the provision of ALT traffic control improves the junction capacity at certain degree without prominent detriment to the safety. It is recommended to install ALT traffic control at junctions which fulfil the criteria outlined in the guideline. It is also suggested to constantly monitor the performance of ALT as the behaviour of motorists change over the time which in long run would expose other road users to the risk of collision.

Evaluation Of Authorised Left (ALT) Traffic Control: A Case Study In Kuala Lumpur

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1 INTRODUCTION

Left Turn On Red (LTOR) or Right Turn On Red (RTOR) traffic control has been implemented in the United States since 1937 (Green, 1980) and is widely adopted in many countries today. This traffic control scheme allows a vehicle on the left turn lane (LTOR under left-hand drive traffic system) to turn left after stopping and giving way to oncoming vehicle / pedestrian. The objective of this scheme is to increase the operating efficiency at junctions. Nevertheless, the benefits and the deficiencies are still being debated today. Though studies have proven that it improves traffic capacity, it is at the same time creating safety problems.

LTOR traffic control has been implemented in several states in Malaysia for years but not without safety issues. Many have complained about the non-standard sign used and safety issues with regard to near-misses between vehicles and pedestrian. A Task Force was set up in 2011 to examine the feasibility of this provision. Based on the preliminary findings, a technical guideline on the implementation of LTOR in Malaysia was produced by MIROS (Alvin et al., 2012). Due to the high demand, the Authorised Left Turn (ALT) traffic control has been widely provided at several signalized junctions in Kuala Lumpur.

2 LITERATURE REVIEW

Right Turn On Red (or left turn on red in countries with left-hand traffic) traffic control was introduced in the United States in the 70's as part of an effort to conserve energy. In fact, it started as early as in 1924 in New York City but was terminated in 1937 though it had significantly contributed in fuel saving and improved traffic capacity (Preusser et al., 1981). Throughout the periods until the 70's, the implementation of RTOR was limited due to issues related to its overall benefits and the negative impact it has on the safety of road users. The turning point was after the Arab Oil Embargo from 1973-1974 and when the Energy Policy and Conservation Act was passed in 1975 which required the states to develop an energy conservation plan to source funding from the Federal Government. Since then the implementation of RTOR was fast gaining popularity and by 1977 almost all states permit RTOR at many signalized junctions. Nevertheless, there were various types of RTOR rules: Totally Prohibited, Generally Prohibited except signed to allow, Generally Permitted except where signed to Prohibit and Totally Permitted.

In view of the varying rules governing the implementation of RTOR, there is a need to have a national policy and guidelines to be adopted. Several studies were then conducted. In general, Mamlouk (1976) found that RTOR consist of 19.5% of total right turns and 1.5% LTOR were out of total left turns. Important factors that affect RTOR movements were signal type, city size and availability of exclusive right turn lanes. Conflicts between the turn on red vehicles with cross traffic did not cause significant problem and also it did not cause significant hazard to pedestrians. Other findings include 1.3 seconds travel times per vehicle (9%) saved in central business districts and an average of 7.7 seconds travel times per vehicle (39%) saved in rural areas as well as 0.3 to 1.7 hours travel times per vehicle saving if accumulated on a year basis (McGee et al., 1976 as cited in Preusser et al., 1981; Clark et al., 1983). Based on these findings, the FHWA suggested that the generally permissive rule be adopted as the national policy. A set of national uniform guidelines in prohibiting RTOR at hazardous junction was then developed based on the generally permissive rule.

On the other hand, the opponents claimed that pedestrian safety had been compensated by the RTOR. Zador (1980) showed a significant increase in pedestrian crashes during the period 1974-1977 when RTOR was implemented in six of the states. Overall, the implementation led to an increment of 57% in single vehicle-pedestrian crashes. Similarly, Preusser (1981) in another study before and after RTOR implementation showed an increase in pedestrian crashes in New York (45%), Wisconsin (86%), Ohio (66%) and New Orleans (96%). On the other hand, South Carolina and Alabama reported no significant evidence that pedestrian crashes increased as a result of RTOR. Clark et al. (1983)

in their analysis indicated that the proportions of crashes involving pedestrians at RTOR junctions and at the control sites did not differ significantly in both the states.

Singapore has adopted sign permissive LTOR traffic control in 1997 with a GIVE WAY sign to regulate the left turning vehicles. Due to majority of motorists failing to come to a complete stop and give way to pedestrian and on-coming vehicles, the LTOR traffic control was changed from a standard “GIVE WAY” sign to “STOP Before Turning” sign. The revised operation is that motorists can turn left on red after a complete stop and give way to on-coming vehicles and pedestrians. A study by Ho (2003) showed that LTOR in Singapore has resulted in time saving and better capacity for left-turning traffic. However, the study had not proven the risk of accident between left turn on red vehicle with pedestrian or on-coming vehicle.

In Malaysia, LTOR has been adopted by several local councils in different parts of Malaysia without a national standard guideline. Realizing the potential time savings as well as the safety deficiencies, a Task Force led by MIROS was set up in 2011 to look into the feasibility of nationwide implementation. MIROS has therefore carried out a preliminary study in Kuala Lumpur to assess the various road characteristics and outline installation criteria in line with existing road design guidelines. Based on the outcome of the study, the LTOR traffic control was adopted in the Road Transport Act and renamed as Authorised Left Turn (ALT). A technical guideline on the implementation of ALT in Malaysia had been published in 2012 to ensure the uniformity of ALT installation in the country (Alvin et al., 2012). The guideline contains the criteria for selection of appropriate junction, the design requirements of ALT sign and best practice with regard to the installation of the sign.

3 DATA AND METHOD

A before-after study design was adopted to evaluate the ALT implementation at three junctions (with a total of eight approaches) installed by the Kuala Lumpur City Hall (DBKL). Several months of baseline data were collected prior to the installation of ALT using video cameras recordings. Video cameras were located at the opposite direction of ALT approach to allow the extraction of parameter such as turning movement, approach stopped vehicles and conflict from the video. The recordings were conducted 3 days a week (Monday, Wednesday, Friday) during morning peak (2 hours), morning off-peak (1 hour), evening peak (2 hours) and evening off-peak (1 hour). The after period was conducted within two to three months after the ALT installation.

The three junctions namely Persiaran Alam Damai, Jalan 21/154 and Jalan Dutamas 1 were assessed for the traffic operation performances during before and after ALT installation periods. The three junctions represent the different level of road hierarchy. Two study hours were identified: 7am – 8am during morning peak and 10 am – 11 am during off peak. Afternoon off peak and evening peak traffic operations were also collected but they are not discussed in this paper as the traffic patterns did not differ much from the morning sessions.

3.1 Study Measures

Parameters such as traffic volume by types, vehicle queues at left turn approach and ALT violation rates were extracted from the video. Violation in this case refers to vehicles turning on red without stopping at the stop line. The total left turning volumes consists of left turn on green vehicles and ALT vehicles.

3.2 Analytical Model

Multiple Linear Regression models were developed to examine the influence factors on ALT violation rates. Violation rates in this study refer to the left turn vehicle which fail to stop before performing turn on red. It is mandatory for ALT vehicles to stop before making a turn on red. Review of literature noted that several factors such as approach lane, volume and median contribute to traffic violation. These suggested variables were included in the model by using the IBM SPSS programme to determine their contributing effect in influencing the ALT violation rates. Stepwise regression method was used to remove the variables that were not significant.

4 RESULTS

Persiaran Alam Damai is a collector road thus high traffic volumes were observed. The traffic volumes for before and after ALT installation had somewhat similar profiles. About 4400 vehicles were recorded during the morning peak which saw a reduction by about 35% after the morning peak. Majority of the traffic were passenger car/MPV/van (55% - 88%) while motorcycle comprised of 12.5% to 43.5%.

About 1400 to 1600 vehicles per hour were recorded during the morning peaks for both before and after ALT installation at Junction Jalan 21/154. The passenger car/MPV/Van accounted for 75% - 90% while motorcycle made up of 8% to 25%.

Jalan Dutamas is located in the Kuala Lumpur city centre and traverses shopping centres and government offices. High traffic volumes were observed at this junction. About 3000 to 3200 hourly traffic during the data collection periods (before and after periods). There was no significant difference in the traffic volume during the two periods of time. Of the total volumes, motorcycle consists of 10% - 33% while passenger cars/van represented 65% - 89% of the traffic.

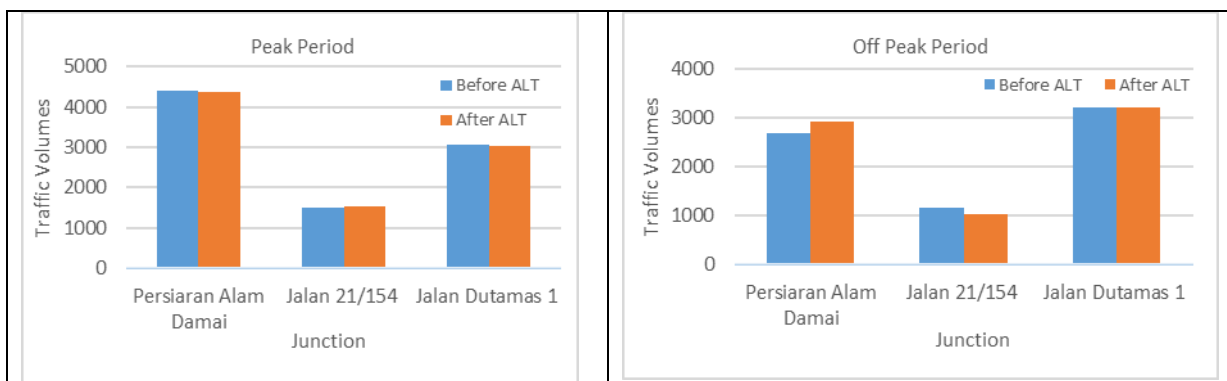


Figure 1 Hourly traffic volume at the study sites during before and after periods

Table 1 presents the percentage of motorcycles and passenger cars turning on red phase during before and after ALT installation periods. It was shown that even before the installation of ALT, there were substantial number of turning on red movements at the junctions. A simple Chi-Square was then performed to examine if the ALT had increased the turn on red opportunity. Nevertheless, no significant changes in the turning on red movement between the two periods at all the junctions.

Table 1. Percentage of left turn on red movements and Chi-Square analysis

		Motorcycle (%)		Chi-Square Test	Car (%)		Chi-Square Test
Persiaran Alam Damai	Morning Peak	48	38.6	Not significant	6	2.8	Not significant
	Off Peak	40.8	24.7		9.5	0.7	
Jalan 21/15	Morning Peak	49.2	66.5	Not significant	22.4	30.2	Not significant
	Off Peak	45.6	63.3		18.6	38.3	
Jalan Dutamas	Morning Peak	38.4	71.65	Not significant	2.8	9.25	Not significant
	Off Peak	51.9	59.4		6.7	5.4	

The output of LOS performance at the junctions by using SIDRA did not show much improvement after the installation of ALT. As the SIDRA software evaluate the junction as a whole, the time savings by certain type of vehicle such as motorcycle might not be reflected in the results. Nevertheless, there were evidences of capacity improvement for motorcyclists on sites. Table 2 presents the average total vehicle queues at the junctions during before

and after. The capacity at Jalan Dutamas during morning peak and at Jalan 21/15 during off peak increased slightly but a simple Paired t test did not indicate any significant changes in the capacity.

Table 2. Paired t test results for average total queues at each junction for before and after ALT installation period

Junction	Time	Before	After	Difference	Paired-t test
Persiaran Alam Damai	MP	74.97	75.67	0.70	t=0.0226, p=0.492
	Off peak	62.83	67.03	4.20	
Jalan 21/15	MP	15.80	16.47	0.67	t=0.3424, p=0.3823
	Off peak	14.67	13.27	-1.40	
Jalan Dutamas	MP	32.13	26.90	-5.23	t=-0.5021, p=0.3327
	Off peak	19.63	24.67	5.03	

Table 3 presents the total interaction and unsafe interaction observed at the junctions during before and after ALT installation. A simple Chi-square analysis was performed to evaluate the effect of the ALT and it shows that there was no significant evidence to prove that ALT increase the traffic risk to the road users. Nevertheless, it should be noted that the observation was conducted within 3 months of ALT installation. The behavior of road users might be changed over the time. Hence it is still inconclusive to justify the effectiveness of ALT.

Table 3. Chi-Square analysis for unsafe interactions

Junction	Time	Period	Total Interactions	Unsafe Interactions	Chi-Square Test
Persiaran Alam Damai	Morning Peak	Before	160	45	Not significant
		After	184	75	
	Off Peak	Before	30	10	Not significant
		After	47	21	
Jalan 21/15	Morning Peak	Before	20	8	Not significant
		After	18	9	
	Off Peak	Before	20	6	Not significant
		After	19	8	
Jalan Dutamas	Morning Peak	Before	103	36	Not significant
		After	133	61	
	Off Peak	Before	25	12	Not significant
		After	39	19	

Statistical modeling of ALT violation data was conducted. Multiple Linear Regression models was developed to examine the influence factors on ALT violation rates. Violation rates in this study refer to the left turn vehicle which fail to stop before performing turn on red. It is mandatory for ALT vehicles to stop before making a turn on red. Review of literature noted that several factors such as approach lane, volume, median and others contribute to traffic violation.

Table 4 lists the attributes used in the Multiple Linear Regression analysis of ALT vehicles turning without stopping which was set as the dependent variable. Independent variables include the followings and it is noted that only motorcycle and car were included as the rates of violation by other vehicles were too little. Other factors such as lane width, turning radius, topography, cycle length, land use are not studied in this project.

Table 4. Variables considered in Multiple Linear Regression Analysis

Variable Description	Data Format
Day of week	Monday Wednesday Friday
Time of day	7 am – 8 am 8 am – 9 am 10 am – 11 am 3 pm – 4 pm 5 pm – 6 pm 6 pm – 7 pm
Type of vehicle	Car Motorcycle
Lane configuration	Exclusive Shared lane
Oncoming vehicle	Total volume of the oncoming vehicles
Total approach volume	Total approach volume of the left turning vehicles
Left turn green arrow	Presence of protected left turn green arrow: Yes No
Receiving Lane	Number of receiving lane 1 2

Table 5. Results of Multiple Regression analysis

Variable	B	Std. Error	t	p-value
Constant	-14.246	9.755	-1.460	0.146
Day of week	0.271	0.768	0.353	0.725
Time of day	-0.144	0.932	-0.154	0.878
Type of vehicle	39.598	2.436	16.255	0.000*
Lane configuration	0.914	2.779	0.329	0.743
Oncoming vehicles	-0.031	0.008	-3.874	0.000*
Total approach volume	-0.030	0.007	-4.366	0.000*
Left turn green arrow	-1.429	2.744	-0.521	0.603
Receiving Lane	1.703	3.615	0.471	0.638

* Significant at $\alpha = 0.05$

The overall model as shown in Table 5 was found to be significant (p-value = 0.00 for F statistics = 37.79). The results of the analysis suggest a relatively strong relationship as explained by the model $R^2=0.655$ with three significant explanatory variables. It is expected that type of vehicles (motorcycle and car) was significant in influencing the rates of violation given that the ease man oeuvre of the motorcycle in traffic stream. As expected, conflicting vehicles affect the violation rates as the volumes of oncoming vehicles reduce the chances of turning during the approach red phase. Higher approach volumes would have higher volume of ALT.

5 DISCUSSION AND CONCLUSIONS

In general, the provision of ALT did not create serious conflict for the motorists (based on site observation). As there was no pedestrian movement at the junctions, this study was unable to assess the effect of ALT on pedestrian safety. Furthermore, no accident crashes at the junction was recorded and it is too early to judge the safety impact of ALT at the moment.

The provision of ALT traffic control at the study sites seems to have some potential gains in traffic capacity though the LOS output (by SIDRA software) was not apparent. The difference of left turning movements between before and after periods was significant at some approaches. This may be due to several constraints such as shared lane, single receiving lane and side parking that have reduced the opportunity for left turning. Besides, it was also observed

on site that substantial vehicles chose to stop for green light may be due to the absence of ALT flashing amber. By and large, the results brought to the conclusion that it improves the traffic capacity without apparent increase of conflict.

On the other hand, it is also important to highlight that the provision of protected left turn green arrow can be hazardous to the motorists. This is because the left turning drivers may not always be aware of the actual activation or end of the protected right turn green arrow. The driver may continue to turn left at the same speed as if green arrow is onset. In the situation where there is right turning vehicle at high speed, collision might occur and the degree of severity would be very serious due to high speed.

Modeling of factors influencing the percentage of left turn on red movement by using Multiple Regression analysis suggests that the conflicting volume and approach volume are important in deciding the turning opportunity. High conflicting volume would reduce the opportunity of ALT movement even increase the risk of collision. On the other hand, high volume of ALT traffic in a relative moderate conflicting traffic stream can improve the stopped delay time. It is also as expected that type of vehicle was significant in the model. Motorcycle with a smaller physical size is easier to maneuver through the left turn bend.

In short, the provision of ALT traffic control improves the junction capacity to a certain degree without prominent detrimental safety effects.

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PAPER TITLE	Advanced Use of Solar Heat-blocking Pavement Technology		
TRACK	Adaption of Road Pavements to Climate Change		
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KEYWORDS:

Climate change; Solar heat-blocking pavement; Reduction in surface temperature; Repair; Small scale application.

ABSTRACT:

In recent years, there is growing awareness on environmental issues due to global warming. In addition, urban areas have been suffering from emerging issue called “Urban Heat Island” which significantly affects pedestrians as well as asphalt surfaces in terms of temperature. In order to tackle the problem, Solar Heat-blocking Pavement technology was developed to gain the following effects: the reduction of surface temperature and the prevention of pavement deterioration. This paper describes the recent development of the technology and its practical effects through experiments and case studies. The following conclusions are drawn from this study. With regard to the performance, the field results indicate that the reduction in surface temperature by use of the solar heat-blocking pavement is approximately 16°C. In terms of application, innovative coating method is highlighted in order to make the technology user-friendly, as well as environmentally friendly.

Advanced Use of Solar Heat-blocking Pavement Technology

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1 INTRODUCTION

In recent years, there is growing awareness that temperatures have been rising worldwide due to the emerging issue of global warming and climate change. This kind of problem has become more pronounced, especially in Japan, where the air temperature during recent summers has tended to set new records, the surface of asphalt pavements may reach 60°C or more, the number of patients requiring ambulances due to the summer heat (i.e. heatstroke) has been increasing, and all in all, the increase in temperatures during summer has become a serious problem. Also, as asphalt pavements cover approximately twenty per cent of urban areas, it is considered to be a factor in the “urban heat island phenomenon” which significantly affects the thermal comfort of pedestrians. Past study suggested that dark asphalt paving absorbs solar rays, including invisible solar radiation (Pomerantz et al.). According to Yoder and Witzak (1975), rising temperatures from the paving surface affect the properties of the surface layer. Therefore, bearing in mind these problems, reduction in the surface temperature has become increasingly important in terms of sustainability as well as the environment.

Pomerantz *et al.* (2000, 2003) indicated that making urban surfaces whiter to reflect both visible and infrared rays (i.e. sunlight) is the most practical way of mitigating surface heat. However, out of consideration for driver’s vision, darker surfaces (e.g. grey) would be preferred as they are more familiar. In addition, taking into account the workability on the existing pavement, surface treatment with a painted coating is one way to deal with the issue on the existing surface. For these reasons, a new surface treatment technology called “solar heat-blocking pavement”, which aims to reduce surface temperatures, has been developed in Japan.

The aim of this paper is to highlight the fundamental study of the solar reflective coating technology on pavements, and addresses practical effects through experiments and field applications.

2 SOLAR HEAT-BLOCKING PAVEMENT

2.1 BASIC CONCEPT

Solar reflective technology was originally developed to mitigate the rising temperature of building roofs (Yoshinaka, 2003). Pomerantz *et al.* (2000) suggested that sealing a reflective material onto the surface layer contributes to both a reduction in surface temperature and the mitigation of surface damage due to ultraviolet rays. As a result, a reduction in surface temperature during summer was expected by applying this technology to asphalt paving.

The function of this technology is based on the higher reflectivity of near-infrared rays and lower reflectivity of visible rays. In practice, the reflectivity of solar rays and infrared rays are represented by albedo (Yoshinaka, 2005). Albedo is defined as the ratio between incoming and reflected solar rays. A higher albedo means that the surface layer has a higher reflectivity for infrared rays, whereas a lower albedo indicates that infrared rays are absorbed into the surface layer, thus increasing the surface temperature. In order to prevent the surface layer from absorbing infrared rays, which contribute to its heating, paint-based materials with a higher albedo were coated onto the existing paving (see Figure 1).

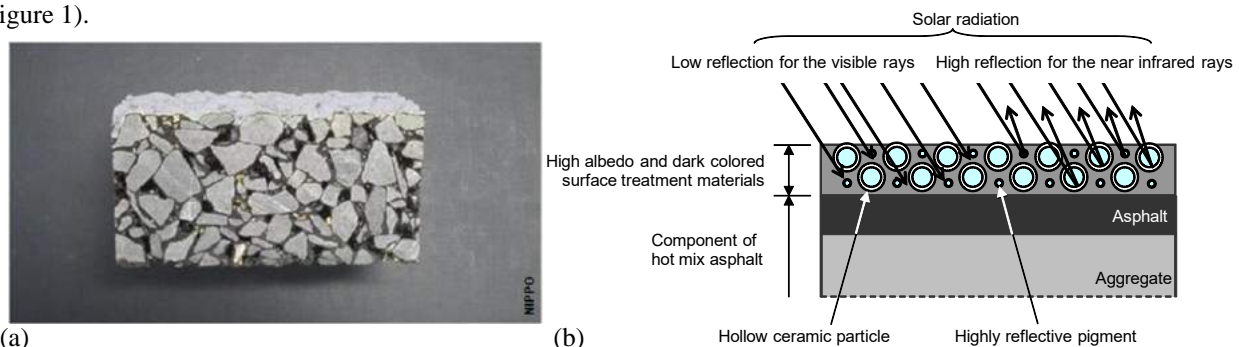


Figure 1. Solar Heat-blocking Pavement: (a) Photograph, and (b) Schematic image

2.2 PROPERTIES OF SOLAR HEAT-BLOCKING PIGMENT

In order to examine the albedo characteristics, a comparison was made between three surfaces: solar heat-blocking pavement, conventional pavement (i.e. dense-graded asphalt pavement) and normal paint material. The test was conducted in accordance with the Japanese standard, JIS A 5759 (1994). In this case, the colour for both the solar heat-blocking pavement and the normal paint is grey, whilst that of the conventional pavement is black.

Figure 2 shows the comparison results. As shown in the figure, there are clear differences between the solar heat-blocking pavement and conventional painting materials. Conventional painting material has almost the same or a less reflective ratio across the entire wavelength. However, in the case of the solar heat-blocking pavement, the reflective rate for near-infrared rays in the wavelength is much higher than others. This indicates that the solar heat-blocking pavement has higher albedo, despite the fact that the normal paint material is the same colour as the heat-blocking pavement.

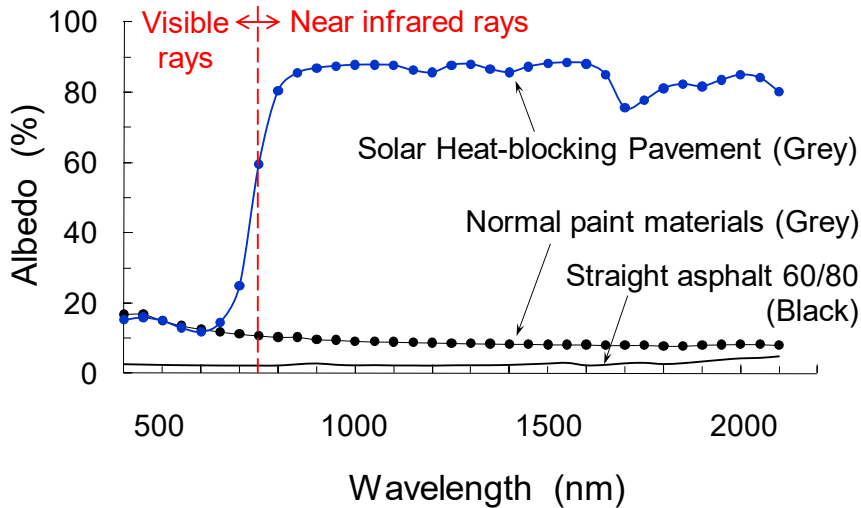


Figure 2. Albedo characteristics of solar reflective pigment

2.3 STRUCTURE OF THE COATING LAYER AND CONSTRUCTION METHOD

The coating layer is about 1.0-mm thick. The coating layer consists of three components: prime layer, second layer and non-skid sand (see Figure 3). Firstly, the prime layer is applied to cover the existing surface; then the non-skid sand is sprayed on immediately after the primary coating to ensure skid resistance; and finally, the second layer is applied as the coloured surface and to sandwich the non-skid sand. After curing for an hour, the site can be reopened to traffic. The coating and spraying work is normally performed with a specialist spray gun as shown in Figure 4. The material specification generally used for this pavement is as follows:

- Size of non-skid sand particles: 0.5 ~ 1.7 mm;
- Density of non-skid sand: 0.5 kg/m²;
- Density of coating: 0.4 ~ 0.6 kg/m² in each layer.

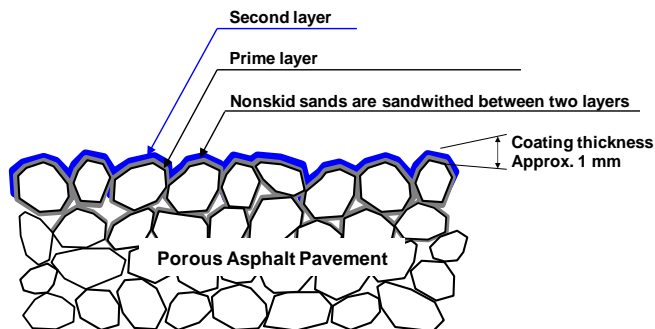


Figure 3. Structure of Solar Heat-blocking Pavement



Figure 4. Application

3 SURFACE PROPERTIES

3.1 SKID RESISTANCE

Skid resistance is a factor affecting the serviceability of paving. It is also closely related to traffic accidents, especially on rainy days. Therefore, non-skid sand is sprayed on immediately after coating with the solar heat-blocking material. For confirmation purposes, the skid resistance of both coated porous asphalt and uncoated porous asphalt surfaces were measured using a Dynamic Friction tester. The results are shown in Table 1.

Table 1. Skid Resistance Results

Dynamic Friction (μ)	Test speed	Porous asphalt paving	Solar Heat-blocking Pavement
	40 km/h	0.60	0.62
	60 km/h	0.55	0.56

Tests were conducted as per ASTM E1911 (2002). From this result, it was found that the skid resistance values of the two surfaces are almost identical. Therefore, it can be said that an appropriate amount of sand needs to be sprayed on to ensure skid resistance.

3.2 STRIPPING RESISTANCE

Considering the service state, the coating layer of solar heat-blocking pavement should be strong enough and have good bonding to the original asphalt surface. In order to investigate these factors, two types of laboratory stripping-resistance tests (Torsion test and rebelling test) were conducted. A schematic representation of the test equipment is shown in Figure 5.

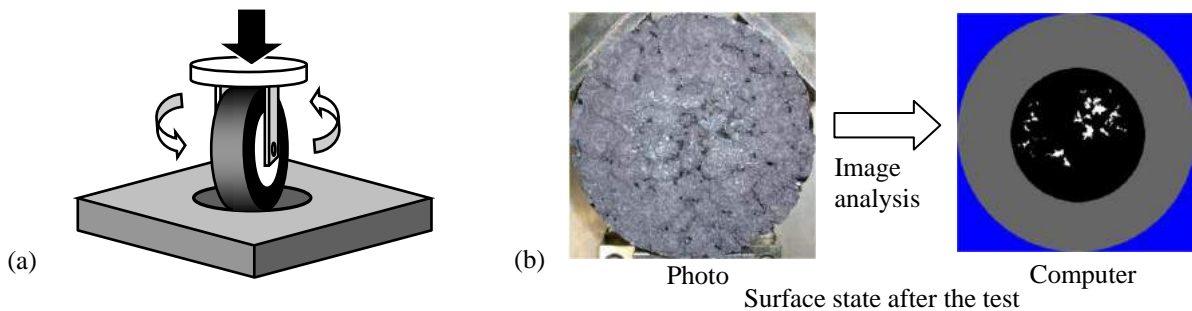


Figure 5. Stripping resistance test (torsion test) (a) Schematic of the equipment, and (b) Image analysis

For torsion test, a test was conducted, following the laboratory stripping-resistance test method for solar heat-blocking pavement material (Minegishi & Ueno 2010). A test load was applied to the specimen by turning the tire left and right. A summary of the test parameters is presented below:

- Loading condition: Turning the tire left and right
- Test temperature: 20°C
- Test load: 686 N
- Number of cycles: 650

For rebelling test, a test was carried out in accordance with the testing procedure for solar heat-blocking pavement materials (Minegishi & Ueno 2014). The test is the same as conventional rebelling test (see Figure 6). However, it was conducting by the following conditions:

- Test temperature: 20°C
- Testing duration: 180 seconds
- Type of testing chain: Cross chain
- Testing speed for wheel: 200 cycles/min.
- Testing speed for specimens: 66 cycles/min.



Figure 6. Stripping resistance test (Rebelling test) (a) Overview, (b) Wheel with chain, (c) Specimen after the test.

After the tests, a digital image of the specimen's surface was taken with a digital camera. Then, the stripping resistance of the solar heat-blocking layer is evaluated using computer image analysis which can record the stripped area of the surface. The result is shown in Table 2. As can be seen the table, compared to the performance criteria set by one organization, the coated layer demonstrates strong adhesion to the existing surface. As a result, a longer life cycle can be expected for solar heat-blocking pavement.

Table 2. Stripping Test Results for solar heat-blocking pavement

Test	Stripping area rate (%)	Performance requirement
Stripping test	6%	Less than 40%
Labeling test	7%	Less than 20%

Note: Stripped area rate = (stripped area/wheel contact area) × 100

3.3 AGGREGATES POP-OUT

The bonding between each aggregate is important factor affecting the durability of pavement. However, as asphalt is thermo-plastic material, it is vital that asphalt pavement is strong enough for torsional shear stress, especially at curved section on the road. Considering the strong coating layer, Solar Heat-blocking Pavement might be a countermeasure for this problem because it makes the surface possible both reduction in surface temperature and reinforcement of surface layer. In order to confirm this effect, laboratory torsional tests were conducted, in accordance with Japanese Standard (2008) (see Figure 7).

Figure 8 shows the test result. As can be seen in the Figure, Solar Heat-blocking Pavement is strong enough to torsional shear stress, compared to conventional pavement. Particle loss rate for conventional pavement gradually increased during the test, whereas that of solar heat-blocking pavement was almost zero percent after the test. Therefore, it can be said that solar heat-blocking pavement is highly effective for improvement of surface durability.

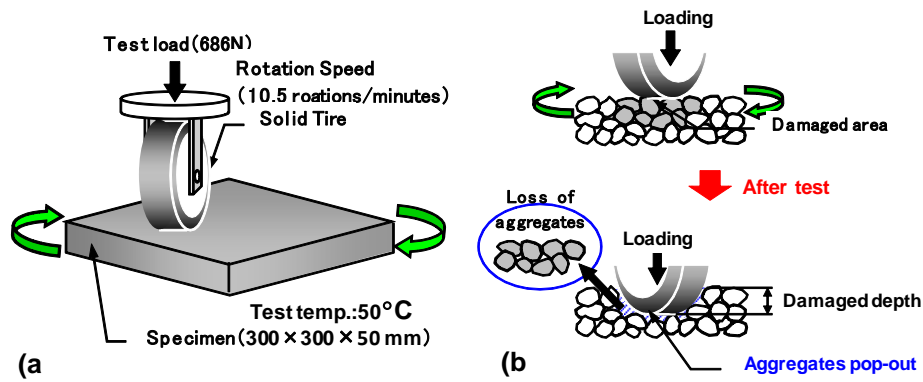


Figure 7. Schematic of torsional test (a) Overview, (b) Loss of aggregates

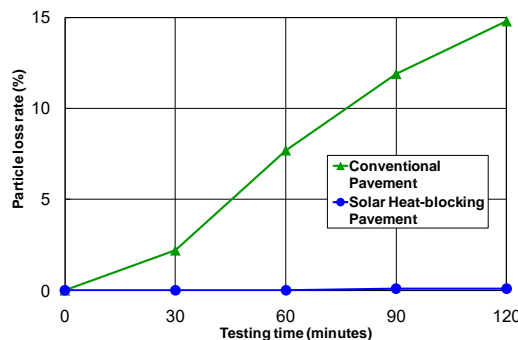


Figure 8. Torsional test results

4 EFFECT OF TEMPERATURE REDUCTION

In order to examine the effect of temperature reduction, conventional pavement (i.e. dense graded asphalt pavement) and solar heat-blocking pavement were compared in the field. The traffic classification of the field is more than 250 and less than 1000 vehicles/day for heavy traffic. Figure 9 shows the temperature of the two surfaces. It can be seen that the maximum surface temperature of the solar heat-blocking pavement was approximately 42°C, whilst that of conventional pavement was around 58°C; the difference between the two surfaces was about 16°C. Therefore, the result clearly shows the advantage of the solar heat-blocking pavement in reducing the surface temperature in summer.

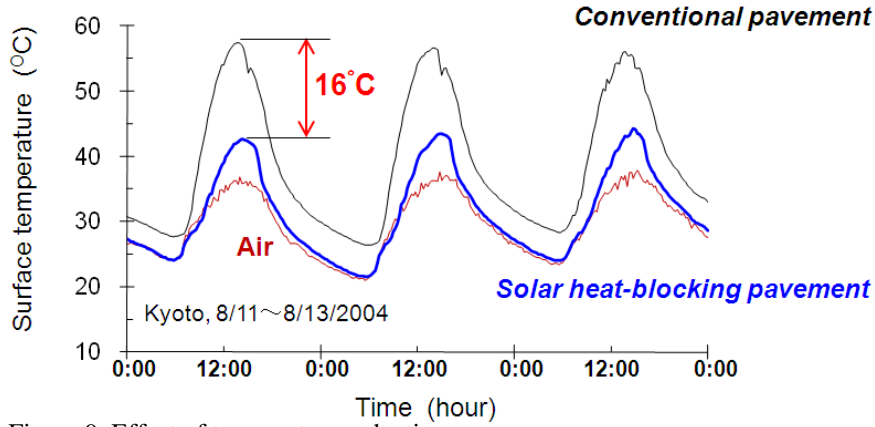


Figure 9. Effect of temperature reduction

Figure 10 compares the surface temperatures of both solar heat-blocking pavement and dense graded pavement through a thermographic image at a site. As can be seen from the thermographic image, the surface temperature of the solar heat-blocking pavement was 35°C, whilst that of dense graded pavement was 48.3°C. Therefore, it is immediately evident that solar reflective coating can effectively reduce the surface temperature.

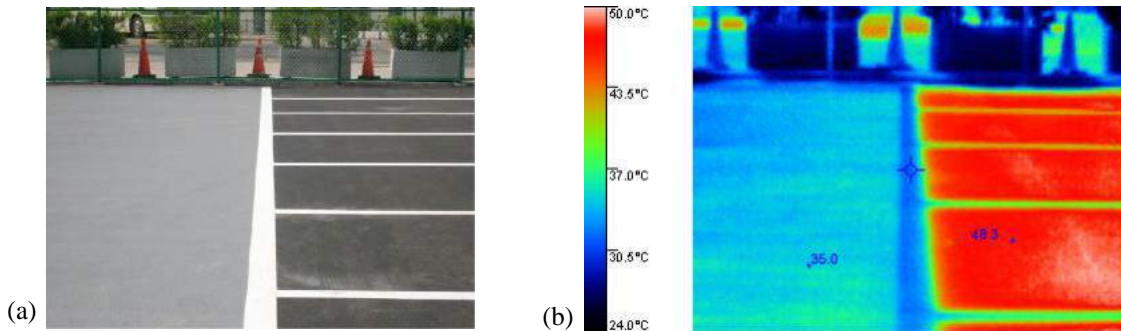


Figure 10. Reduction in surface temperature
 (a) Photographic image, and (b) Thermographic image
 Left: Solar Heat-blocking Pavement, Right: Dense graded pavement

5 DEVELOPMENT OF REPAIR KIT

5.1 CONCEPT

The area of Solar Heat-blocking pavement has reached to 1700,000 m². Some of the construction sites have experienced more than ten years in service, whereas other sites have had buried piping work immediately after construction. In such case, larger machined unit which is used for large scale application was also taken to the repair site even small area (See Figure 11).



Figure 11. Small scale repair and conventional application train (a) repair area, (b) application machine

In order to avoid such inconvenience, the development of the repair kit for solar heat-blocking pavement was strongly demanded to meet the public demand. In particular, the repair kit for solar heat-blocking pavement was developed to meet the following demands: good workability and easy to handling. Figure 12 shows the repair kit developed. As can be seen in the Figure, the kit contains the following materials:

- Softener and hardener for the paint;
- Non-skid sands; and
- Handy roller for application

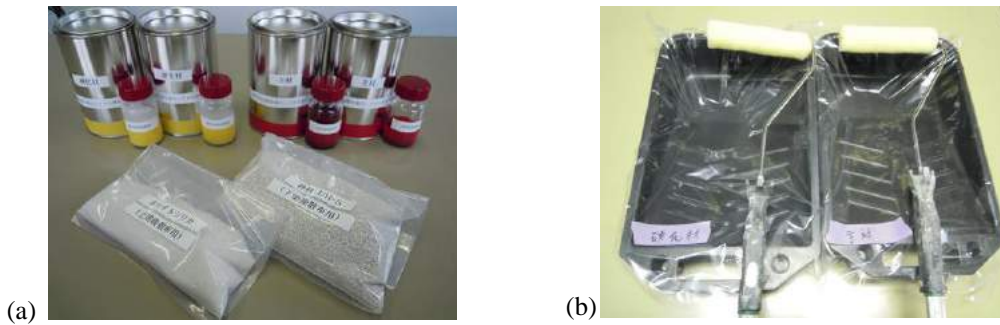


Figure 12. Repair kit for solar heat-blocking pavement (a) Softener, hardener and non-skid sands, (b) Handy roller.

5.2 APPLICATION

Solar heat-blocking pavements can be constructed by applying the developed solar reflective pigment to existing surfaces. The repair kit for solar heat-blocking materials uses MMA based resin. In general, MMA-type material is suitable for heavy traffic roads as quick curing and strong adhesion to the surface are required. The application is conducted by the following procedure:

- Clean the existing surface,
- Apply hardener as 1st layer, then Apply softener as 2nd layer,
- Spray non-skids,
- Apply hardener as 3rd layer, then Apply softener as 4th layer and
- Spray non-skids.

The kit can be applied to the repair area of 1.0 m². The coating work is conducted by hand. The kit and application work are shown in Figure 13.



Figure 13. Application of the repair kit (a) Application of 1st layer, (b) Application of 2nd layer, (c) spray non-skid sand.

5.3 DURABILITY

The repair kit is used to the solar heat-blocking surface which is tore off in service. Therefore, it should be durable enough to snow removal action as well as traffic loading. In order to confirm the durability for these actions, laboratory stripping and raveling tests. The stripping area was evaluated after the tests. The results is shown in Table 3.

Table 3. Stripping area results for the repaired solar heat-blocking surface

Test	Stripping area rate (%)	Performance requirement
Torsion test	5%	Less than 40%
Labeling test	1%	Less than 20%

From the results, it can be said that the surface coated by the repair kit is strong enough to traffic loadings. Despite the fact that the coating was conducted by dividing the surface layer into four layers, the coated layer has the same performance as conventional product. This means that hardener and softener reacts well to make the coating layer.

5.4 ODOR

Solar reflective material contains Methyl Methacrylate (MMA) resin. If applying solar heat-blocking material in urban areas, there are some concerns that the smell of the material may affect people living nearby. Therefore, the emission of odors should be minimized as much as possible during construction.

In such case, a less odorous type of material (i.e. odor-reduced resin) can be selected as the solar heat-blocking material rather than conventional material. In order to evaluate this effect before construction, odor testing is conducted in accordance with laboratory odor measurement method (Minegishi et al. 2010). The test equipment is shown in Figure 14.

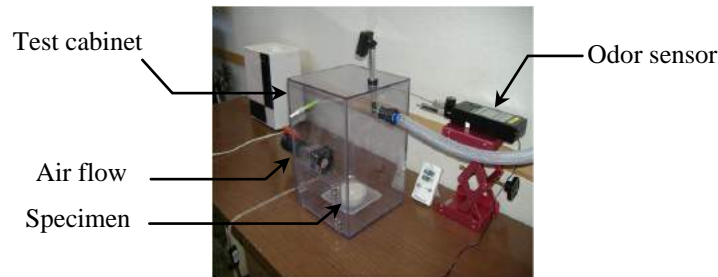


Figure 14. Odor testing equipment (Minegishi et al. 2010)

As can be seen in the above photo, solar heat-blocking material is set in the cabinet; the odor of the materials is measured using an odor sensor. The maximum value of the odor is recorded during the measurement. The detailed test conditions are shown below:

- Air flow: 0.14 L/s (wind velocity: 0.2 m/s)
- Test duration: 20 minutes
- Measurement value: Odor level
- Test temperature and humidity: 20°C, 50%
- Specimen mass: 1.0 g

Table 4 shows the measurement results. These results clearly show the difference between conventional and odor-reduced materials. The odor level for the conventional material is about four times that of the odor-reduced type. Also, the odor-reduced material meets the requirements set by the organization. In addition, the same surface performance as conventional material has been confirmed for the odor-reduced material. Therefore, it is possible to apply the repair kit for solar heat-blocking pavement even in urban residential areas.

Table 4. Odor level of repair kit for solar heat-blocking material

Types of Solar Heat-blocking Pavement	Odor level	Performance requirement
Repair kit for Solar heat-blocking pavement	203	Less than 300
Conventional solar heat-blocking material	889	

6 CONCLUSIONS

This paper presents the development and application of Solar Heat-blocking Pavement. Based on the field results, the following conclusions can be drawn:

- With regard to improvement of durability, the solar heat-blocking surface contributes to improving the surface layer since the technology makes the surface possible both reduction in surface temperature and reinforcement of surface layer.
- With respect to the performance of solar heat-blocking pavement, the surface temperature is reduced by approximately 16°C compared to conventional dense-graded asphalt paving due to the curtailment of solar radiation.
- In terms of small scale repair, the performance of the repair kit developed in this study is almost identical with conventional solar heat-blocking pavement.

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THE EFFECTIVE USE ON EXPRESSWAY TO REDUCE TRAFFIC ACCIDENTS IN TOKYO METROPOLITAN AREA

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1. INTRODUCTION

The Ken-o Expressway, Tokyo's ring road, was opened to the public on June 2014. Since the expressway connects the heavy-traffic intercity expressways, it has been regularly suffering from severe traffic congestion arising from traffic concentration. Once congestion occurs, the traffic accident risk increases at the tail of traffic jams, leading to serious traffic accidents.

Under the circumstances, it was concerned about the increase in traffic congestion and traffic accident due to expressway network expansion on October 2015.

Since the widening of expressway was impossible in the urban area, it was necessary to select the most effective measure from some countermeasures in the limited ROW.

Therefore, we carefully analyzed the ETC probe data to identify the factors causing the congestion in the first step, we selected and carried out the most effective congestion solution.

In this paper, we would like to describe the congestion solution.

2. OVERVIEW OF KEN-O EXPRESSWAY

The Ken-O Expressway is one of ring roads in Tokyo Metropolitan area, which is planned about 40-60 km away from central Tokyo.(Figure 1) This Expressway is expected to make traffic smooth, improve the roadside environment, encourage closer connections among cities, and to become an alternate route at the time of disaster. As of 2016, about 80% of the expressway has been completed, the traffic volume has been increasing year by year. In this section, a daily traffic is 30,000 vehicles per day on one lane.

Due to this heavy-traffic, it has been regularly suffering from severe traffic congestion at the junction of two Expressways.

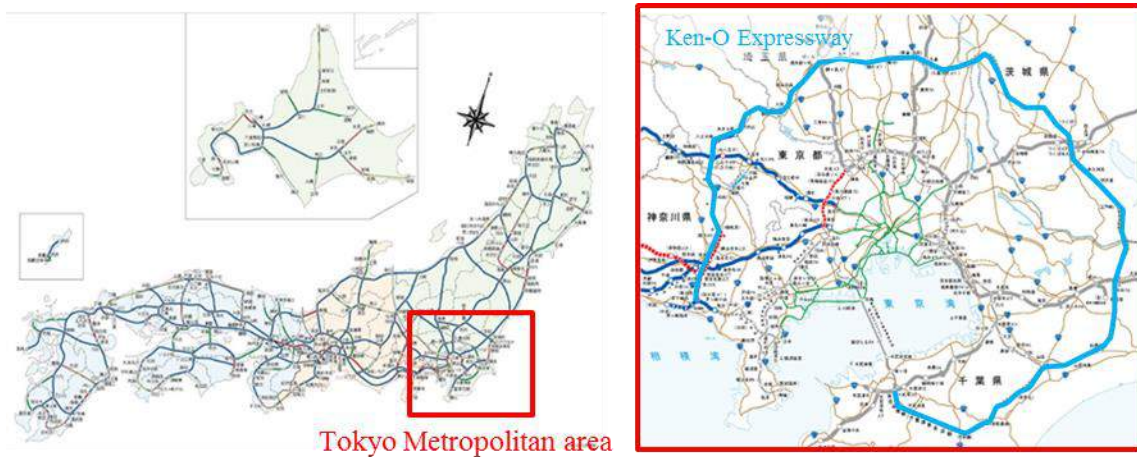


Figure 1. Location map of The Ken-O Expressway

3. MATERIALS (FACTOR OF CONGESTION)

The cause of traffic congestion at the Junction, mainly comes from over capacity arising from traffic concentration. According to the analysis of the ETC probe data, it was confirmed that the following two factors contribute to traffic congestion additionally.

Factor-1 Decline in traffic speed at the junction and small in traffic capacity

At the time of traffic concentration, it has been suffering from severe traffic congestion because of the decline in speed at the Junction as well as small in the traffic capacity. (Figure 2.)

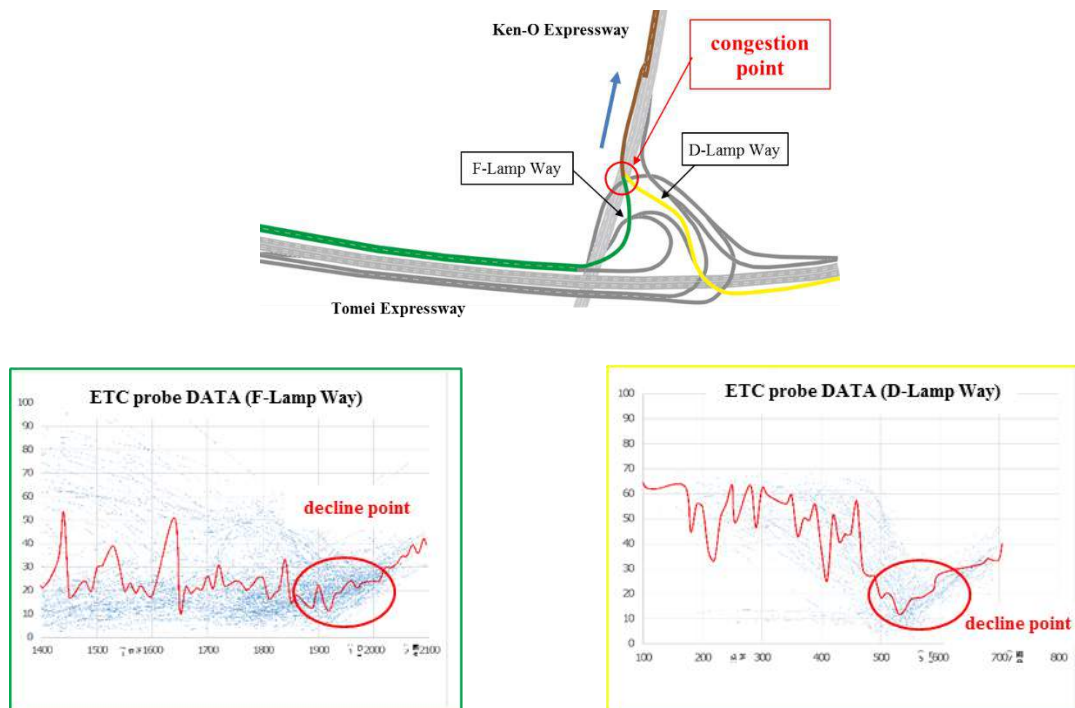


Figure 2. Decline in traffic speed at the Junction

Factor-2 Decline in traffic speed on the 6% uphill ramp

6% uphill ramp has contributed to the decline in the traffic speed of large vehicles at the Junction.

(Figure 3)

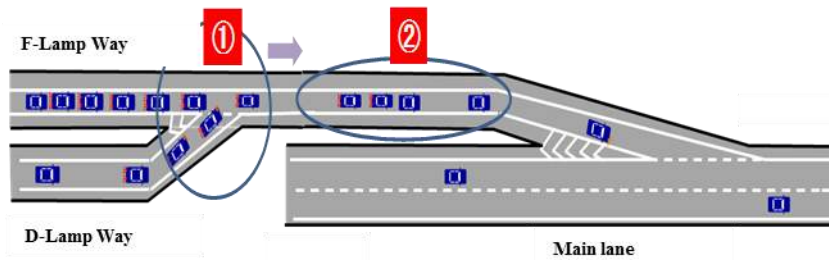


Figure 3. Decline in speed on 6% uphill ramp

4. METHODS (CONGESTION SOLUTION)

It was confirmed that the best solution is to prepare for the split lane by analyzing the factor of traffic congestion. By introducing this solution, it was considered that traffic capacity will increase and recovery effects of reduced speed are expected.

But, in this case, the targeted section is located on bridges in urban area, it was necessary to take the solution in a limited ROW. Therefore, we have selected the solution of “temporal split lanes” without losing safety and required performance.

Moreover, we also improved how to merge the ramp with main lanes while keeping smooth and safety roadways.

(1) Width of main lane and shoulder

In order to ensure the two lanes in the one-lane-operated ramp, we narrowed the main lane width from 3.50m to 3.25m, the right shoulder width from 1.00m to 0.38, and the left shoulder width from 2.50m to 0.38m respectively. (Figure 4)

Even though the cross section in this portion does not satisfy the law with respect to the road structure, the road structure, two-lane operation, was adopted because this would be the temporal operation until the completion of expressway network and ensures the width required for snow removal and construction works. We prioritized reducing traffic congestion and accidents in this plan.

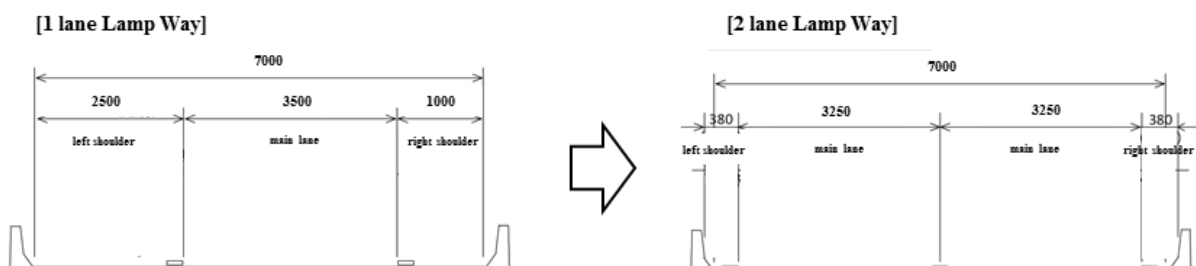


Figure 4. Width of main lane and shoulder

(2) The modification of merging section with the main lanes

Even if two-lane operation is adopted, it was expected that another traffic congestion occurs at the merging section with the main lane. Therefore, we decided to improve the structure of not only the merging section but also the main lanes. Of three measures proposed, we decided to proceed with the plan 3 described below from the comprehensive point of view including the reduction of traffic congestion. (Figure 5)

CASE-1. 2 main lane & 2 junction lane

This road structure cannot ensure the required length of the lane for acceleration.

CASE-2. 1 main lane (right side) & 2 junction lane

There is a possibility to occur another traffic congestion at the merging section of main and ramp lanes, because of the increase in the length of merging section.

CASE-3 1 MAIN LANE (LEFT SIDE) & 2 JUNCTION LANE

The probability that another traffic congestion occurs in the merging section is low because of the increase in the length of merging section.

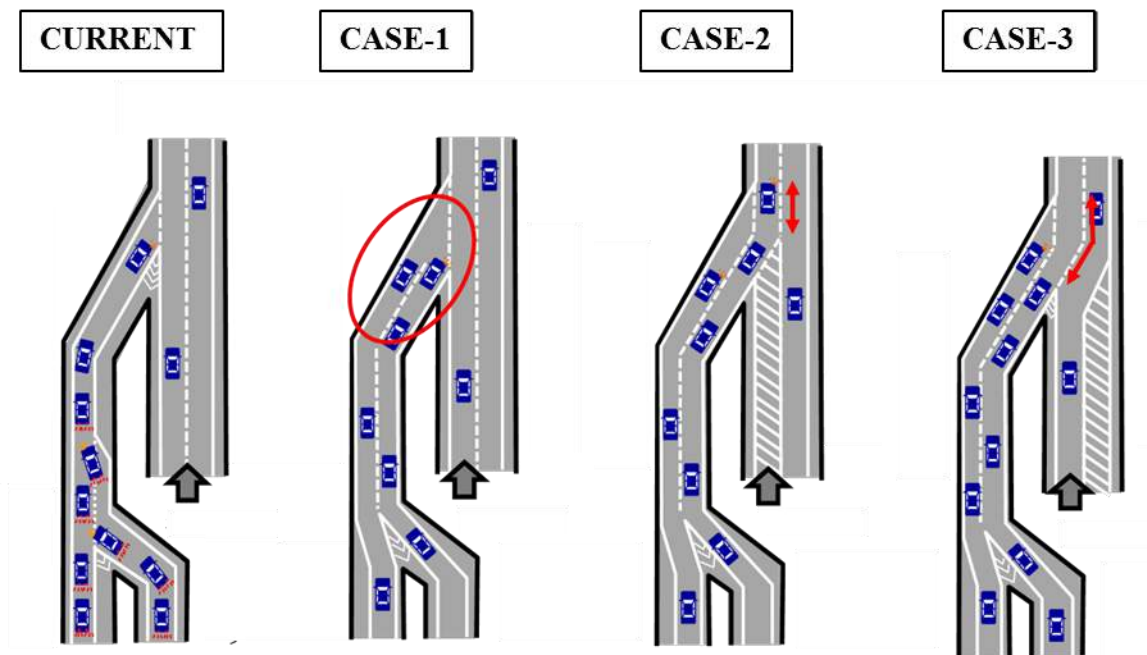


Figure 5. Merging section with the main lanes

(3) Traffic forecast

If the traffic capacity in one lane would be set to 1,530 vehicles by hour, demand traffic volume would be estimated to 1,625 vehicles by hour after the measures of traffic congestion by using both demand time factor on no-congestion days and real traffic volume on congestion days.

Therefore, when we introduce two lanes in the ramp, if 95 vehicles or more by hour shift up to the merging section with the main lane, the traffic congestion is not considered to occur.

5. IMPLEMENTATION OF SOLUTION

It was necessary to review the speed limit from the current 50km/h to 40 km/h, because the solution of traffic congestion is provisional. Therefore, we decided to implement various accident measures according to the traffic congestion solutions.

The major accident measures are as follows.

1. Improvement of the lane mark
2. Installation of additional warning sign

6. RESULTS

Even though the average traffic volume in the section increased by about 4,000 vehicles by day, no traffic congestion have occurred at all. The result shows that the effect of improvement is sufficient.

(1) Comparison of traffic volume and congestion

Even though the average traffic volume in the section increased by about 4,000 vehicles by day, no traffic congestion have occurred at all. The result shows that the effect of improvement is sufficient.

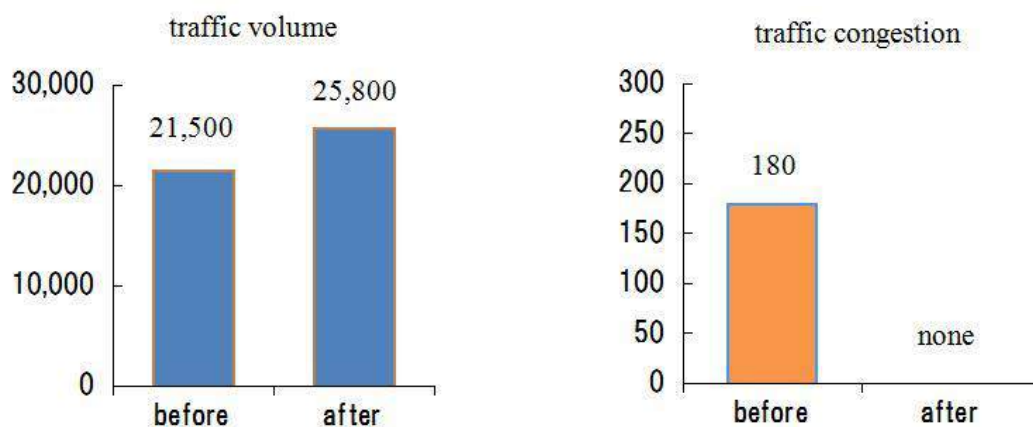


Figure 6 Comparison of traffic volume and congestion

(2) COMPARISON OF TRAFFIC CAPACITY

According to the verification on the traffic capacity in the section, the traffic capacity at the time of traffic congestion has dramatically increased from 1,530 vehicles by day to 2,000 vehicles by day.

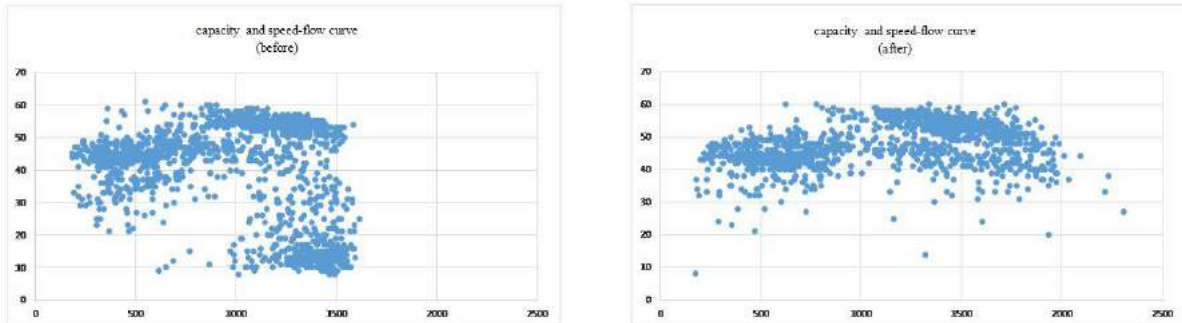


Figure 7. Traffic capacity of lanes

(3) TRAFFIC VOLUME OF CHANGING LANE

The comparison of traffic distribution at the two sections in the two-lane ramp shows effective lane change, clearly indicating the increase in traffic capacity.

In addition, it was observed that 15% of the traffic is shifted in the merging section. The result is beyond our estimations. It is inferred that this contributes to the resolution of traffic congestion.

<section-1 → section-2>

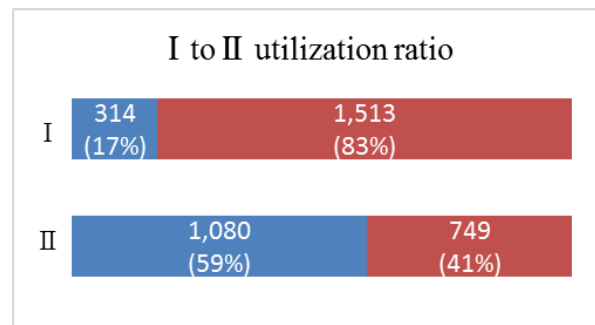
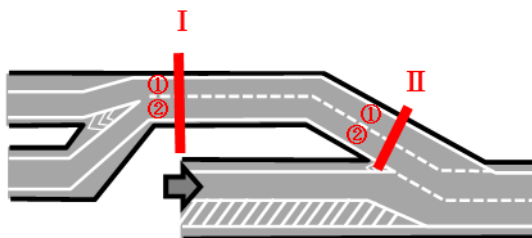


Figure 8. utilization ratio (two-lane ramp)

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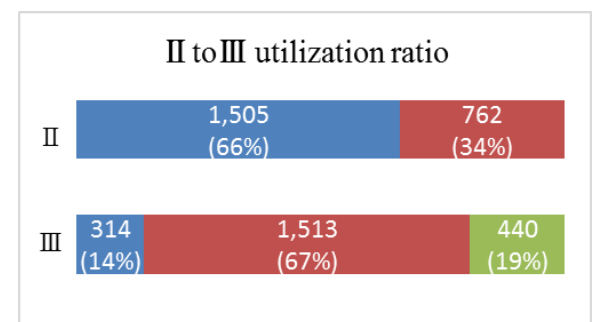
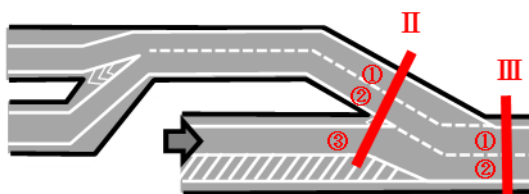


Figure 9. utilization ratio (merging section)

7. REPTATION OF THE USER

We carried out user survey to understand the effect of the traffic congestion solutions. The result shows the solutions provides the improvement of traffic congestion in not only expressway but also local roads. We are also receiving a favorable opinion from the local residents.

8. CONCLUSIONS

These solutions can be useful examples, in which we analyzed and identified the causes of the decline in traffic speed, and implemented the effective and temporal solutions in the maximum use of the current road structures under the incomplete expressway network.

This case has provided big effects by minimum time and low cost. No traffic congestion has occurred in the targeted section as of June 2016. A similar type of traffic congestion at another ramp near the section is observed. Another solution is needed while considering a different gradient and traffic flow. We hope that this solution will become one of the guidelines for similar types of traffic congestion solution while reflecting the measures against current issues and their effects.

“URBAN TRANSPORT SOLUTIONS/ BRT VS TRAM”

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1. INTRODUCTION

Urban transport projects are much more than transport systems. They are not only a public means of mobility; its urban and social relevance can involve profound changes in the cities in which it is located.

Public transport is also a social tool to promote equity, providing affordable transportation to access services and opportunities otherwise not reachable for low-income population.

In addition, they will be under operation during decades. Any decision should be taken by a huge analysis to solve mobility problems in cities, processing capacity of historic urban zones, the possibility of structuring new urban development and boost the life of cities and considering the benefits and the cost during the whole life of the project.

Public transport generates a series of direct impacts for passengers and cities to take into account:

- Passenger's Benefits:
 - Accessibility: Time travel savings
 - Economical: Vehicle cost savings (parking, fuel and maintenance)
 - Health: Increases people physical activity, Reduces people stress, Reduces number of traffic accidents, Reduces costs with public health care
- City's Physical Benefits of reducing car traffic:
 - Reduces energy consumption and CO2 emission
 - Facilitates better walking and cycling conditions
 - Improves urban life quality: enhance public spaces, less sound pollution

The present paper analyzes those aspects that should come to decide best way of transport in a city: BRT and TRAMS by a decision maker in each city. It presented the objectives, methodology, similarities and differences between BRT and TRAMS, in order to find best option to each case.

2. OBJECTIVES AND GOALS FOR THE IMPLEMENTATION OF A BRT/TRAM IN A CITY

The objectives and main challenges in the implementation of a new transport system in a city should be:

- **CONNECTIVITY AND INTERMODALITY:** Enhance land public transport connectivity across urban conurbations and access, especially where the population density is higher, in a high quality and integrated service transport network includes, not only the public transport, but also “last/first mile transport”. The analysis of private cars, taxis, bicycles, parking lots, etc. it is very important for an efficient transport.
- **ACCESSIBILITY:** Ensure affordable and accessible public transport services by enhancing industry structure.
- **URBAN INTEGRATION IN A SAFETY WAY:** Implementation of the urban transport solution has social and urban relevance and can involve important changes to the city in which they are located. Urban integration and alignment design will be carry out to exacting international standards, considering all national and international safety and security regulations and in full coordination and integration with all other modes of transport, existing and planned for the future.
- **A SUSTAINABLE INFRASTRUCTURE (TRANSPORTATION COMPONENT)** Enhancing safety levels of public transport. The strategies must focus on reducing accidents, incidents and improve passenger's confidence in the efficiency of the system.

- **QUALITY OF LIFE:** Reduce congestion, noise and vibration, pollution, and increase quality of life for the city, benefits for passengers and for citizens.

To establish an efficient and attractive way of transport for the citizens and provide the basic network for the planned future development we need to think *main characteristics* that differentiate BRT or TRAM services from other local bus services including:

- higher service frequency along the corridors
- higher operating speeds
- limited key stops along the BRT corridors
- transit priority measures – queue jumps, traffic signal priority, segregated lanes
- distinctly branded high capacity vehicles
- enhanced passenger stations
- enhanced/integrated local feeder service

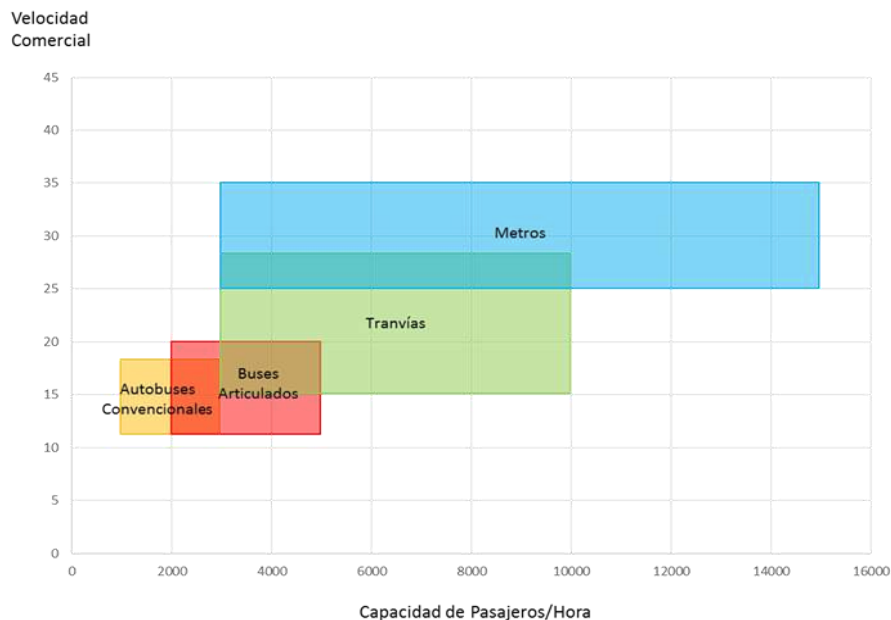
To reach this functional requirements decisions on design of the network, in the definition of the vehicles, alignment, station, and user facilities will be very important and constitute the fundament for the success of the BRT / TRAMWAY and its acceptance by the citizens, users and non-users.

3. MASS TRANSPORT MODES: BRT'S/ TRAMWAYS

The new trams and BRTS, through the application of technological advances, have become a new form of public transport with a high level of performance for its accessibility, low noise, speed, regularity, comfort and ecology.

Generally the definition of modes are related to capacity and demand, but other issues has consequences in the definition of the right mode to implement in a city.

Below is a typical definition of different transport mode, starting in less capacity and growing up.



Características	Tramways	BRTs	Conventional Buses
Capacity Passengers/ hour	3.000-10.000	2.000 - 5.000	< 3.000
Commercial speed (km/h)	20-28	12-20	12-18
Accesibility	Low floor	Low floor	None
Investment average Million Euros / km/ Km (*)	8-30	6-16	4-8
Average between stops(meters)	400-800	400-800	400
Length of life of vehicles	30-35 years	10-12 years	10 years

Table 1. Comparison between transport modes (Speed (km/h) versus Capacity (passenger/hour))

(*) This average varies depending on country, project and length of the line.

The capacity increase put the TRAMWAYS as a mode of efficient transport way with a significant reduction in operational costs compared to BRTs or conventional buses.

The implementation of a modern tramway system in the cities had to face the challenges of mitigate the traffic congestion, noise reduction, CO2 emissions reduction, and to contribute to the development of cities and the recovery of pedestrian spaces (dusty historical buildings by pollution from cars, filled squares cars and parked cars were usual in several cities.)

The design of modern tramways project should be focus on aspects as: safety and environmental aspects; Mitigation measures; noise and vibration (track and rolling stock design, Emissions); Conditions of accessibility at station areas and rolling stock; Safety; urbanization, signaling and operation; Compatibility of tram with priority traffic lighting; Tram-compatibility with other modes of transportation during the construction and design phases (temporary site, pedestrian and bicycle path alternatives, traffic detour, bus stops); Configuration of Power-supply system (power substations, redundant configuration, rush; special feeding system for areas without power lines; fail-safe systems, insulation and protection to third parties (land network); Travel information systems and ticketing.

BRT is also a high capacity and an efficient transport way if the demand is in the range described. Same than tramways industry has developed several technology that can offer great capacity, comfort and environmental solutions.

We can find in the market different BRT solution based on Diesel, CNG and electrical vehicles.

Taking the consideration of electrical vehicles we could reach the goal to reduce and minimize pollution and CO2 in the city.



Electrical - Quito

Diesel - Bogota

CNG - Jakarta

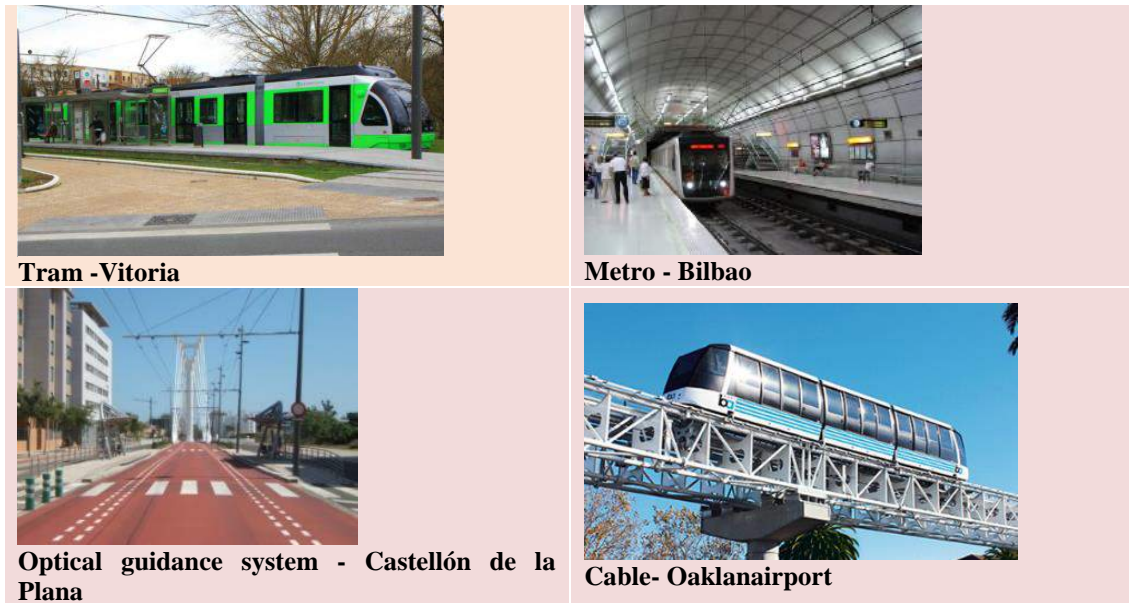


Figure 1. Different types of mass transit modes

4. BRTs versus TRAMWAYS

To compare each system we will divide the analysis in eight topics that need to be considered in the implementation of the line:

- Vehicles
- Permanent Way
- Stops / Stations
- Power supply
- Workshops and Depots
- Operation and Maintenance costs
- Investment cost

4.1. Vehicles

It is difficult to compare vehicles because of the very different natures of light rail and BRTs, but attached are main differences:

- BRTs is subjected to road vehicle oriented legislation and standards, contrary to TRAMWAY which is submitted to a legislation and standards identical or that derive from railway standards.
- Usually much higher requirements for trams in regards with comfort, safety, reliability, availability.... This concerns for example the great surface of glass, the large driver's cab and the equipment redundancy of trams
- The materials and technologies employed in trams have a much longer life duration than for BRTs (approximately three times)
- Bidirectional vehicles in tramways nor in BRTs,
- More capacity in tramways than BRTs A common basis for comparative studies is often the number of places available for passengers. In this paper, it is included the number of square meters available for passengers, because the number of places available is depending on many factors such as:
 - Percentage of seats (usually between 15 to 40%)
 - Number of square meters per seats (usually 1,4m² per seats in trams but only 1,15m² per seats in busses)
 - density of standing passengers (Normal load is defined as 6 pax/ m² in busses and only 4 pax/m² in trams)
 - exchange rate (ratio of door openings over vehicle length)

Type of vehicle	Dimensions	Square meters available for passengers
Long Trams	44 x 2,4 m	90
Medium Tram	32 x 2,4 m	64
Long BRTs	18 x 2,5 m	38
Small BRTs	12 x 2,5 m	24

Table 2. Type of Vehicles and dimensions Source: Libertin

- Energy demand is clear difference between TRAMS and BRT (if we include all types not only electrical ones) The tram is clearly much better than busses in regards with sustainable development, notably because:
 - it uses electricity, which reduces pollution and all its consequences
 - it is renewed once every 30-40 years instead of once every 15 years
 - the energy consumption is lower. The small running resistance of the wheel-rail contact and the regenerative braking. Even if heavier, the tram consumption is four times less kWh consuming than a bus per passenger (with occupancy rates of 50%) and 24 times less per passenger than a car (with an average occupancy rate of 1,2 people per car).

4.2. Permanent way

BRTs and TRAMWAYS will be implemented in a segregated lane with some areas where shared traffic could appear, but in both cases some investment is required if we look for a high capacity system.

Main differences will be in alignment requirements: Tramways will go in a horizontal minimum radius over 20 -25m (1 m or 1,435 m gauge) and BRTS could reach 12 m radii.

About maximum slope iron wheels limits to 8,5 % for tramways meantime BRT could reach 12%.

BRT	TRAMWAY
Not necessary special infrastructure. Typical section of a carriage way	Necessary a special infrastructure: track slab, rail fixation or embedded system
If there is not electrical BRT, less special ducts (multitubular) are needed.	Multitubular ducts required for electrical supply and drainage works of the rails
Not possible green surface	Possible green surface
Relocation of existing utilities could be not required thinking that BRT could change the alignment in case of a disruption of service	Relocation of utilities is considered to avoid any disruption during operation because of any failure in utilities

4.3. Stops /Stations

The design of the stops or station is a tailor-made solution for the city, considering climatic and operational conditions. Accessibility is always priority in the design incorporating latest technology in travel information systems and ticketing, as we consider the correct contact interface between the user and the urban light rail solution to be essential for the system to be effective, safe and attractive.

Many things should be decided:

- Stops can be close (platform doors) or open
- Central or lateral platform.
- Equipment: benches, shelters, passenger information system
- Fare collection system at stop or on board

Some differences comes from vehicle differences:

- Elevated platform from the pedestrian way to fit with vehicles floor (accessibility)
- Length of platform: BRT needs shorter platform than tramway because length of vehicles.
- Tramways run in both directions and have doors at both sides, BRT must be manufactured with double doors if needed.



Figure 2. Type of stations/stops BRTs

4.4. Power supply

In BRT vehicles energy demand established huge differences the type of vehicles. Here we compare electrical BRT. Buses can based the energy on batteries, rapid charge on stops or a overhead contact line during the line.

Tramways based their energy in overhead contact line, but also newest technology were developed free catenary systems based on embebed and protected third rail, ultra capacitors, batteries and recharging points at stops. In this last case tramways has a rapid charge (less than 30 seconds) in comparison with buses (4-6 minutes).

4.5. Signaling / Traffic priority

A goal for a high capacity and efficient mode of transport is the travel time and commercial speed. To reach this goal traffic priority is required in BRTs and TRAMWAYS.

A special signaling and on board systems to be installed and need to be connected with CMC and traffic control Centre.

Tramways will required additional signaling and control for the switches devices and maneuvers zones.

4.6. Workshops and Depots

Functional definition of workshops and depots are engaged to operational drive.

BRT is able to run outside the permanent way. Tramway can not run without infrastructure. BRT is able to avoid obstacles in the infrastructure and to be implemented in a less radius than a tramway.

Tramways needs rails and maintenance pits at workshop

BRT is similar to a typical bus and does not need same special equipment as tramways

BRT needs less space at depot than tramway (for same fleet size)

BRT workshops and depots are cheaper than Tramways ones.

4.7. Investment costs

Investment is generally divided in costs of:

- Infrastructure (including road works, utilities relocation, tracks, stations, signaling, communications, ticketing and information to passengers, energy)
- Vehicles
- Depot and maintenance facility

In infrastructure investment there are many differences between each city and lines. Infrastructure budget depends on length of the line, number of stops and additional works included in the investment (utilities relocation and urban roads can be third part of the infrastructure budget). Generally a TRAM line is more expensive than a BRT but length of life of components are longer. For these reason a total analysis should be done to compare BRT and TRAMWAY projects.

For example for a BRT line in London for two lines of 12,5k m and 14 km had an investment of 381,8 million dollars, this means \$14,4 million/km.

Capital Investment – BRT Strategy	
Description	Amount (millions)
Road work (including passenger amenities)	\$ 300.0
Fleet	51.8
Maintenance and storage facility	25.0
Downtown terminal	5.0
	\$ 381.8

BRT line for TEOR line in Rouen buget split was 29,5 million Euros for 66 vehicles and 135 million Euros for infrastructure of 29,8 km length , that means 6,4 million Euros (\$ 7,5 million/km).

TRAMWAYS are more expensive in chapters of vehicles, depot, energy and tracks as initial investment, but reduced budget for maintenance during the length of life of the vehicle (2- 3 times a BRT life)

Type of vehicle	Dimensions	Average price Euros	Average price Euros per square metre available for passengers
Long Trams	44 x 2,4 m	2800000	31000
Medium Tram	32 x 2,4 m	2200000	24000
Long BRTs	18 x 2,5 m	450000	12000
Small BRTs	12 x 2,5 m	300000	13000

Table 3. Type of Vehicles and prices (Source: Libertin)

The higher ratio for trams results from:

- the higher level of equipment provided (intercom, dot matrix display ...)
- very small series in comparison with busses

4.8. Operation and maintenance costs

To analyze the costs attributable to the operation and maintenance, emphasis is on those that are most characteristic of this service, such as:

- Operational personal cost (drivers, managers, PMC controllers, etc. ...)
- Cost maintenance
- Cost of energy to traction

Trams however provide economy in operation:

- lower energy consumption – 4 times less per square meter available, the price of the “fuel” being approximately 0,1 per kWh in both cases –
- lower driving costs because of the higher vehicle capacity

Choosing best way of transport between BRT and tramway should be based in addition to capacity and demand approach to mean characteristic.

Following figure from London transport Authority showed a curve for London transport systems that comparing operational cost per passenger and km per year for three different transport modes (bus, BRT, and Tramway) we could see that in different ranges of demand each mode is more efficient.

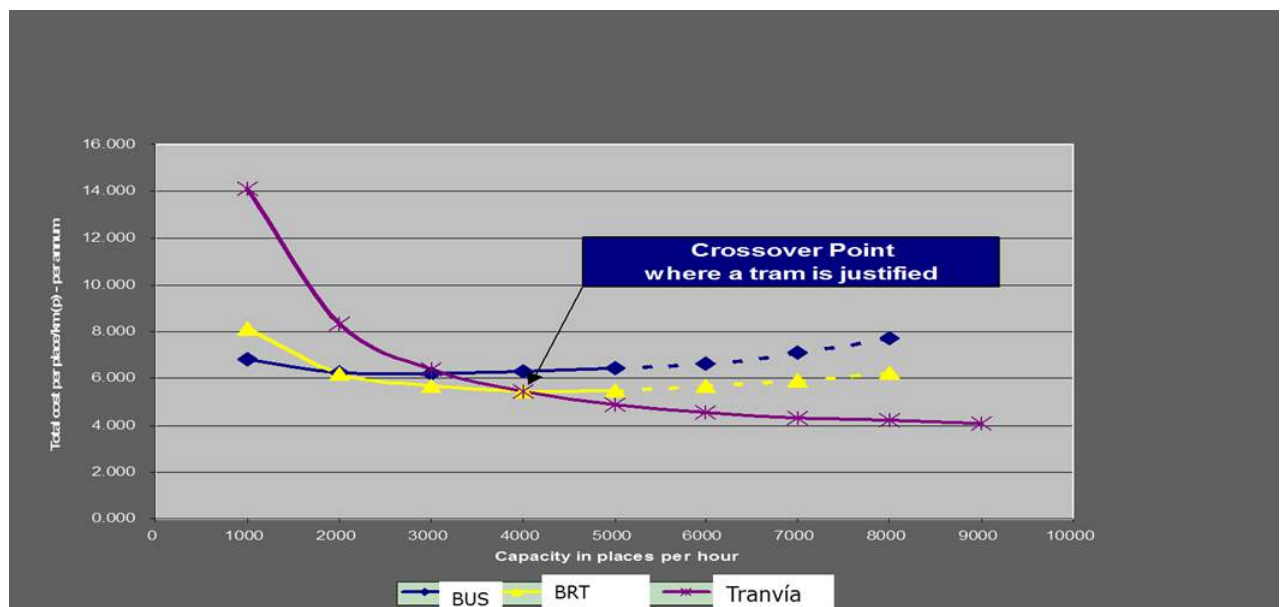


Figure 3. Type of Vehicles and prices (Source: ATM London)

This exercise is made for London city but should be done in each city in planning phases in order to compare and analyze each mode according demand prognosis to choose best mode in each case.

5. CONCLUSIONS

Different characteristics and economical ratios clearly reflect the importance of the different industrial environment of busses and trams. As a consequence, it is recommended to use more systematically the analysis BRT versus Tram comparison to detect areas where technical solutions for BRTs should be transferred to TRAMS (and reciprocally) and areas, where the specification for trams should be relaxed.

One method to define the transport mode should integrate the analysis of growth city, the expansion of the transport system; innovation; life quality, greater interaction using the attractive high design specification service and; sustainability, conscious of the environment and the economy of the project.

Nowadays BRTs and TRAMs reach this topics. In order to find the best solutions taking into account all the goals of the project the strategy for accomplishing this:

- The realization of a comprehensive transport study (Integrated Transport) which analyses existing bus lines and the possible restructuring (location of stops, areas served) to achieve an efficient system compatible with the tram system or a BRT.
- Use of demand analysis software like VISUM; EMME or others to model networks and calculate demand prognosis for each mode and corridor.
- Operational analysis to propose frequencies, fleet size and operational and maintenance cost.
- Preliminary study of the corridor, location of stations, transport technology and fleet size should be done to calculate investment works for each system.
- Financial and economical analysis for length of life of each system BRT and TRAM in order to compare total cost and benefits of each mode to estimate most appropriated.
- Comparison of each mode taking into account main challenges:
 - Urban integration
 - Transport efficiency: operational and maintenance cost analysis during whole life
 - Passengers and non-passengers benefits

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PAPER TITLE	Wheel load running tests on the embedded expansion Joint using high-flexibility stone mastic asphalt		
TRACK	Asset Management		
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KEYWORDS: embedded expansion joint, high-flexibility Stone Mastic Asphalt, wheel load running test

ABSTRACT:

The embedded expansion joint is applied on the pre-stressed Concrete Bridge in urban areas in Japan for noise reduction and run stability. Conventionally, gussasphalt is used for the base course of the embedded expansion joint. However, gussasphalt is constantly damaged under low temperature condition; therefore, improving durability of the base course is highly demanded. Under this situation, it has been studied to develop new embedded expansion joint using high-flexibility stone mastic asphalt, instead of a conventional gussasphalt in Hanshin Expressway in Japan. In this study, wheel load running tests simulating an actual viaduct are conducted, in order to compare the fatigue resistance of three embedded expansion joint types using gussasphalt, using whole and partially high-flexibility stone mastic asphalt. This paper describes the improvement of fatigue resistance of the embedded expansion joint.

Wheel load running tests on an embedded expansion joint using high-flexibility stone mastic asphalt

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1 INTRODUCTION

Generally, to improve trafficability in Japan, embedded expansion joints are used for replacing the old expansion joints of prestressed concrete bridges, with a span length below 25 m and joint movement below 20 mm. However, these embedded expansion joints have a durability problem because of damage occurring over a few years after their repair. The primary source of the defects in such expansion joints is considered to be the gussasphalt used in the base course. Therefore, high-flexibility stone mastic asphalt (SMA) is adopted as the base course in new designs.

In this study, simulation of wheel load running tests on an actual viaduct was performed to compare the fatigue resistance of three types of embedded expansion joints: (i) joints with base course made of gussasphalt, (ii) joints with base course completely made of high-flexibility SMA, and (iii) joints with base course partially made of high flexibility SMA. This paper describes the improvement in fatigue resistance of these embedded expansion joints.

2 HIGH-FLEXIBILITY STONE MASTIC ASPHALT

High-flexibility SMA uses a special polymer-modified asphalt binder, which is very flexible at low temperatures. According to several previous studies, high-flexibility SMA is superior to gussasphalt in terms of resistance to cracking by repeated wheel load, especially at low temperatures. In addition, using glass-fiber sheets under the base course for reinforcement can enhance the durability of the embedded joints.

Examinations were conducted to compare the high-flexibility SMA and gussasphalt. Bending strength test was performed to characterize asphalt bending stiffness. Bending fatigue test was performed to characterize asphalt bending resistance by repeated load. Penetration with crack test was performed to characterize crack resistance of the base course by repeated wheel load, as shown Figure 1.

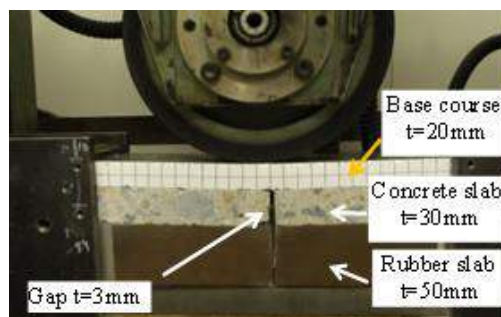


Figure 1. Penetration with crack test by wheel load

The result of the bending strength test is shown in Figure 2(a). SMA has a better performance than gussasphalt with respect to both bending strength and fracture of strain at -10°C . In addition, SMA is more flexible than gussasphalt. The result of the bending fatigue test is shown in Figure 2(b). This test was conducted under different temperatures of 0, 5, and -10°C . The SMA was observed to have a better performance than gussasphalt for every temperature.

Based on the result of the penetration with crack test, compared with gussasphalt, high-flexibility SMA has superior resistance to cracking by the repeated wheel load, especially at low temperatures, as shown Figure 6(a). To further enhance the durability, glass-fiber sheets were used under base course for reinforcement, in which case the resistance to crack is higher than when using no reinforcement, as shown Figure 3(b).

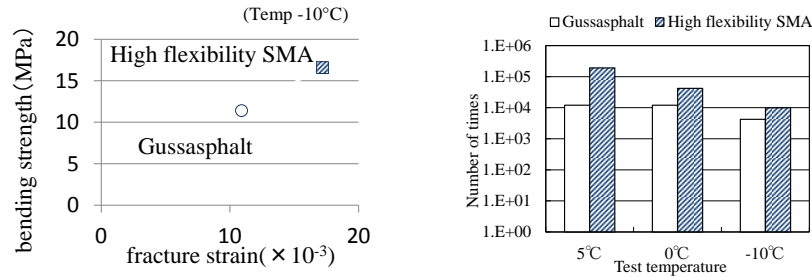


Figure 2. Test results: a) bending strength and fracture of strain by bending strength test at -10°C , b) Bended fatigue test.

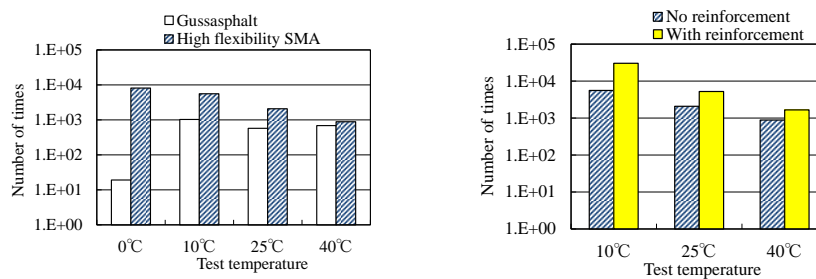


Figure 3. Test results: a) Penetration with crack test, b) compared with the effect of reinforcing material

3 SPECIMENS FOR WHEEL LOAD RUNNING TEST

The wheel load running test was conducted on the structures of three embedded expansion joints. One of these three structures is Type A conventional embedded expansion joint. A conventional embedded joint, shown in Figure 4, uses gussasphalt and an expanded metal, as the base course. A sliding sheet (PM-N sheet) is placed between gussasphalt and the concrete slab for reducing frictional resistance. Gussasphalt was cast via human construction because of small-scale site work.

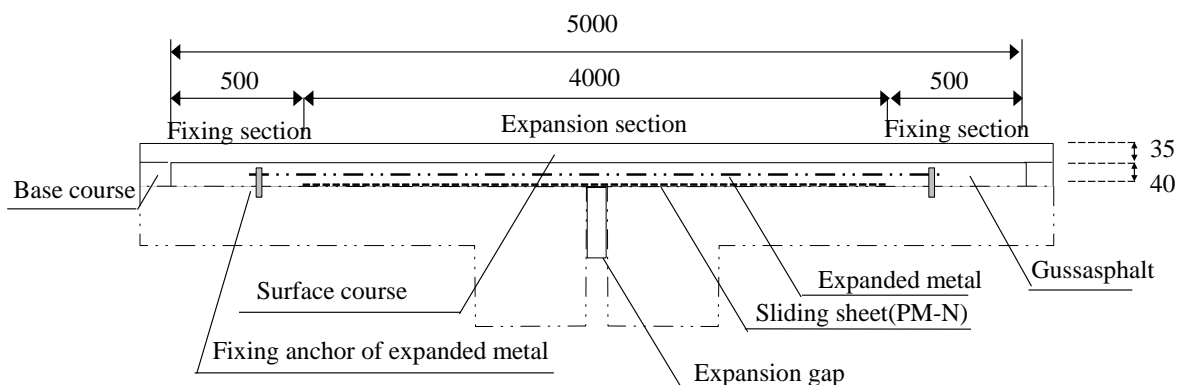


Figure 4. Type A conventional embedded joint.

The other two structures are new structures that using high-flexibility SMA in the base course. The SMA of Type B structure, shown in Figure 5, is cast via machine construction for quality control. The SMA of Type C structure, shown in Figure 6, is cast via human construction and is used in maintenance work because the traffic regulation time is short, and the construction spot is small.

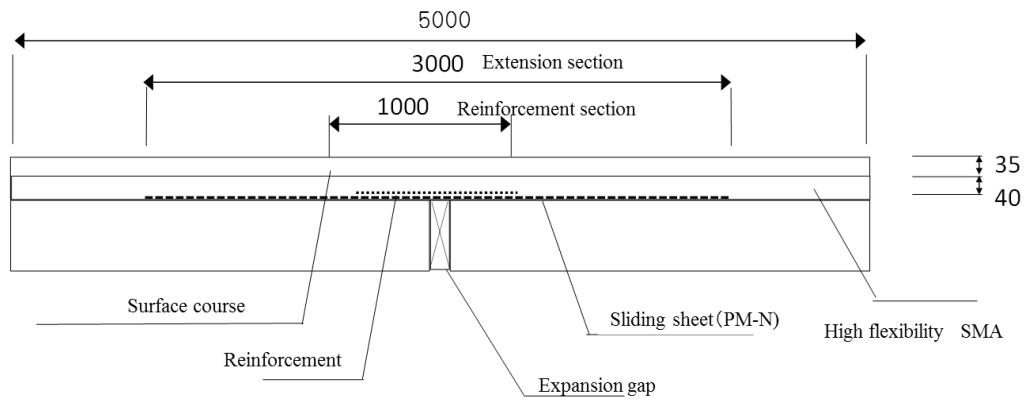


Figure 5. Type B embedded expansion joint cast via machine construction.

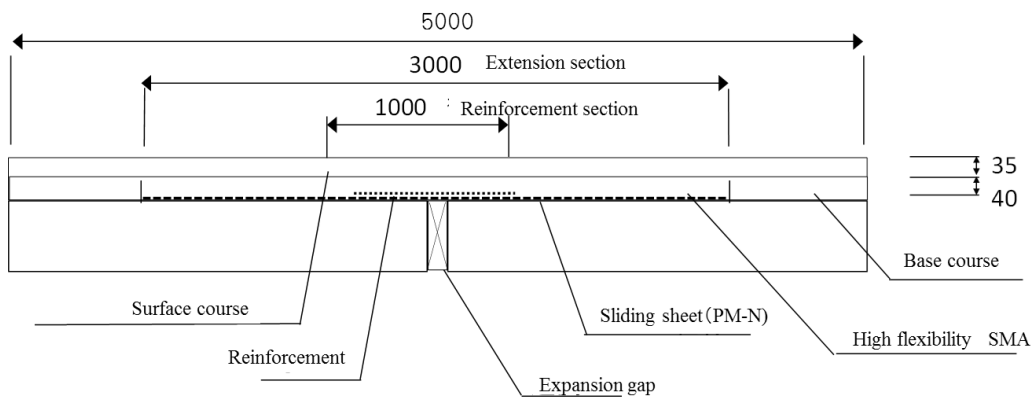


Figure 6. Type C embedded expansion joint cast via human construction

4 WHEEL LOAD RUNNING TESTS

Wheel load running tests were conducted using a device shown in Figure 7 to evaluate the rut, surface condition, and strain. The test load was set to 69 kN, adding 40% impact load to 49 kN of design load. The test load increased as the test construction was not damaged. It was tested 140,000 times for 69 kN, 160,000 times for 83 kN, and 200,000 times for 98 kN load.



Figure 7. Wheel load running test device

5 EVALUATION OF RUT

A rut from each type of embedded expansion joint was evaluated to confirm the pavement damage. Ruts were measured at a point 300 mm from the embedded expansion joint, as shown in Figure 8. The results are shown in Table

1. It was observed that the ruts from all three types of embedded expansion joints were undamaged because ruts measurements are small.

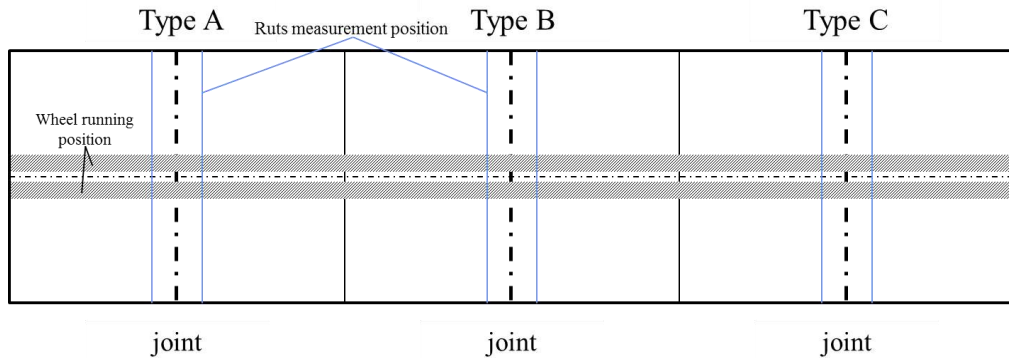


Figure 8. Rut measurement positions

Table 1. Results of rut measurements

	Left wheel	Right wheel
Type A	1.3 mm	1.8 mm
Type B	1.5 mm	1.8 mm
Type C	1.6 mm	2.1 mm

6 PAVEMENT CONDITION

The surface condition of the pavement after the wheel running test. It was performed 200,000 times is shown in Figure 9. Potholes and cracks were found in the Type A embedded expansion joint. However, the Type B and Type C embedded expansion joints were mostly undamaged.

Test structures were dismembered to confirm the condition inside the pavement. The Type B and Type C embedded expansion joints were not damaged. However, the Type A embedded expansion joint showed small cracks rising from under the base course and moving towards the top surface, as shown in Figure 10.

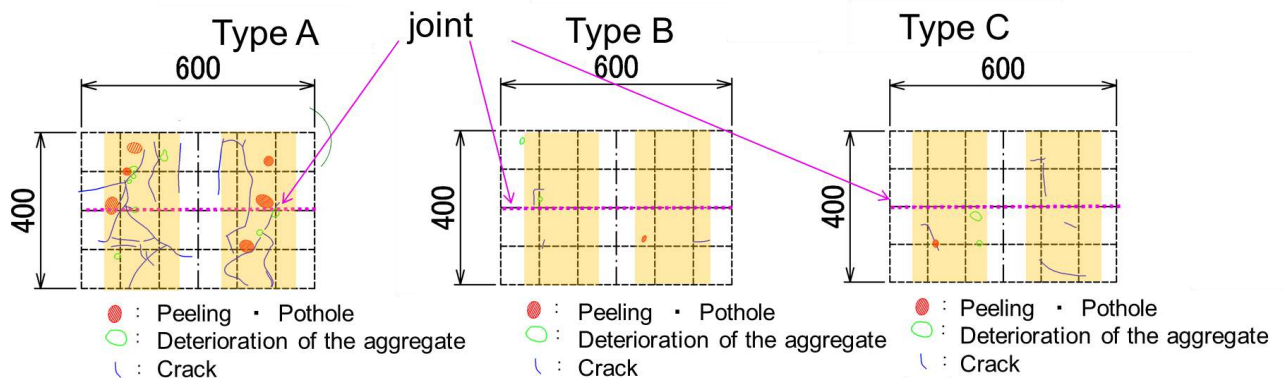


Figure 9. Surface condition of the pavement.

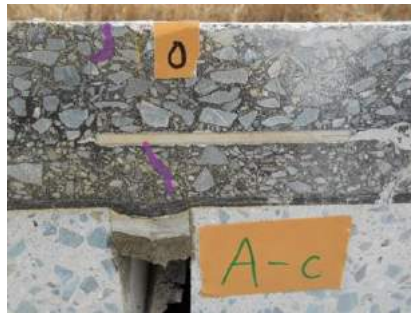


Figure 10. Cracks inside the pavement

7 MEASUREMENT OF STRAIN

Strain sensors were set at three points: surface and upper part of base course and under side of the base course, as shown in Figure 11. In order to examine the change in strain, strain distribution graphs were plotted on the basis of the results of the strain measurements. As shown in Figure 12, strain distribution upper part of base course in Type A embedded expansion joint changed from tensile strain to compressive strain. This result shows that damages in the conventional embedded-joint occur between the surface course and the base course. Strain distribution in the Type B and Type C embedded expansion joints are shown in Figures 13 and 14, respectively. Type B is the most stable embedded expansion joint. The Type C embedded expansion joint is initially stable like Type B, but the strain sensor on the surface of the Type C embedded expansion joint was broke after the wheel running tests were conducted 160,000 times. In addition, strain distribution of the Type C embedded expansion joint changed in a way similar to that observed for the Type A embedded expansion joint when the wheel running test was conducted 180,000–200,000 times. It shows that the damages in the new embedded joints are similar to those observed in the Type A embedded expansion joint.

Considering the abovementioned results, high durability is expected in the two new embedded expansion joints, especially Type B.

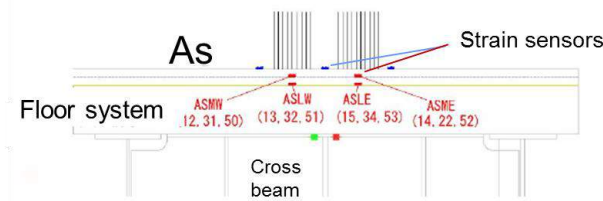


Figure 11. Position of the strain sensors

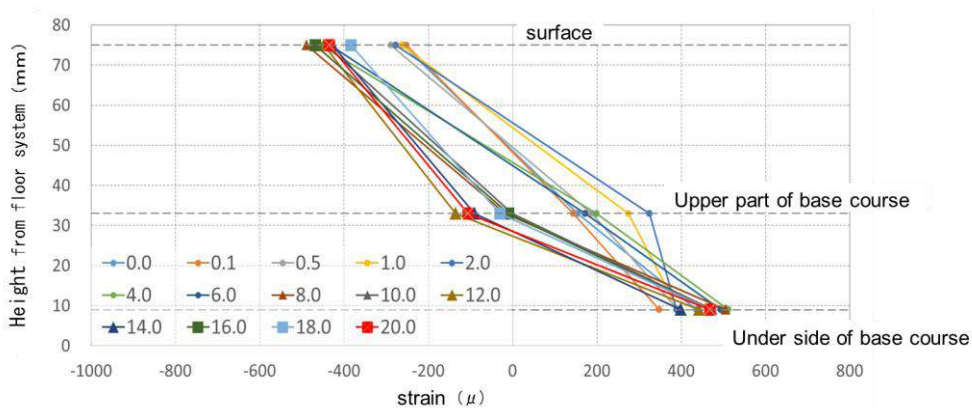


Figure 12. Results for the strain distortion of the Type A embedded expansion joint.

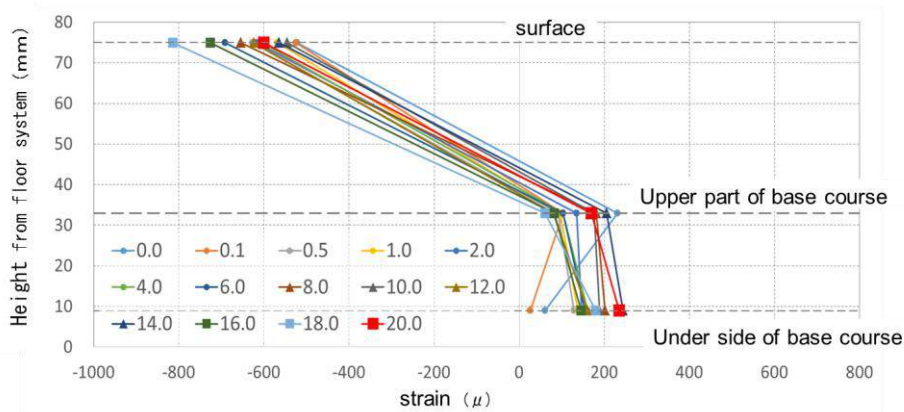


Figure 13. Results for the strain distortion of the Type B embedded expansion joint.

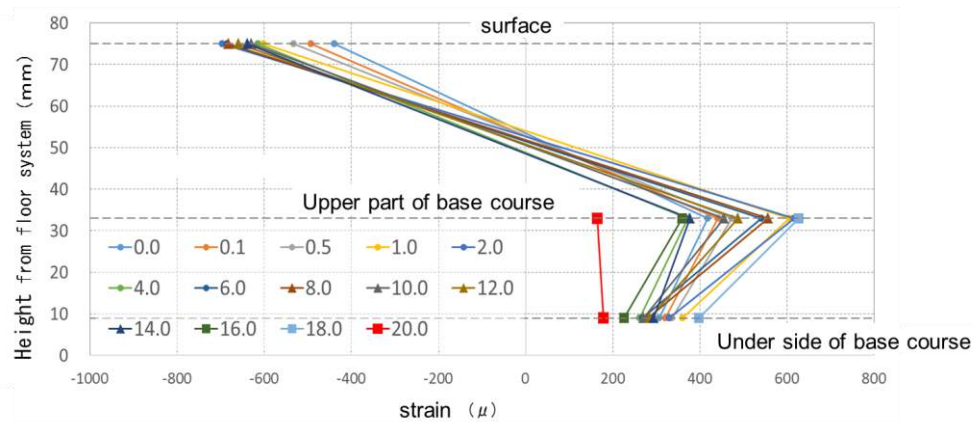


Figure 14. Results for the strain distortion of the Type C embedded expansion joint.

8 CONCLUSION

The findings of this study are as follows.

1. The wheel running tests show that the damage to the surfaces of the new embedded expansion joints was lesser than that to surfaces of the conventional embedded expansion joint. In addition, the test construction of the new embedded expansion joint was no damage.
2. Cracking inside the pavement in the case of a conventional embedded joint was confirmed.
3. Strain distortion in the conventional embedded joint caused more damage between the surface course and base course with increasing number of load running tests. In addition, a similar tendency was observed in a new embedded expansion joint fabricated via human construction.
4. Machine construction of high-flexibility SMA resulted in quality.

Our results show that the new embedded expansion joints, which were fabricated using high-flexibility SMA, possess high durability when used in the all base course in maintenance work.

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PAPER TITLE	Network Evaluation of Flood Impact for Rural Highway Management		
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KEYWORDS:

Network vulnerability, Highway Management, Scenario Analysis, Priority Ranking

ABSTRACT:

This practical paper presents the network analysis under flood situation for rural highway. It proposes a guideline for highway management both pre and during a flood crisis. This general guideline may be adopted and applied in any area in Thailand. The road network modelling is an important tool to evaluate the vulnerability and sensitivity of the network under various scenarios such as that all main road links are submerged, etc. The result is shown as volume-capacity-ratio and is considered with the other considerations such as the primary function of each link, its flooding history, probability of flood occurrence, the present traffic volume to be affected, etc. A case study in Nakhon Sawan province is presented to demonstrate the applicability of the proposed framework. The finds shade light to prepare and improve the highway network in the province and its vicinity so that the crisis could be proper tackled and relieved.

Network Evaluation of Flood Impact for Rural Highways Management

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1. INTRODUCTION

After the flood events in 2011, it had brought a severe loss to the country, many roads were cut off, and people were suffered from the events. This was significantly affecting the enormous economic value, the Department of Rural Roads (DRR) was affected with more than 1,000 roads in 64 provinces with a total value of approximately 9,000 million baht. DRR is responsible for rural highways and serve with conveniences and safety. Especially in the event of flooding, DRR is taking part in reducing the economic loss, helping suffered people and resolving the crisis quickly and effectively with respect to the government policy. DRR has realized that this duty is very important therefore DRR has been studying and preparing to set guidelines and tools to take in action during the flooding event so that the staff can perform more effectively.

1.1.1 STRAHNET Network

The United States has imposed a citywide strategy (STRAHNET: Strategic Highway Network) to provide a stable road for using in the logistics when a natural disaster occurs, for example; natural disaster, war etc. (as shown in Figure 1). It contains the road with multi-agency including inter-state highway and State Highway.

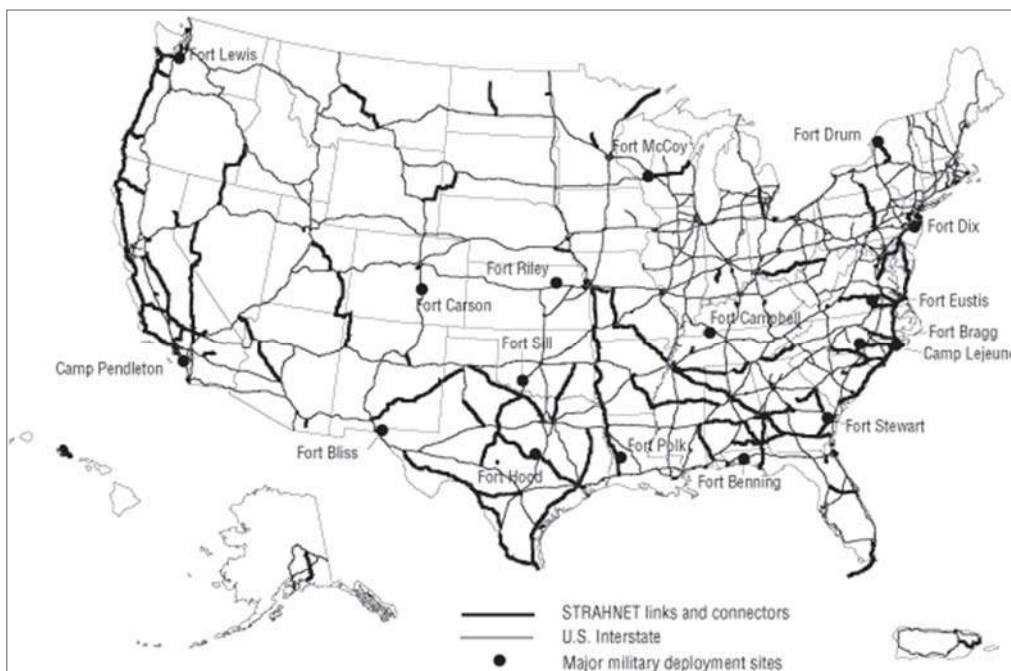


Figure 1 STRAHNET

2. GUIDELINE TO TACKLE FLOOD EVENTS

Department of Rural Roads has set guidelines to deal with floods, which consists of two phases before and during the floods as shown in Figure 2

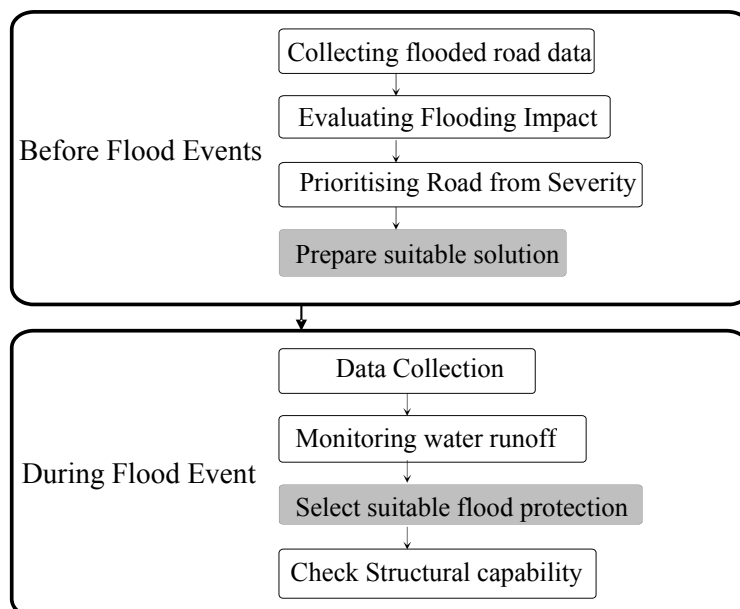


Figure 2 Guideline to tackle flood events

Guidelines for planning preparation to tackle floods during the events are shown in Figure 3.

3. NAKHON SAWAN REGION

The topography of this province is mostly flat, the west end area is mountainous and forest areas and the eastern tip is a highland slopes down to the lowland areas is the center of the province. There are rivers that flow into the province in the north, including Ping and Nan Rivers. (Yom River join Nan River at the upper area of the Province) and these rivers join together into one river called Chao Phraya River. Therefore, this province which has two large rivers, during the raining season there is a large amount of water flows into many areas of the province, especially in 2011 the flooding that occurred causing a lot of damages.

4. ASSESSMENT OF FLOOD AREA

Assessment of flood area are considered from causes of the flooding. There are three factors that are the main causes, amount of rain fall on the area, water runoff from the upper area and sea water level. These factors have critical effect differently to each area such as in Nakhon Sawan, water runoff from the upper area and the rain that fell in the area were the most important factors, but the impact of the sea level was not considered. On the other hand, Bangkok Province has all three flooding factors affecting flood event. It can be concluded that the province locates in the upstream areas has the main factor that caused flooding is the amount of rain water that falls in the area. The province which locates in the middle of the basin, the water runoff from the upper area and the amount of rain would be the most important factors. Finally, for the province in the upper watersheds, particularly flat area near the sea has all three factors causing the floods. Thus, in a study to determine ways to assess the risk of flooding in the area, it depends on the location and topography of the area of interest are important.

The main areas affected by water runoff from the northern are the central areas and the areas towards the east of Nakhon Sawan, for the west area of the province and the eastern tip, which is the area higher level than the central area, the cause of flooding occurred mainly due to the amount of rain fall in the. It can be concluded that the causes of the flooding in Nakhon Sawan are water runoff from the upper area and rainwater that falls in the area.

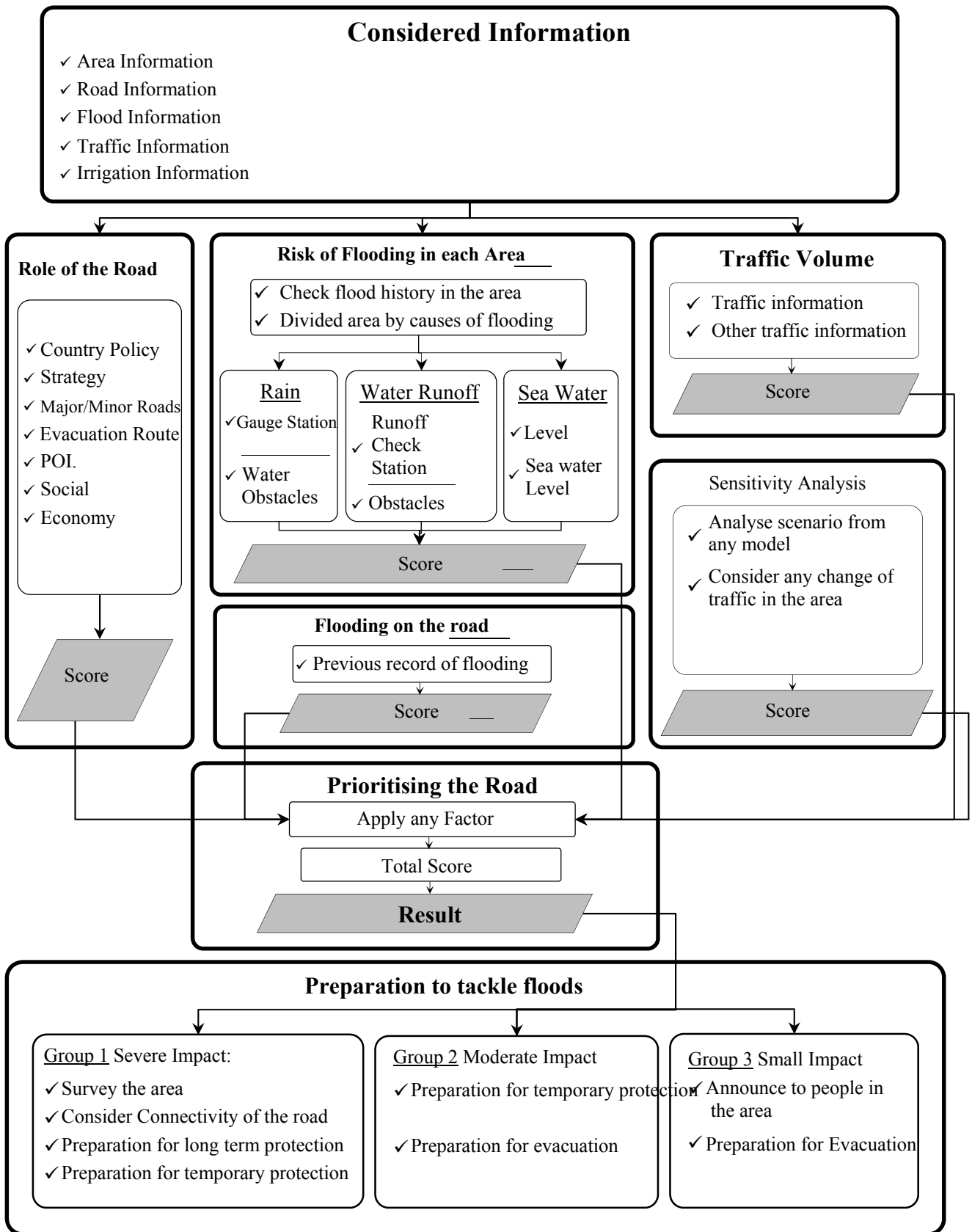


Figure 3 Guidelines for planning preparation to tackle floods during the events

4.1 Rainfall

Data collection from rain gauge stations located in the province and neighboring in total of 67 stations have shown in Figure 4. It was found that the average rainfall is 1044.1 mm / year, with a range between 524.9 to 1545.0 mm / year. The analysis of rainfall found that the amount of precipitation annually average of 1044.1 mm, most rain was falling between May and October which is 87.57% of the average annually rainfall, or equivalent to 914.3 millimeters of rainfall, while the remaining percentage of 12.43% or 129.7 mm rainfall was occurred during November to April. Also an analysis of the highest rainfall period from 1 day to 3 days for applying in determining the repeat amount of rain water of heavy rain in the area from 2 years to 1,000 years which can be summed up the highest rainfall at any period of the rain gauge stations located in the province and neighboring areas.

4.2 Runoff

The runoff gauge stations are located in the province and neighboring stations with a total of 25 locations as shown in Figure 5. The area can reserve rain water from 457 to 120,693 km² with the range of average annual runoff per unit area is between 3.483 to 32.045 liters/sec/km². For the river runoff station Banphotphisai (P.17) was unusually high during the dry season, which is caused due to the release of water for irrigation and power generation of the Bhumipol reservoir in Tak. Nan River station at Koei Chai (N.67) had similar situation from Sirikit reservoirs which released water for cultivation during the dry season. This is quite different from the variability of river runoff at Wang River Station (CT.4) and the Yom River (Y.5), which has no reservoir to control the flow. Thus it was seen that the amount of water flowing down from the north, part of it, was controlled by two large reservoirs. However, such a large number of uncontrolled runoff from the rain in the south of the reservoir, Yom and Wang rivers. The runoff is a major cause of flooding in the central area of the province. Local flooding that had occurred in the past seven years since 2005 to 2011 is shown in Figure 4, it can be seen that there are 3 zones of flooding, which are;

- Zone 1 West Bank.
- Zone 2 Central Lowlands.
- Zone 3 Eastern tip.

The cause of local flooding zones 1 and 3 are mainly due to rainfall in the Northern Province itself, and flows into the high central area of the province itself. The cause of the flood zone 2 was water runoff from the Ping and Nan rivers.

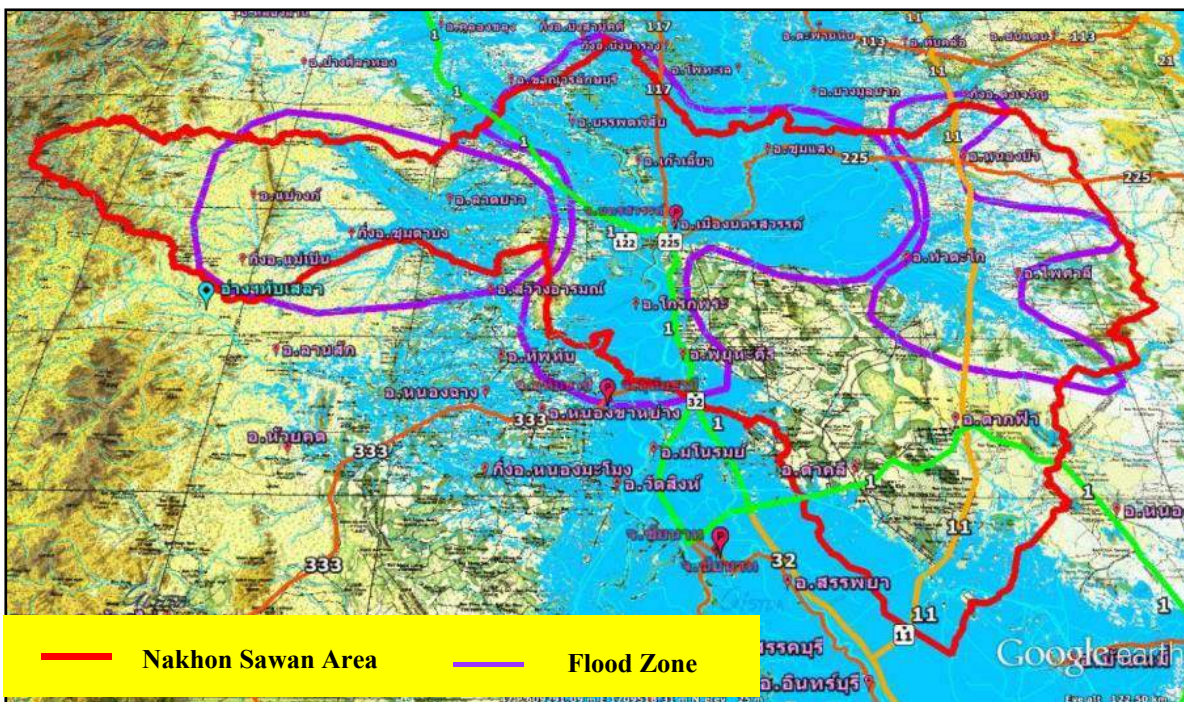


Figure 1 History of flooding in Nakhon Sawan during 2005-2011

5. THE MODEL OF ROAD NETWORK

Road Network analysis was done by applying models of road network using a computer program. It was done by replication traffic on the road network.

5.1 subarea and road network

The study area covered the province and neighboring areas according to its economic and social conditions.

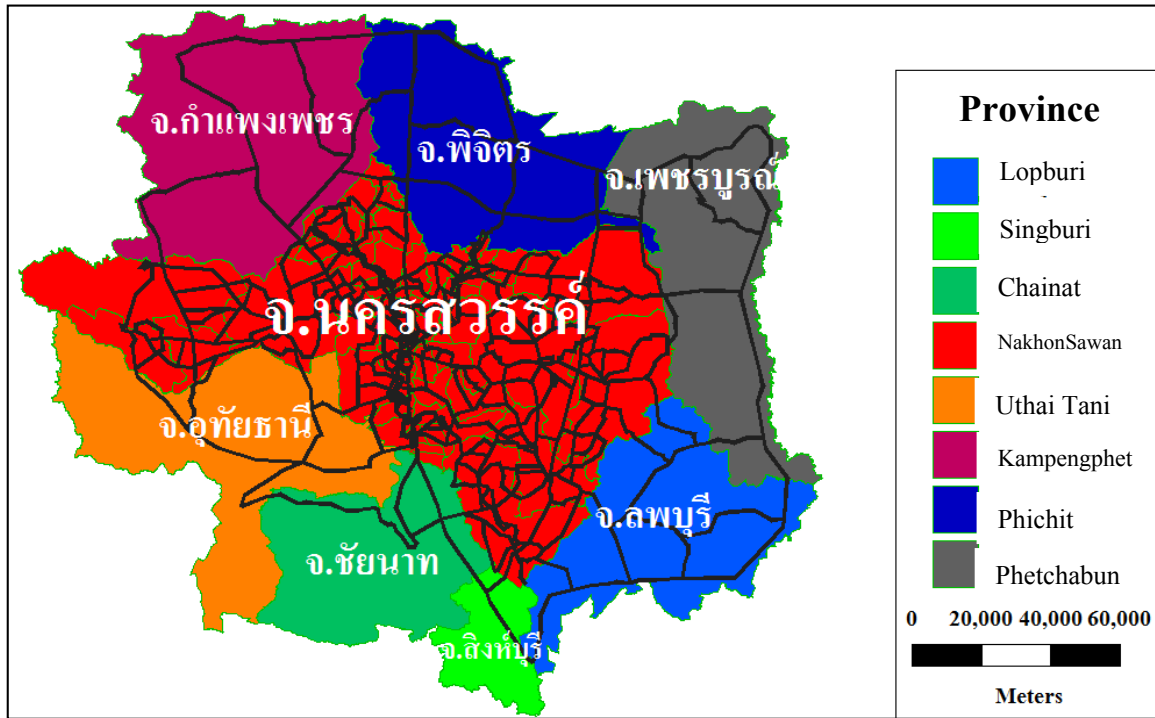


Figure 5 Case Study Area

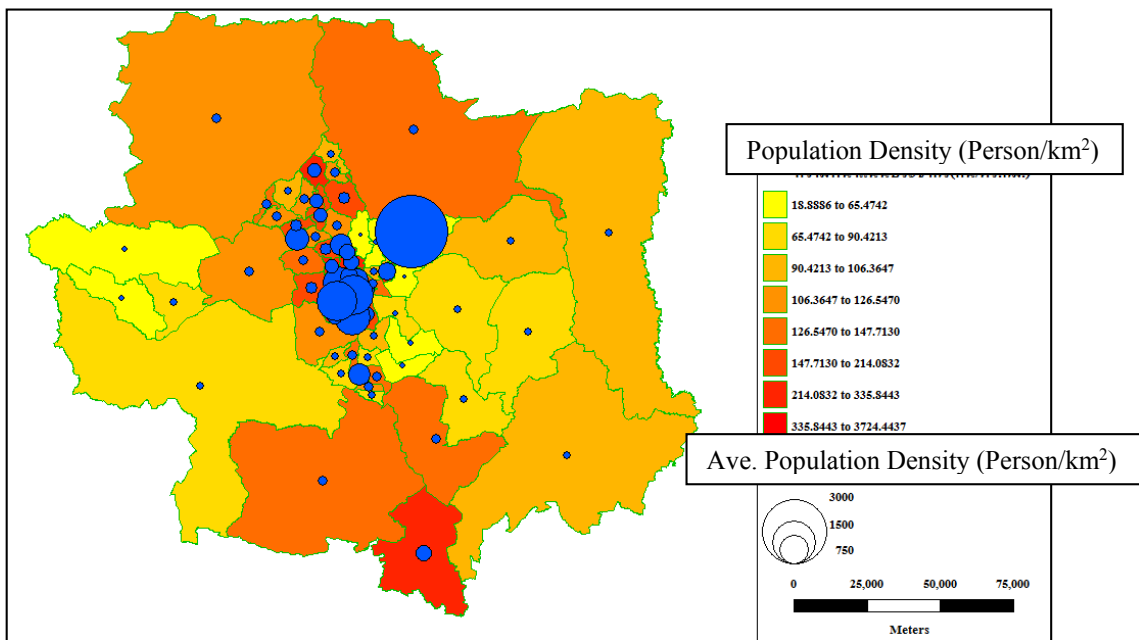


Figure 6 Population Density in the Case study area

5.2 Landmarks

Various locations in the area and important role during the floods are as follows.

- Health centers and hospitals of 22.
- Police Station 23
- Evacuation centers 49
- Fire Station 8
- Army Base
- Bus Terminal
- Nakhon Sawan Railway Station etc.

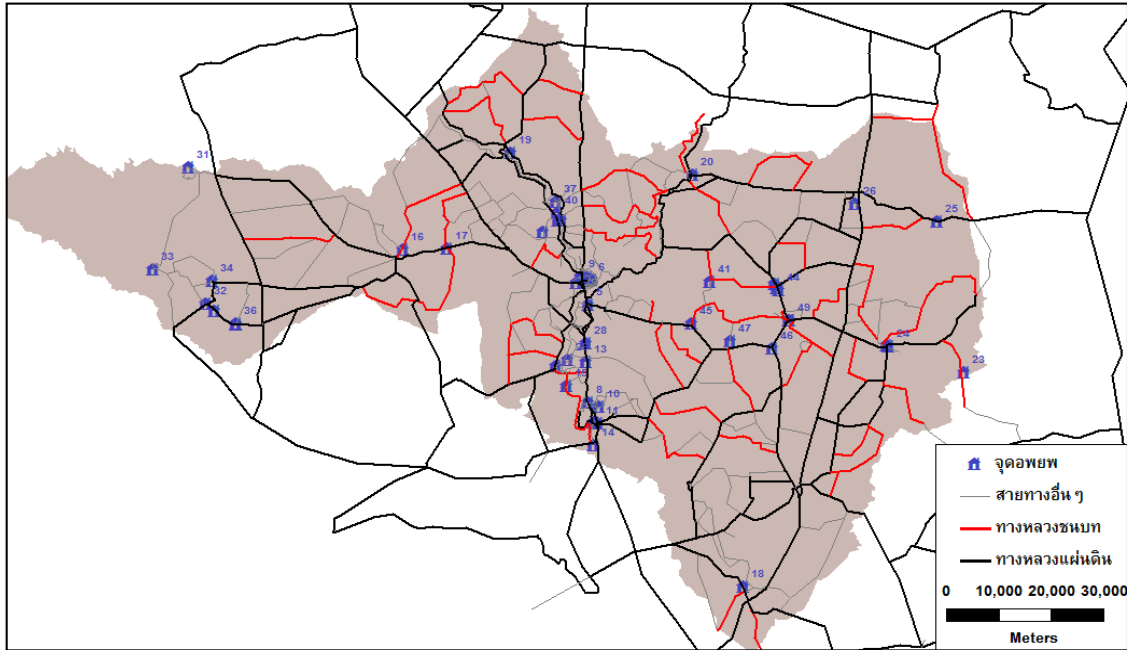


Figure 7 Migration point in Nakhon Sawan

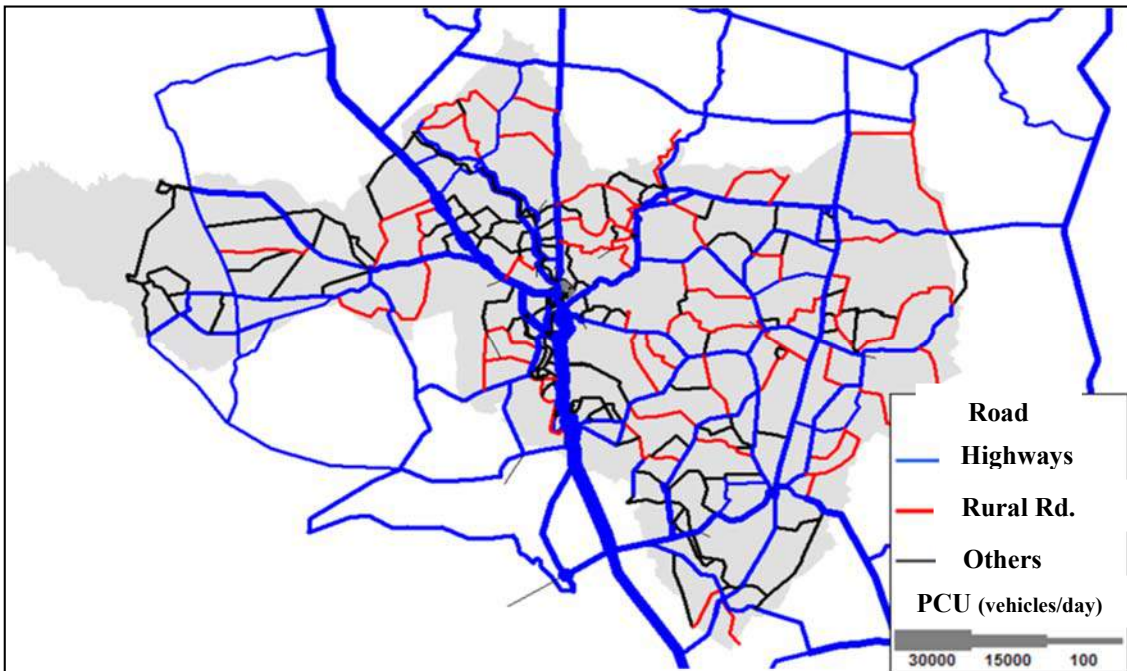


Figure 8 Traffic Volume on the rural road network

6. IMPACT ASSESSMENT OF ROAD NETWORK

Road Network analysis to assess the severity of the impact of floods in each area was able to manage the network effectively and prepare for the flood and managed network during the incident. The factors used in determining factor divided into 3 groups.

- (1) Road Category
 - (2) Cause of flooding.
 - (3) Traffic Volume
- 6.1 Factors and scores

Assessment of the impact was made by the following factors and weighted accordingly in each area. In this study, recommended for general use, as shown in Figure 9.

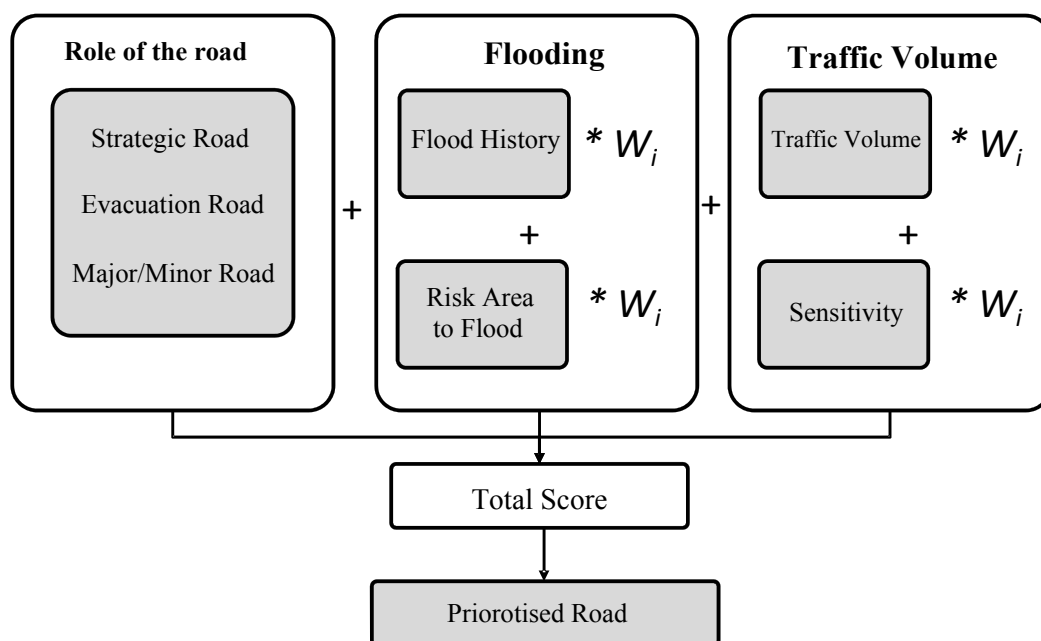


Figure 9 Guideline for scoring and weighting factors

The analysis will have to consider the scores, weighted factors and adjusted as appropriate to the situation in the area. The assessment consists of two parts:

(1) The actual previous incident; based on actual situation which happened to the network, the score is 0.7% of the main factors.

(2) Risk assessment; based on an analysis of possible future flood risk areas for example, assessed by the risk analysis and sensitivity analysis of the network. The score is 0.3% of the main factors, each of the following will be considered:

- The role of the network: considering the importance of applications in areas such as the route which has immigration landmark and is parallel to the highway. This can be used as an escape route when the highway has been affected by the floods
- The previous record of flooding in the area: considering the previous flooding of the area that were affected between the years of 2007 to 2011.
- Risk of flooding in the area: considering the possibility of flooding to the area from various causes.

- Current traffic volume: considering the amount of traffic traveling on those roads.
- Sensitivity: considering the changing traffic conditions in the event of future events which may use the model to analyze traffic networking.

6.2 Scenario

The situation is predicted the impact that would occur on the road network, the detail of analysis can be applied for modelling the road network based on transport engineering. However, if the network is not very complex, the analysis could use statistical information, experiences of traffic analysis in the past, as appropriate. For example, the following are the considered scenarios;

- Under normal circumstances; considering the amount of traffic going in each direction during the flooding.
- Scenario F1; requiring the decrease in some of highway roads ability which were affected by flooding by reducing the capacity of the road and reducing in speed. On the other hand roads under the Department of Rural routes are still fully in service.
- Scenario F2; requiring the condition that ability of highways declined on some routes which were affected by flooding by reducing the road capacity and speed. Also the roads of DRR are not available based on the routes affected by the floods (2011).

6.3 Network analysis

The ratio of traffic volume to capacity (V / C Ratio) under various circumstances are shown in Figure 10 to Figure 12.

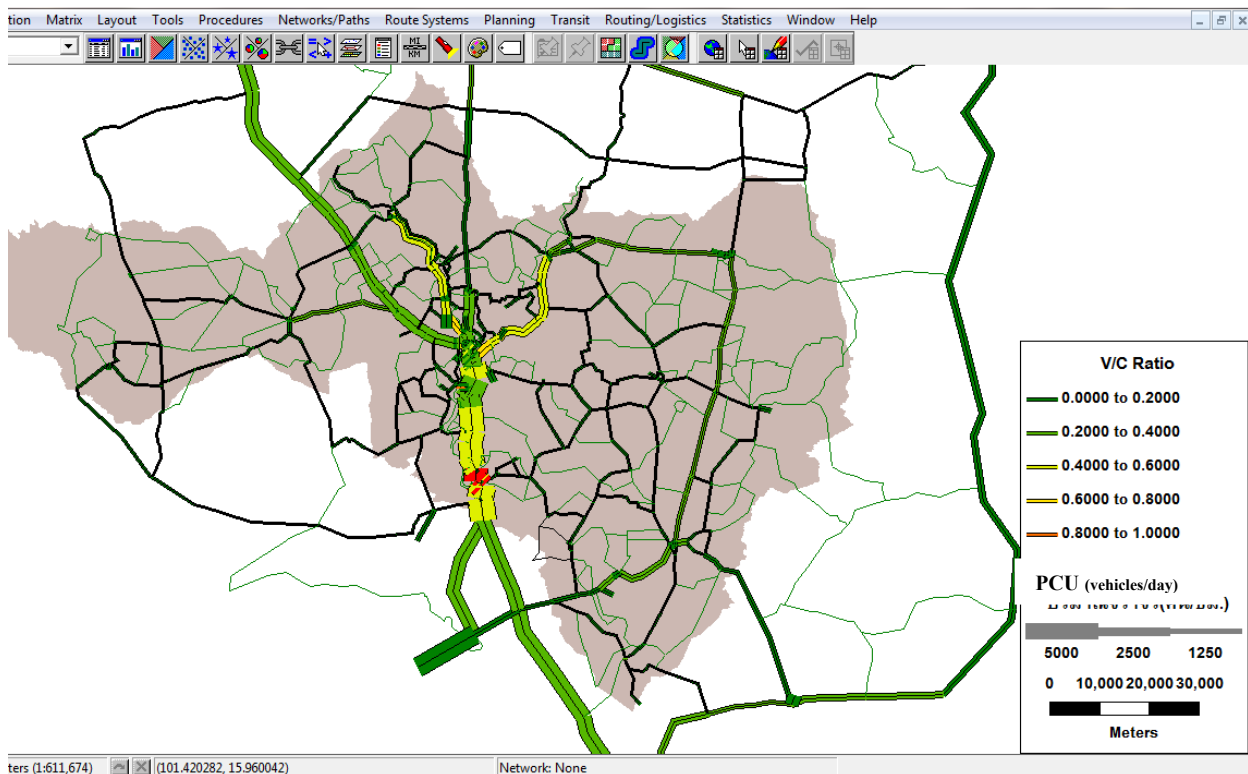


Figure 10 Under normal circumstances

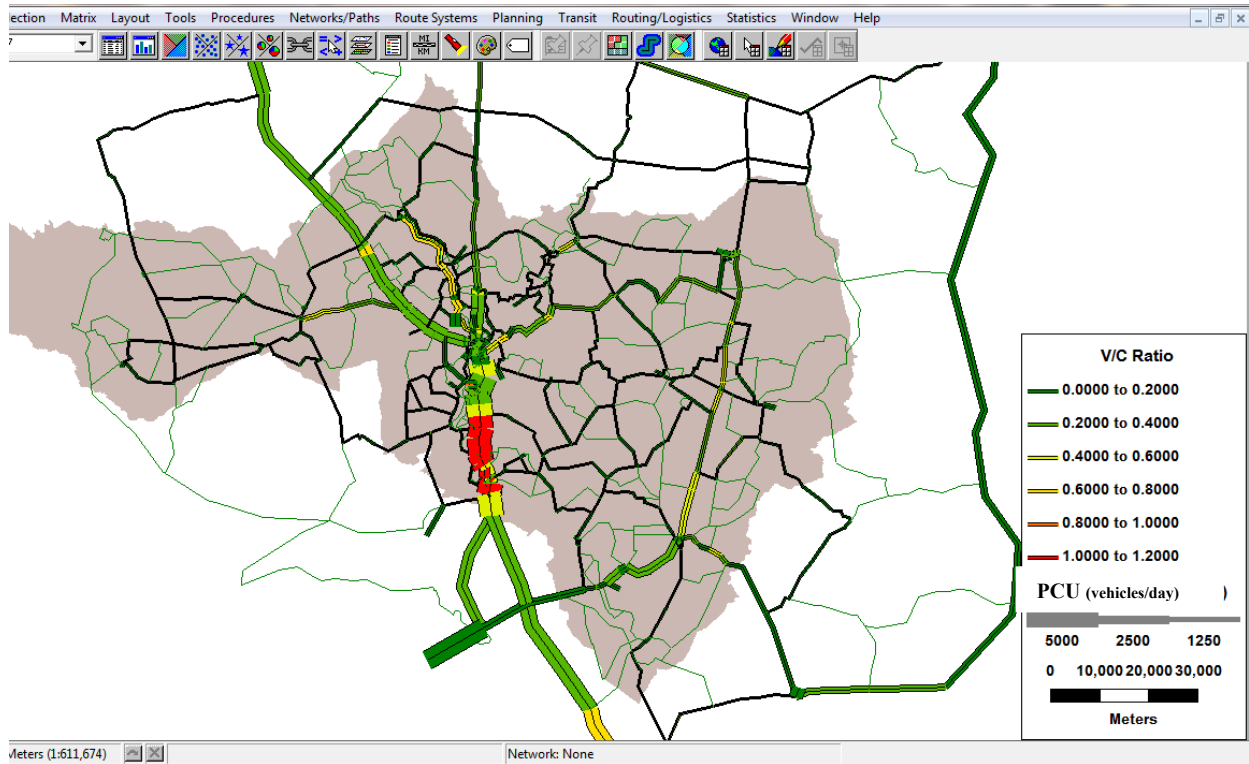


Figure 11 Scenario F1

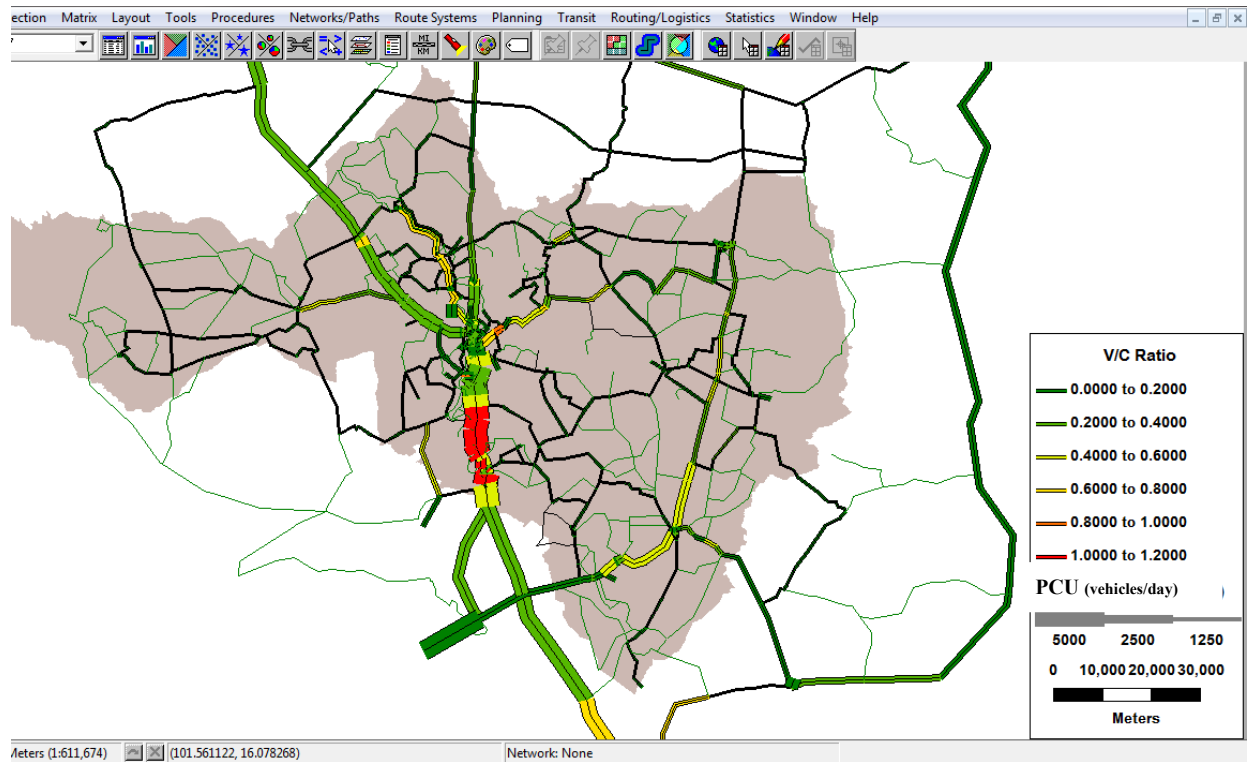


Figure 12 Scenario F2

7. HIGHWAY MANAGEMENT PREPARATION

7.1 Assessment of the flood impact.

Road network in Nakhon Sawan province had all been evaluated according to the guidelines and shown in Figure 13 by considering various factors, including the role of the road, recorded of previous flooding, the risk of flooding, traffic volume, and the sensitivity of the network. These networks were analyzed by the road network model which allows the model to group rural access roads in Nakhon Sawan respect to the impact of flooding into 3 groups, which were;

- Group 1 severe impact: the road with a total of 18 roads which 16 roads from the assessment and additional of two roads from surveying.
- Group 2 moderate impact on the 18 roads.
- Group 3 small effects on the 8 roads.

7.2 The forthcoming plan for severe impact roads.

From assessment of the road network above it was found that the road in Nakhon Sawan which were severely affected by the floods are the main network serving as a secondary routes in the event of flooding, therefore, it needs to be maintained at any time.

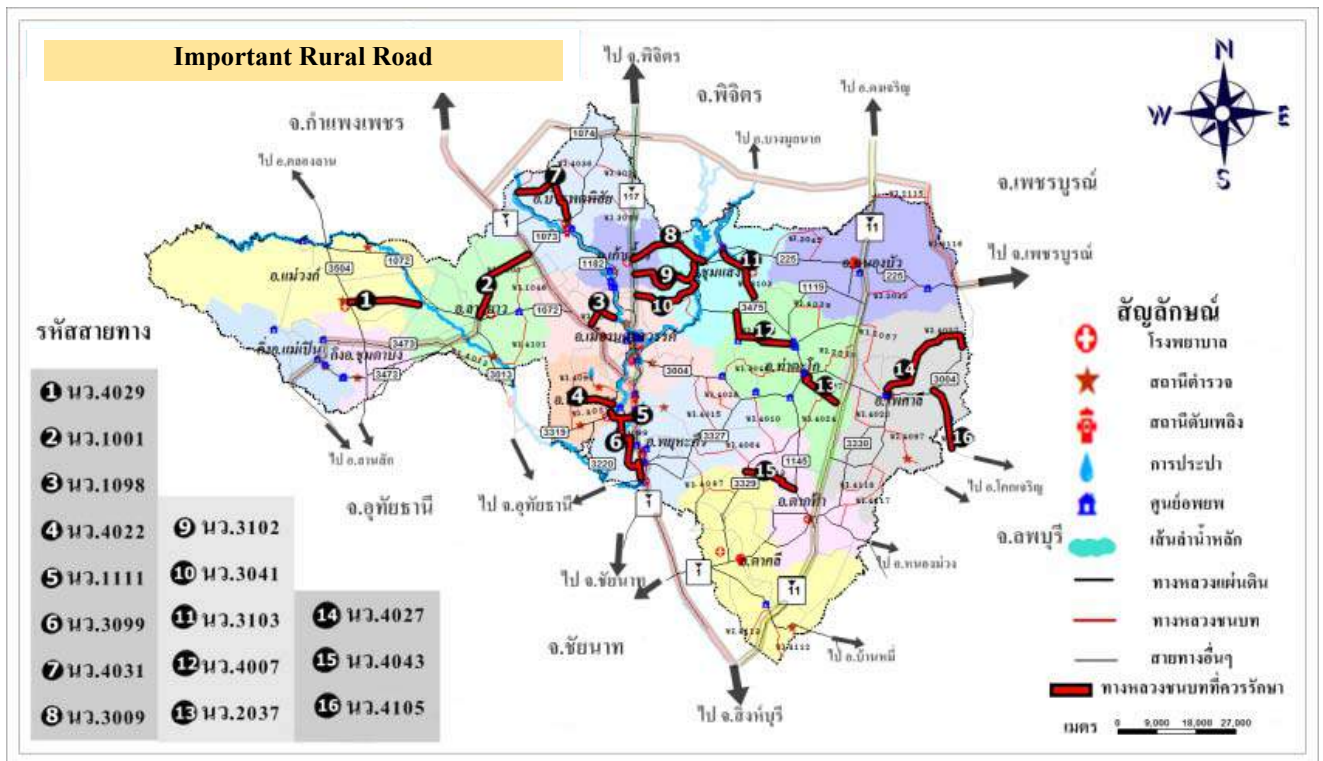


Figure 13 the forthcoming plan for severe impact roads

8. CONCLUSION

This article presents an analysis applications integrated with transportation model planning in the management of road network under the flood circumstances such an analysis is necessary to be prepared to cope with the effects of quantitative analysis makes it possible to understand and verify the update. The case of Nakhon Sawan province confirmed that the analysis gave tangible results that are essential for managing network services to facilitate the traffic with fully safety and in the allocation of the budget is fairly consistent with the reality situation in the area.

9. REFERENCES

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Investigation of engineering properties of bio-based binder derived from palm oil as a replacement for bituminous binder

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This article introduces a new bio-binder containing palm kernel oil polyol (PKO-p) used as a partial replacement for bituminous binder. PKO-p was produced through esterification and condensation for use in pavement mixtures. The proposed PKO-p was blended with a base binder (BB) to produce three variations of bio-binders (BIB), namely 100/0, 80/20, 60/40, and 40/60 bio-binders (weight of BB to BIB). The preliminary results for the conventional properties, high-temperature rheological properties (i.e., viscosity in terms of mixing and compaction temperatures and non-dimensional viscosity analysis), attenuated total reflection Fourier transform infrared (FTIR) spectrometer, and optical microscopy of the bio-binders are highlighted. Results showed that the bio-binder can be used as an alternative for petroleum binder in the construction of road pavements.

Keywords: bio-binder, bitumen, palm oil, physical, rheology, chemical.

1 INTRODUCTION:

Asphalt binder, a sticky and highly viscous material, is produced through the refining of petroleum, and is derived from the bottom of the barrel cut through refinery stages. Asphalt is one of the basic materials used in construction, especially in the construction of pavements. Studies conducted by TOTAL bitumen, Oilibya and MAS showed that 90% of the asphalt produced world-wide are used in the construction of road pavements while the remaining of 10% is used for other purposes such as coating. The high demand for petroleum products, including asphalt, has led to decreasing petroleum reserve and increasing energy cost; this in turn increase the cost of pavement construction. The quality of refined petroleum binder has been declining steadily, and this has encouraged the exploration of new alternatives for asphalt. Research industries are currently investigating a new binder, namely bio-binder (bio-asphalt) extracted from biomass or bio-waste products, which are ubiquitous and a bio-renewable resource.

According to Airey et al. (2008) there are three ways in which bio-binders can be used to decrease the demand for fossil fuel based bituminous binders. They are used (1) as a direct alternative binder (100% replacement); (2) as bitumen extender (25 to 75% bitumen replacement); or (3) as bitumen modifier (<10% bitumen replacement). William et al. (2009) studied bio-oil fractions as an extender in original and polymer modified asphalt binder. The results of this research showed that “the addition of bio-oil enhances the performance grade of polymer binder by 6 °C and it’s shown by this research that the resource of biomass is a significant factor affecting bio-oil properties”. A study conducted by Fini (2010) showed that asphalt performance at low temperature could be improved by using swine-manure bio-binder as an asphalt modifier. Raouf and Williams (2010) studied the rheology of bio-oils made from corn stover and found that the rheological properties of corn stover bio-oil are similar and comparable to that of asphalt binders and hence could be used as a renewable alternative for traditional asphalt binders. Raouf et al. (2010) took the research further and investigated the properties of the bio-oil modified with polymers. Emmanuel et al. (2011) conducted a research titled as an alternative binder from microalgae showed that The bio-asphalt, which dripped, has rheological properties similar to fossil fuel asphalt.

Another study by Seidel and E. Haddock (2012) used soy fatty acids, which is chemically similar to asphalt, to obtain a sustainable modifier. This bio-binder was used as an additive for the main binder and resulted in increased viscosity and stiffness of the asphalt binders, thereby increasing its workability and lowering its compaction and mixing temperature. Joana et al. (2011) conducted a study using varying percentages of bio-binder combined with varying percentages of crumb rubber; they concluded that a bio-binder containing bio-oil reacted with crumb rubber could produce a binder which is comparable to asphalt binders derived from crude petroleum. Furthermore, the bio-oil could be successfully reacted with crumb rubber at 125 °C, which is a substantially lower temperature than that used in normal asphalt binders, which is typically around 185 °C. Zofka and Yut (2012) investigated the rheological and ageing properties of traditional asphalt binders modified with waste coffee grounds and found that coffee beans could serve as a solvent in traditional asphalt binders. Fini et al. (2010) and Mills-Beale et al. (2012) found that bio-oils generated from swine waste could enhance the low temperature performance of asphalt binders and save costs. Asphalt binders modified or partially replaced with bio-oil, together with other normal modifiers such as nano-material and crumb rubber, were also investigated.

Malaysia has the potential of leading the research in biotechnology considering that the country has a rich source of greenery, with massive amounts of timber, palm oil, rubber, cocoa, bamboo and rice. According to the 2013 National Biomass Strategy Report published by Agensi Inovasi Malaysia (AIM), “A significant amount of biomass is generated every year across a variety of crops, including but not limited to palm oil, rubber and rice. Within agriculture, by far the largest contributor to gross national income (GNI) is the palm oil sector, contributing about 8 percent or over RM 80 billion. The palm oil sector correspondingly generates the largest amount of biomass, estimated at 83 million dry tons in 2012. This is expected to increase to about 100 million dry tons by 2020, primarily driven by increases in yield”. Malaysia also has millions of tons of biomass from other sources.

Given this fact, it is worth the effort to explore the possibility of finding new technologies for producing asphalt. The objective of this study is to evaluate a new bio-binder as a good extender for asphalt binders. In achieving this aim several tests and analyses were conducted, such as tests on conventional properties, high-temperature rheological properties (i.e., viscosity in terms of mixing and compaction temperatures and non-

dimensional viscosity analysis), attenuated total reflection Fourier transforms infrared (FTIR) spectrometer, and optical microscopy.

2 EXPERIMENTAL DESIGN

PREPARATION OF MATERIAL

The materials used in this study include palm kernel oil polyol (PKO-p), 80/100 control asphalt binder, and diphenyl methane diisocyanate (MDI). The proposed PKO-p was blended with the base binder (BB) to produce three types of bio-binder (BIB), namely 100/0, 80/20, 60/40, and 40/60 (weight of BB to BIB), and MDI was added in the amount of 30% weight of PKO-p. Thus, a total of four asphalt binders were investigated in this study. The bio-binders were prepared by heating both the bio-oil and control asphalt binder to 110 °C. The bio-oil and control asphalt binder were then blended using a shear mixer at 120°C for 60 minutes at a mixing speed of 1000 revolutions per minute. MDI was then added and the mixing was continued for an additional five minutes. The names of the samples are given in Table 1.

Table 1 Sample compounds

Sample #	Name (Abbreviation)	Samples Variations	
		Base Binder (%)	Bio-oil Compound (%)
Sample 1	BB	100	0
Sample 2	BIB1	80	20
Sample 3	BIB2	60	40
Sample 4	BIB3	40	60

LABORATORY TESTINGS

The tests for conventional properties, high-temperature rheological properties, attenuated total reflection Fourier transform infrared (FTIR) spectrometer, and optical microscopy were conducted to determine the characteristics of the new blends. The conventional test comprise of penetration and softening point tests while the rotational viscometer (RV) test was conducted as a high-temperature rheological test. This allows for the determination of the mixing and compaction temperatures of asphalt mixtures and for the non-dimensional viscosity analysis. While the FTIR test was able to determine the functional group of the blends, optical microscopy was done to determine the structural components of the new blends.

✓ Penetration Test

This test measures the hardness or softness of asphalt binder by measuring the depth in tenths of a millimetre to which a standard loaded needle penetrate vertically in five seconds. The equipment and test procedure have been standardized by the BIS. The penetrometer consists of a needle assembly with a total weight of 100g and a device for releasing and locking in any position. The asphalt binder was softened to pouring consistency, stirred thoroughly, and poured into containers at least 15 mm thicker than the expected penetration. The test was conducted at 25°C as specified by the ASTM D5.

✓ Softening Point Test

The softening point (ring and ball test) was carried out on all samples in accordance with the ASTM D36-76. In this test, asphalt binder disks were cast in shouldered rings. The disks were then trimmed to remove excess asphalt binder and heated at a constant rate of 5°C/min in a water bath using a special apparatus.

✓ Rotational Viscometer (RV) Test

The RV test was conducted to determine viscosity at high temperatures and the workability of asphalt binders. The RV test for asphalt binder and blends could determine the optimal mixing and compaction temperatures of asphalt mixtures during the construction. The RV test conducted in this study was in accordance with the standard AASHTO T 316 and test temperatures were e 100, 120, 135 and 165°C.

✓ **Optical Microscopy**

Optical microscopy was used to the morphology of unmodified and polymer modified asphalt binders. A drop of heated sample was placed between two microscope slides and the samples were observed at room temperature using an optical microscope.

✓ **Fourier Transform Infrared Spectroscopy (FTIR)**

Fourier transform infrared spectroscopy (FTIR) is one of the two vibrational spectroscopy (Infrared and Raman) techniques widely used in industry. FTIR provides qualitative (through fingerprinting), semi-quantitative, and quantitative information of chemical structures and physical characteristics. Solid, liquid, or gas samples can be analysed in the form of a bulk or thin film. In infrared spectroscopy, infrared radiation is passed through a sample. Some of the infrared radiation is absorbed by the sample while others passed through (transmitted) the sample. The resulting spectrum represents molecular absorption and transmission, thus creating a molecular fingerprint of the sample. Just as a fingerprint, no two unique molecular structures produce the same infrared spectrum, as shown in Table 2.

Table 2 FTIR spectra main bonds (Functional Groups)

Functional Group	Characteristic Absorption(s) 1/cm
Alkyl C-H Stretch	2950-2850 (m or s)
Alkenyl C-H Stretch	3100-3010 (m)
Alkenyl C=C Stretch	1680-1620 (v)
Alkenyl C-H Stretch	~3300 (s)
Alkenyl C=C Stretch	2260-2100 (v)
Aromatic C-H Stretch	~3030 (s)
Aromatic C-H bending	860-680 (s)
Aromatic C=C bending	1700-1500 (m,m)
Alcohol/phenol O-H Stretch	3550-3200 (broad, s)
Carboxylic Acid O-H Stretch	3000-2500 (broad, v)
Amine N-H Stretch	3500-3300 (m)
Nitrile C=N Stretch	2260-2220 (m)
Aldehyde C=O stretch	1740-1690 (s)
Ketone C=O stretch	1750-1680 (s)
Ester C=O stretch	1750-1735 (s)
Carboxylic Acid C=O Stretch	1780-1710 (s)
Amide C=O Stretch	1690-1630 (s)
Amide N-H Stretch	3700-3500 (m)

3 RESULTS AND DISCUSSION

✓ **PENETRATION TEST**

The penetration test was conducted to determine the stiffness of the blends. Figure 1 shows that the values for BB and BIB1 are within the standard of 80/100 penetration grade asphalt binder while the values for BIB2 and BIB3 are higher than the value for 80/100 penetration grade asphalt binder, which indicate that these blends have higher stiffness, namely closer to the 60/70 penetration grade asphalt binder.

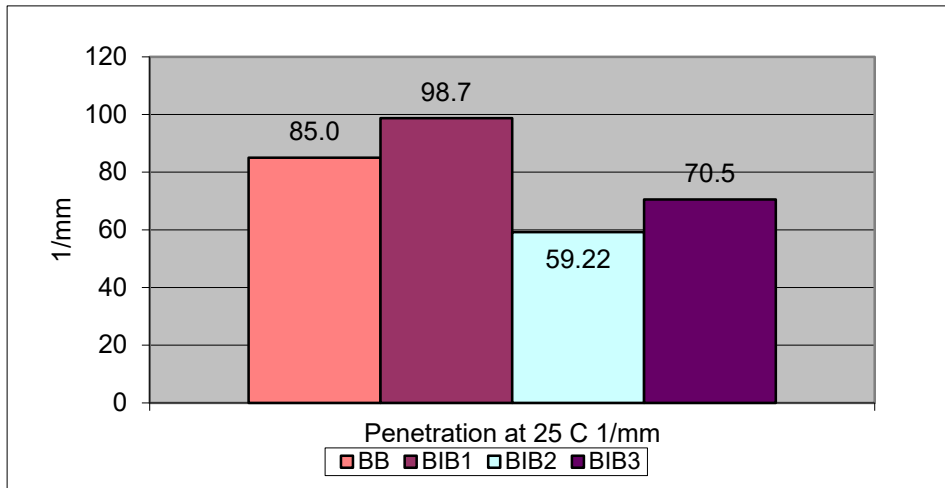


Figure 1 Results for Penetration Test

✓ **SOFTENING POINT TEST**

The ASTM D-36 and asphalt binder 80/100 softening point standard state that the values for this test should be between 45 and 52 °C. The results for all samples is shown in Figure 2. The value for BB match the standard value; the values for BIB2 and BIB3 are 46 and 45.5 °C respectively, while the value for BIB1 is 40 °C. The values for the penetration and softening points showed the same trend, in that samples with higher penetration value have lower softening point. However a general trend looks not applicable to be derived and viscosity analysis has to be conducted to better understanding.

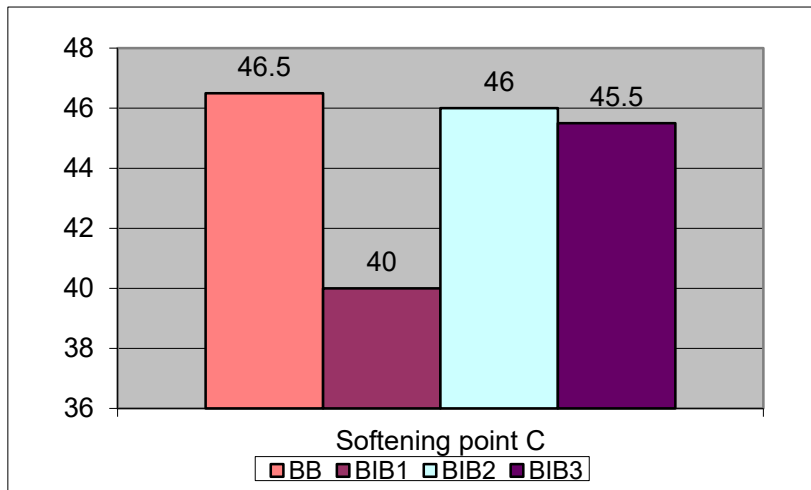


Figure 2 Results for Softening Point Tests

✓ **ROTATIONAL VISCOMETER (RV) TEST**

Figure 3 illustrates the rotational viscosities for control asphalt binder and bio-binder blends. The rotational viscosities for BB, BIB2 and BIB3 have the same altitude of viscosity, while BIB1 has a slightly lower viscosity value. This indicates that all BIB blends are similar to BB. The figure also show that the trends for BB, BIB2 and BIB3 are close to one another. When the mixing and compaction temperatures were evaluated, it was found that the temperatures of BIBs are 10-15% lower than that of the BB, which indicate that less energy was utilized. This is shown in Table 3.

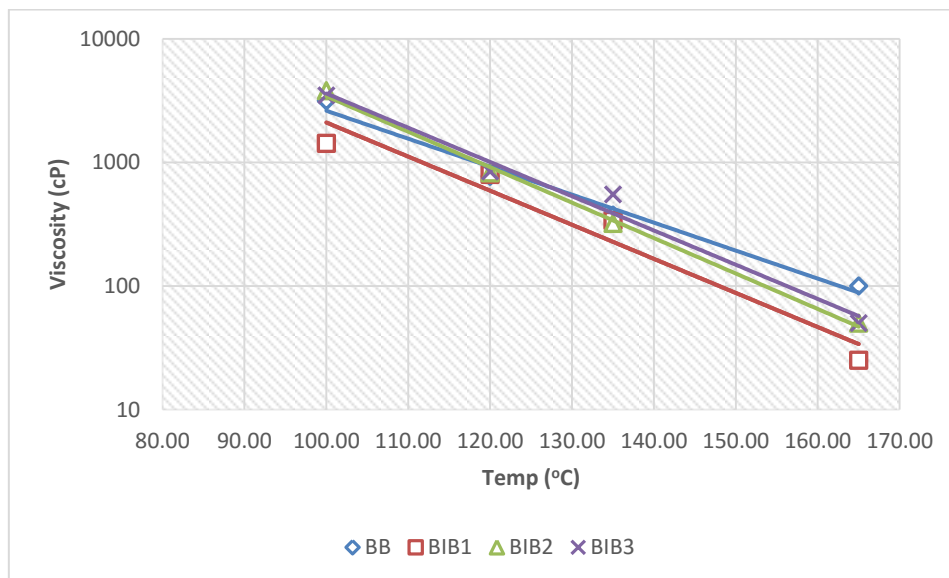


Figure 3 Viscosity vs. Temperature

Table 3 Mixing and compaction temperatures

Construction Temp (°C)	BB	BIB1	BIB2	BIB3
Mixing	153	136	148	147
Compaction	143	129	141	139

✓ **NON DIMENSIONAL VISCOSITY**

A non-dimensional viscosity index, expressed by Equations (1) and (2), was used to compare and characterize the rate of change in the rheological properties of BIBs with BB in terms of viscosity. .

$$\eta = \frac{v}{v_c} \quad \text{-----(1)}$$

$$\nabla_{\eta} = \left(\frac{\delta \eta_N}{\delta N} \right) = \left(\frac{\Delta \eta_N}{\Delta N} \right) = \frac{\eta_N - \eta_C}{N} \quad \text{-----(2)}$$

where η is the relative viscosity of the blends;
 v is the viscosity of the binder blends,
 v_c is the viscosity of the control binder; and
 ∇_{η} is the non-dimensional viscosity index.

In addition, $\nabla\eta$ was used to determine the effect of adding one-unit percent PKO-p material on the viscosity of the bituminous binder at each test temperature.

Figure 4 and Table 4 show the relationship between $\nabla\eta$ and temperature for each bio-binder blend. The non-dimensional viscosity index values for BB, BIB1, BIB2 and BIB3 are different from one another. For instance, at 100 °C, the $\nabla\eta$ values for BIB1, BIB2 and BIB3 blends are -2.74%, +0.516% and +0.179%, respectively. These results showed that adding 1% PKO-p decreases the viscosity of BIB1 by -2.74%, while the viscosity for BIB2 and BIB3 were increased by +0.516% and +0.179% respectively. At 165°C the $\nabla\eta$ values for BIB1, BIB2 and BIB3 blends are -3.75%, -1.25% and -0.83% respectively. These results showed that the addition of 1% PKO-p decreases the relative viscosity of the bitumen blends by -3.75%, -1.25% and -0.83% for BIB1, BIB2 and BIB3, respectively. This means that the change in performance caused by adding one unit per cent of PKO-p vary for the same test temperature when measured in terms of $\nabla\eta$.

The performance of asphalt binder mixed with bio-binder changed proportionately with test temperature and the weight of the additive. However, no obvious trends was observed, as can be seen in Figure 4. For example, the $\nabla\eta$ for BIB2 is +1.688% at 120 °C, while at 135 °C the value decreased to -0.36%. The evaluation of the relative change in viscosity caused by adding 1% PKO-p can be complicated.

Table 4 Computation of relative viscosity

TEMP C	Viscosity MPa.s				Relative Viscosity				Diff. in relative visc. BIB %		
	BB	BIB1	BIB2	BIB3	BB	BIB1	BIB2	BIB3	BIB1	BIB2	BIB3
100.00	3150	1425	3800	3488	1.00	0.45	1.21	1.11	-54.76	20.63	10.73
120.00	770	800	822	843	1.00	1.04	1.07	1.09	3.90	6.75	9.48
135.00	375	338	321	550	1.00	0.90	0.86	1.47	-9.87	-14.40	46.67
165.00	100	25	50	50	1.00	0.25	0.50	0.50	-75.00	-50.00	-50.00

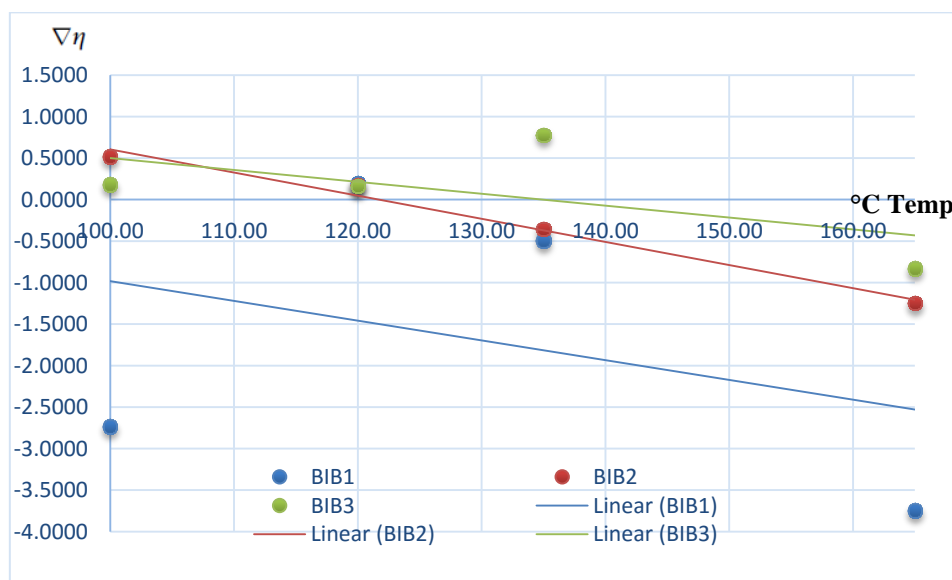


Figure 4 Temperature (°C) versus non-dimensional viscosity index

✓ OPTICAL MICROSCOPY

The morphology of the samples were developed to monitor separation. Figure 5 (a) and (b) show that the components of the new blends have formed new materials and no separation was observed in the microscopy images. Also, the morphology of the blends show the formation of homogeneous materials without any clots. FTIR was done to evaluate the internal bonds.

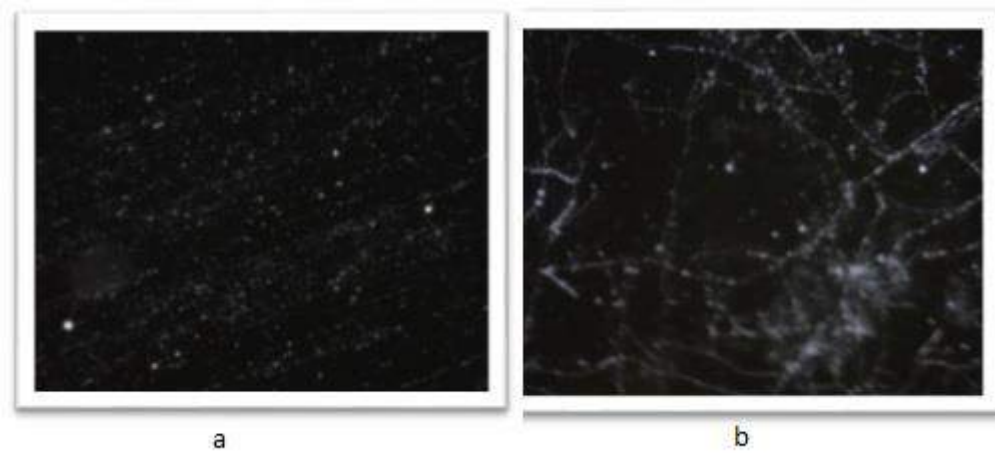


Figure 5 Optical microscope images for (a) BB and (b) BIB2

✓ FTIR

FTIR is used to obtain the absorbance/transmittance spectrum of solid as well as liquid and gas materials. The bonding of the samples investigated using the FTIR. So, Figure 7 shows the chemical structures which is useful as a reference for bonding study. Investigation of the bonding of BB and BIB2 (Figure 6) show a higher peak at 3350-3400, which is caused either by the N-H (amides), which is a part of the base binder, but increased in the new BIB due to the reaction of PKO-p with MDI, or O-H (alcohols) which comes from the residue of PKO-p which have not reacted with the MDI. However, the microscopy indicate that the peak was not caused by residue. On the other hand, the peak at 1700-1740 for C=O was formed by hydrogen bonding which is crucial for the stability of the material. Also the C=C aromatic ring at 1500-1540 was produced by the MDI reaction. Generally, the other main bonds of the material has reformed well. To conclude that the investigated bio-binder may consider a significant development of a new material in asphalt technology.

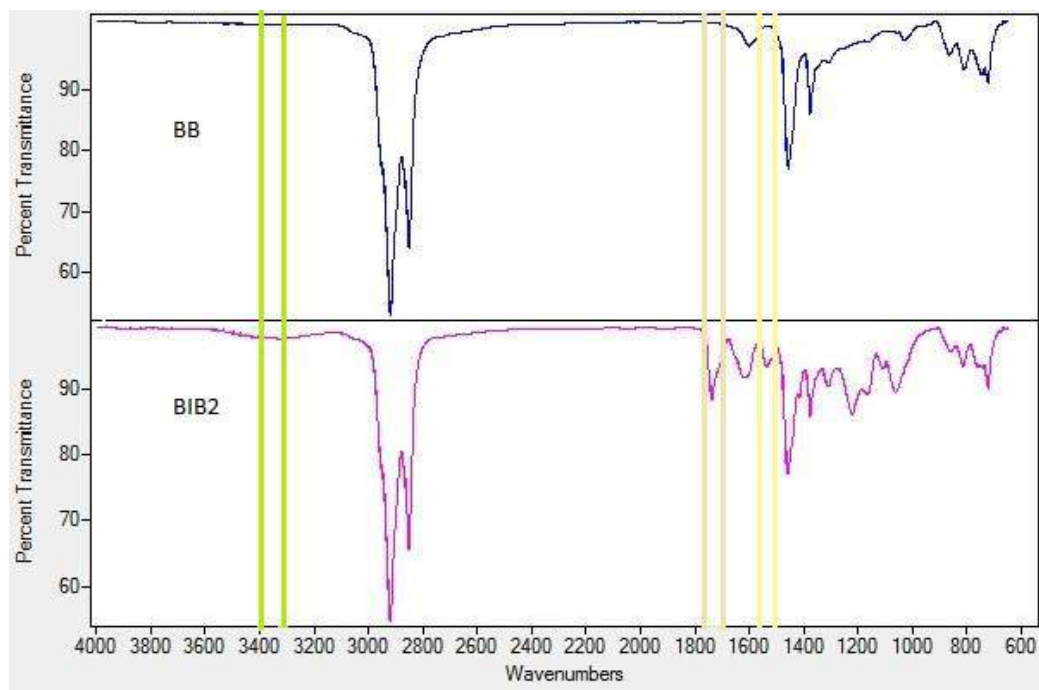


Figure 6 FTIR spectrum indicating Functional groups in BB and BIB2

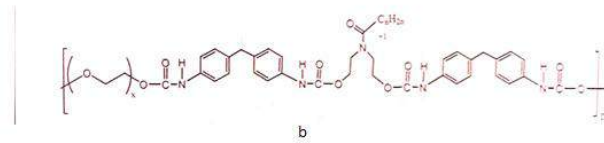
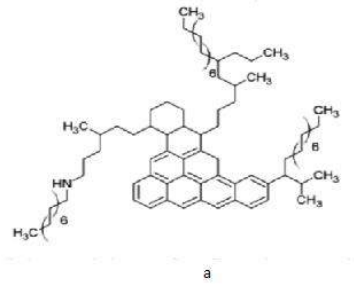


Figure 7 Chemical Structures of (a) BB and (b) PKO-p and MDI.

4 CONCLUSION

This article evaluated the performance of a new bio-binder made from palm kernel oil polyol (PKO-p) as a partial replacement for asphalt binder. The tests conducted revealed that:

- The values of RV for the BIBs are very compatible with the control binder, addition to that the mixing and compaction temperatures are less than of control binder up to 10%.
- The results for the softening point and penetration tests for BIB2 and BIB3 give a good indication of the stiffness of the new blends.
- The FTIR and microscope images showed that the new material formed its own bonds due to the physical and chemical changes in the molecular bonds of the control binder and the absence of separation.

This study shows a successful development of new bio-binder from PKO-p as a replacement material. Additional research need to be done and the behaviour of the new binder in the field need to be evaluated.

5 ACKNOWLEDGEMENT

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The natural frequency measurement of pedestrian bridge using smartphone

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Abstract

Presented in this paper is the use of smartphone to measure the natural frequency of a pedestrian bridge in Ho Chi Minh City. This measurement is compared with other three methods including: theoretical equation, Midas/Civil modelling and vibration measurement using Dynamic Strainmeter SDA-810C/SDA-803C. The Midas/Civil modelling and theoretical equation provides 12-14% and 15-17%, respectively, lower results compare with the other two measurements. These differences are due to the assumptions when producing the theoretical prediction and the modelling of bridge can be different from the actual structures. It is interesting to find that the difference between the measurement from smartphone and the SDA-810C/SDA 03C is only 2.3%, which can be neglected. From these measurements and comparisons, the authors suggest that the smartphone can be used to give a rough approximation of bridge vibration where official measurement methods using proper machines are not applicable.

Keywords: *natural frequency, smartphone measurement, finite element, dynamic strainmeter*

1. Introduction

Pedestrian bridge is structure built for pedestrians and other way of light transportations such as cyclist, animal traffic, etc. The design of pedestrian bridge follows the same principles as for highway bridge with a different load type. Carrying only light load, the pedestrian bridge is light-weight and thus is significantly vulnerable to vibration. In addition to stiffness, damping and mass are key factors in design for dynamic of pedestrian bridge. In design, the natural frequency is of the most important factor. Generally, when the range of natural frequencies lies between 1.25 Hz and 2.3 Hz [1], the vertical and longitudinal vibrations may occur. For lateral vibration, this range is 0.5 Hz to 1.2 Hz. AASHTO standard [2] requires that natural frequencies lower than 3.0 Hz should be avoided in pedestrian bridges.

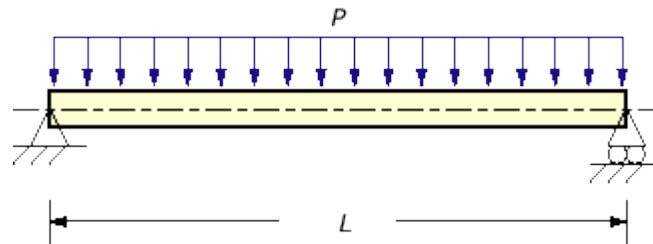


Figure 1 Simply supported beam method

The natural frequency can be approximated using a simplified method. For simply supported beam (Figure 1), the natural frequencies for the 1st and 2nd mode for vertical vibration are, respectively:

$$f_1 = \frac{1}{2\pi} \frac{9.869}{L^2} \sqrt{\frac{EI}{m}} \quad (\text{Eq.1})$$

1)

$$f_2 = \frac{1}{2\pi} \frac{39.478}{L^2} \sqrt{\frac{EI}{m}} \quad (\text{Eq. 2})$$

in which m is mass per meter length of the structure (kg/m); EI is the vertical stiffness (kN.m²)

The use of finite element method in modern bridge design is widely spread with the determination not restricted only to stress and deformation, but also the natural frequencies. By modelling the bridge using Midas/Civil software package [3] and using eigenvalue analysis for natural periods, the frequencies can be obtained.

In reality, the most accurate way to measure the frequencies of a structure would be to use a dynamic strainmeter. This equipment provides dynamic strain for a structure that change with time. By using accelerometer, the acceleration of a structure can be measured from which frequencies can be extracted.

With the development of technology, the smartphone nowadays provide way more functions than general mobile phone. Most of the smartphone can sense the vibration of the structure

and the vibration data can be obtained using particular application. This is another way to study the dynamic properties of the structures.

In this study, the authors adopt four methods stated above to investigate the natural frequencies of a pedestrian bridge. By comparing the results of the four methods, their reliabilities are shown and evaluated.

2. Fundamental frequencies approximation

A pedestrian bridge crossing over Pham Van Dong road at Ho Chi Minh City, Vietnam is investigated. The total length (including platform) of this bridge is 55.7 m, separated into two simply supported spans. The calculated length for each span is 26.7 m. The general span arrangement of the bridge is shown in Figure 2. The typical reinforced concrete box-section of the bridge is presented in Figure 3.

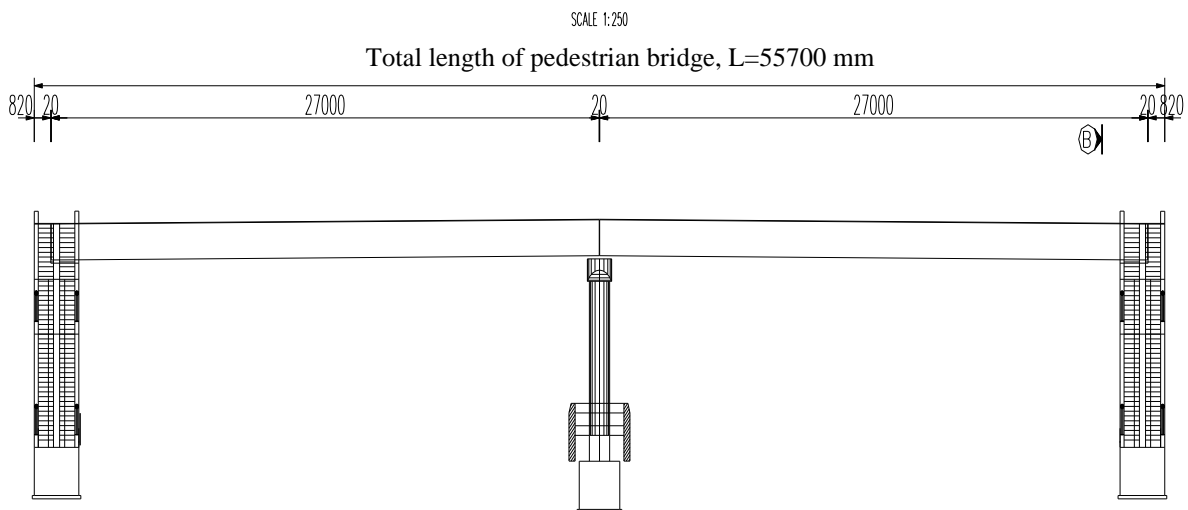


Figure 2 General arrangement of pedestrian bridge

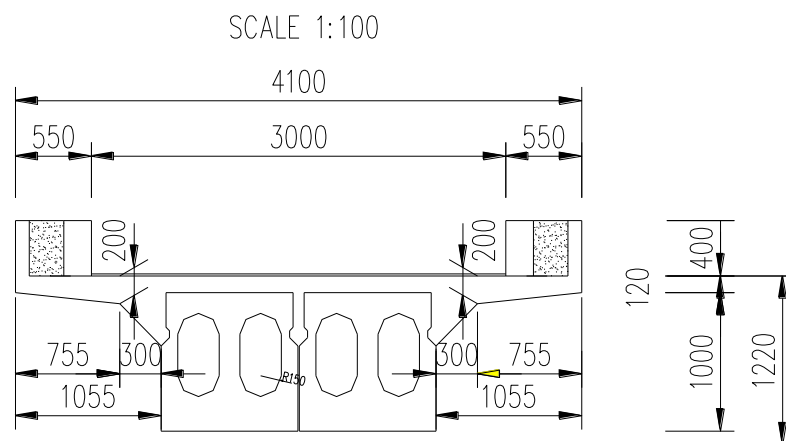


Figure 3 Typical cross-section

By using Eq. 1, in which the mass of the structure is 5100 Kg/m; simply supported span $L = 26.7$ m; the elastic modulus of the structure (assuming it is for concrete only) $E = 30$ GPa; The second moment of area over the vertical axis is $I_x = 2.91 \times 10^5 \text{ mm}^4$.

The 1st mode of natural frequency is:

$$f_1 = \frac{9.869}{2 \times 3.14 \times 26.7^2} \sqrt{\frac{3 \times 10^{10} \times 291000}{5100}} = 2.88(\text{Hz})$$

The 2nd mode of natural frequency is: $f_{12} = \frac{39.478}{2 \times 3.14 \times 26.7^2} \sqrt{\frac{3 \times 10^{10} \times 291000}{5100}} = 11.52(\text{Hz})$

The modelling of pedestrian bridge using Midas/ Civil starts with the input of material properties and the definition of the cross-section. The nodes, elements were then created. The boundary condition is defined as simply supported. The load on the structure for the natural frequencies consideration include only uniformly distribution dead load for self-weight of concrete beam, deck and the wearing surface. There were defined accordingly.

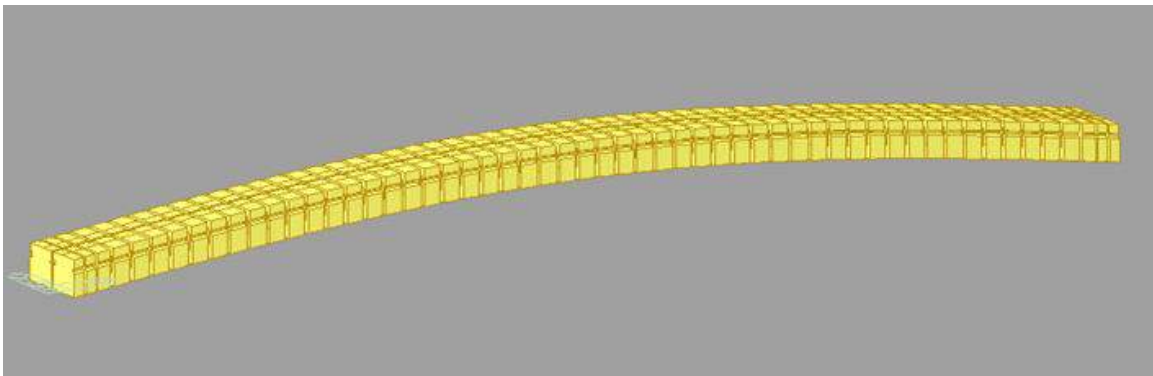


Figure 4 First mode of vertical vibration

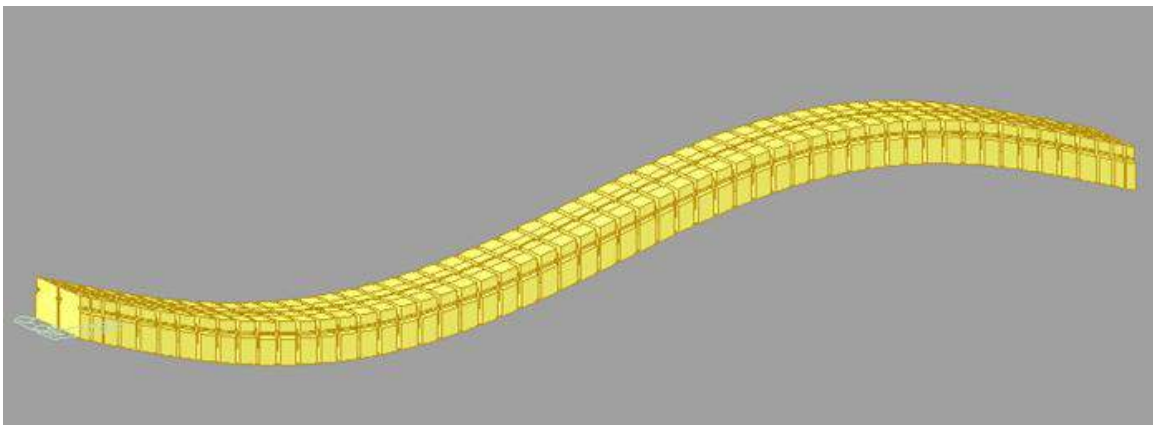


Figure 5 Second mode of vertical vibration

By adopting the eigenvalue analysis, the mode shapes and the periods are found. Figure 4 and Figure 5 show the eigenvalue mode shape for the 1st and 2nd vertical vibration. The periods of 1st mode calculated by Midas/Civil is 0.328 (s) and for 2nd mode, it is 0.083 (s). This is equivalent to the frequencies of 3.05 and 12.05, respectively.

Comparing the approximation using theoretical equation and Finite element analysis, the difference is 5.9% for 1st mode and 4.6% for 2nd mode. This minor difference shows that for simply structure (i.e. simply supported bridge), the theoretical equation can be quite reliable.

3. Fundamental frequencies measurement

The 1st measurement is by using the dynamic strainmeter (Figure 6 (a)). The ARF-20A-T type acceleration transducer (Figure 6 (b)) is placed at mid-length of the span of the bridge. The vibration is induced by a person walks from one end of the span to another. The strainmeter is connected to a computer on which all the measurements for acceleration of structure will be recorded with time [4]. The data is used to obtain the natural frequencies of the structure using the Fourier series transforms [5].

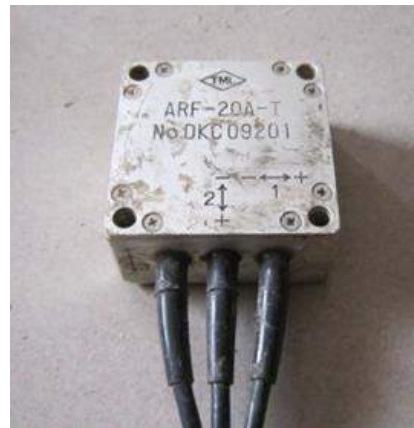


Figure 6 (a) Dynamic strainmeter;

(b) Acceleration transducer

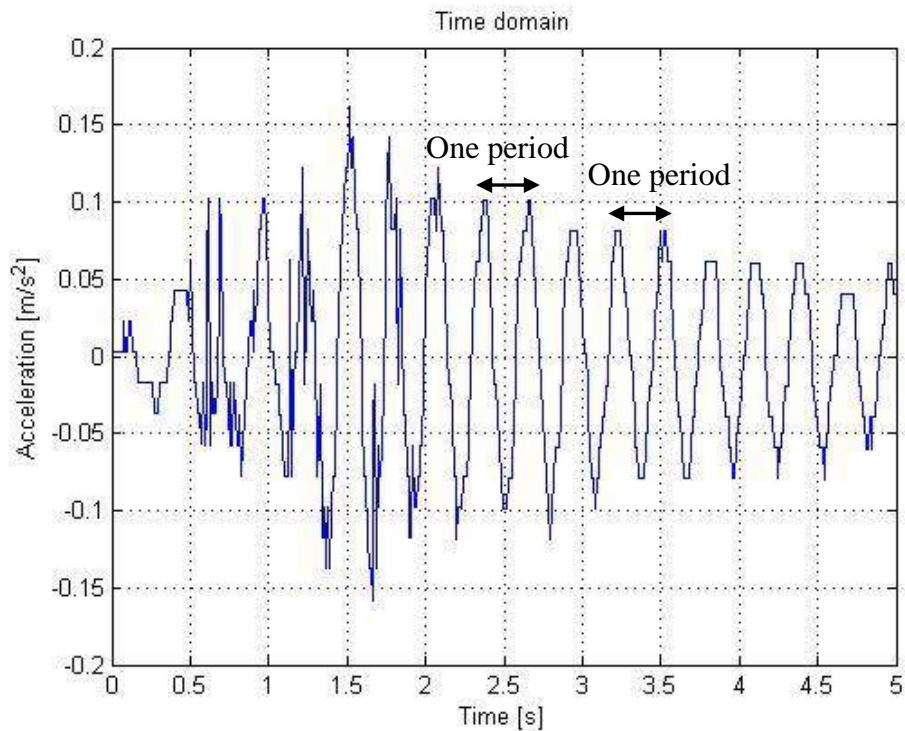


Figure 7 Acceleration with time domain

Figure 7 presents the changes of acceleration with time. From this figure, one can approximate the frequency of the structure by measuring the period T of vibration, from which the frequency is calculated as $f = 1/T$. It is noted that several periods may be found in the graph for different frequencies.

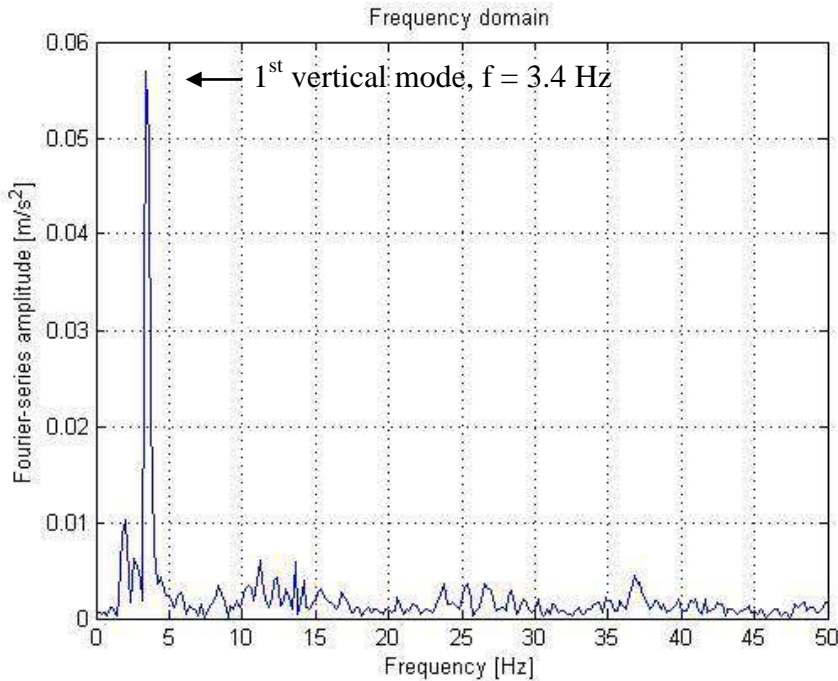


Figure 8 Fourier amplitude spectrum with frequency domain

Figure 8 presents the Fourier amplitude spectrum with frequency domain. Each peak on the graph shows a possible frequency for vertical vibration. For the mid-span measurement, the critical mode would be the 1st vertical mode. From the figure, the 1st vertical vibration mode happens at frequency of 3.4 Hz.

A similar measurement is adopted using application “vibsensor” on the iphone. This software, developed by Now Instrument and Software, Inc (UK), allows user to collect, analyse and export accelerometer data of a vibrated object [6]. By placing the smartphone at the mid-span of the bridge and recording the acceleration of the structure when a person walking from one end of the span and to another, the similar data is obtained.



Figure 9 Measurement of vibration using smartphone

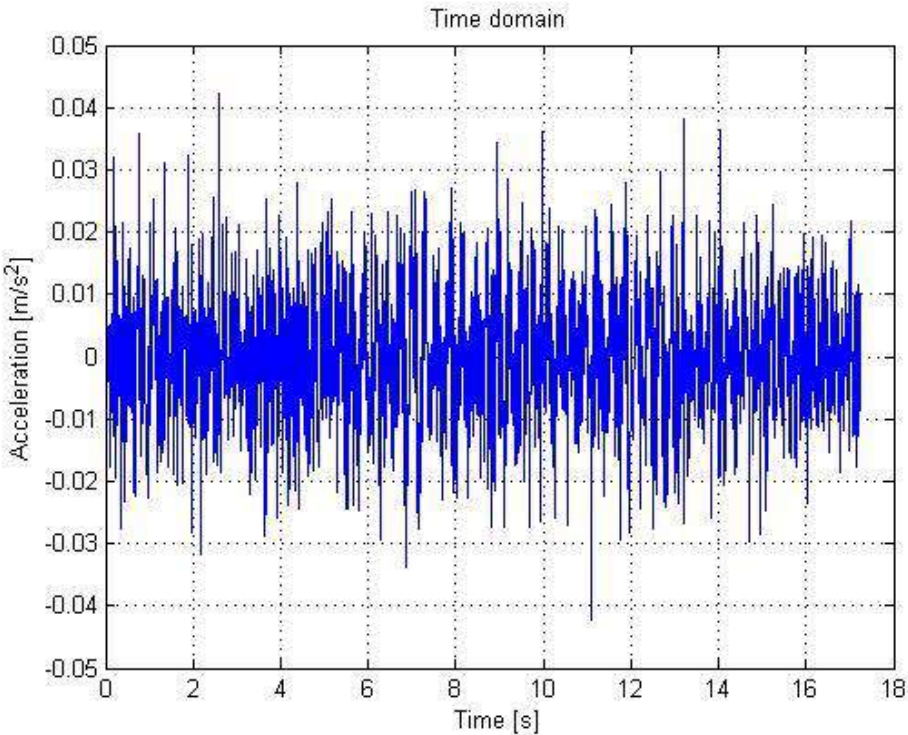


Figure 10 Acceleration vs. time using smartphone

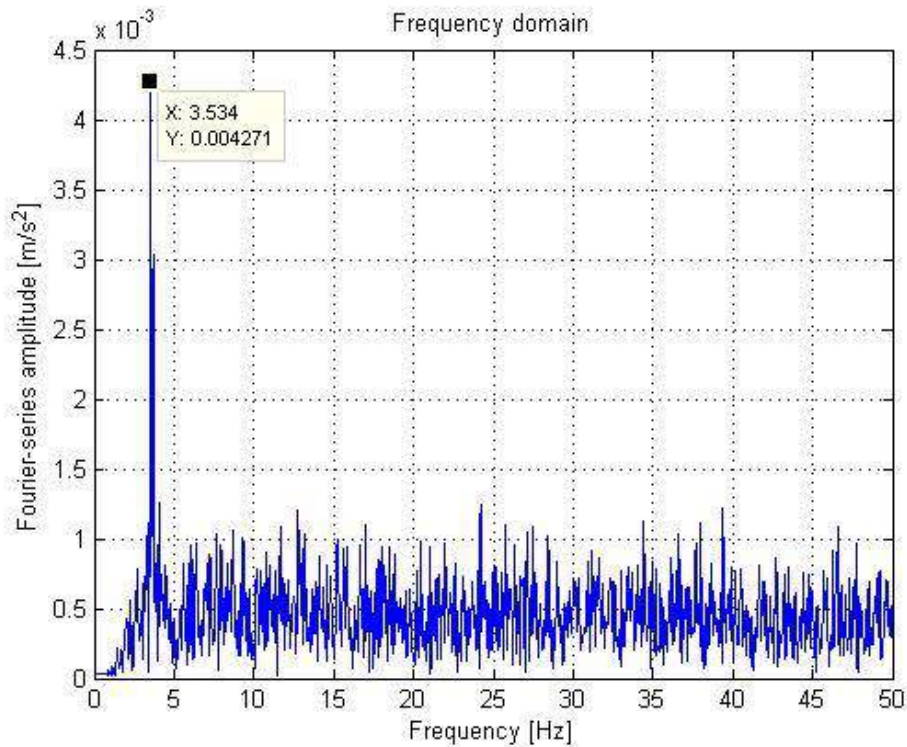


Figure 11 Acceleration with frequency domain using smartphone

Adopting the same technique when analyse the raw data from measurement using smartphone, it is found that the 1st vertical vibration occurs at frequency of 3.48 Hz as shown in Figure 11.

It is seen that two measurements provide similar results for 3.4 Hz and 3.48 Hz at the 1st vertical vibration mode, which is only 2.3 % difference. These measurements have differences from 12-17% when comparing with the 1st two approximation methods. These differences can be partly due to the assumption in theoretical approximations.

Concluding remark

Four different methods have been performed to study the natural frequencies of a pedestrian bridge. It is found that the approximation using theoretical equations and finite element analysis provide similar results for the same bridge. The theoretical equations give the natural frequencies of 2.88 Hz for the 1st vertical mode and 11.52 for the 2nd vertical mode. The finite element analysis with Midas/civil approximates the frequencies of 3.05 and 12.05, for the 1st and 2nd mode, respectively. The differences between two methods are only for 5.9% and 4.6%, which is fairly small.

By using the vibration measuring machine (dynamic strainmeter), the acceleration of the bridge can be obtained at mid-span. By transforming the data to frequency domain using Fourier transformation, the 1st vertical mode of vibration is found at frequency of 3.4 Hz. The same measuring technique is used with the smartphone as measuring machine. It is found that the frequency for the 1st vertical mode is 3.48 Hz. The difference between these two measurements is only for 2.3% showing that the smartphone can be reliable when measuring the vibration of structure. It is recommended that for the preliminary investigation of natural

frequency of pedestrian bridge, the smartphone can be adopted to provide the first impression of the dynamic behaviour of the structure. This method is simple without particular equipment required. It can be convenient when measuring bridges at rural area.

Comparison between theoretical predictions and the measurement at the bridge shows that these differences are between 12 and 17% which is acceptable for the concept design.

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REQUIREMENTS OF AN EFFECTIVE ASSET INFORMATION MANAGEMENT SYSTEM (AIMS) FOR ROAD INFRASTRUCTURE

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Introduction

In the last decades we have witnessed the increasing adoption of Asset Management (AM) in maintaining highway and road assets in road authorities and agencies around the world. There is a paradigm shift from the conventional reactive maintenance approach to the modern AM centric processes and systems. Standards such as PAS55 and ISO55000 have been introduced to set specifications and guidelines for its implementation, compliance and certification.

There is general consensus that AM will bring about numerous benefits to the stakeholders in road assets, primarily the owner and the end road users. Understanding how an AM approach to road maintenance will realise the envisaged benefits is of primary importance to the practitioner.

The source of benefits which AM can derive stems from its strategic and whole lifecycle approach to planning and maintaining assets. AM aims to “*deliver the required levels of service at the minimum Whole Life Cycle cost*” of assets. The focus of AM is based on:

- ❖ A long term or *whole life-cycle* perspective.
- ❖ Balancing Levels of Service of Assets with the cost of maintenance and operations.
- ❖ Maximising the value or benefit of investments in maintenance.

A *whole life-cycle* approach to managing assets necessitates the extension of the planning horizon from the short and medium terms to the long or strategic perspective. In AM terminology, the long or strategic periods denotes the expected useful lifespan of the road asset. This means the design or required service life span of assets, which is often more than 10 years in road pavement assets.

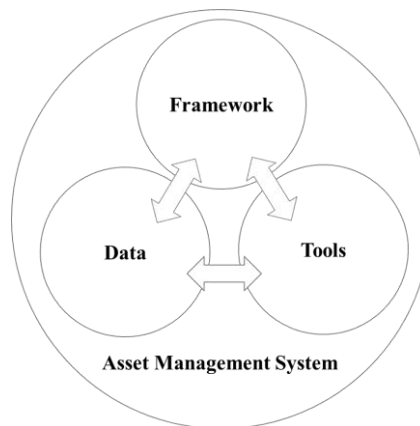
To plan and sustain the required level of service of road assets over the whole life-cycle, the capability to assess the condition, model the deteriorations and planning for the most beneficial impact for each dollar investment spent sets AM apart from the ‘find and fix’ (reactive approach) of conventional Maintenance Management.

AM aims to optimise maintenance strategies and maximise benefit-costs over time. Unlike Maintenance Management, AM aims to sustain the required level of service, as defined under considerations of operational and budgetary constraints by prescribing when, where and what work is to be carried out that would maximise benefit-cost. In contrast, Maintenance Management focus on restoring serviceability of assets to its original design capacity or to the highest level possible, without attempting to assess benefit-cost of alternatives or options.

An Asset Information Management System (AIMS) is the IT component of an enclosing Asset Management System (AMS). It must be aligned with and support the objectives and the whole lifecycle approach of AM. The adoption of work processes that focus on industry’s best practices in AM and the institutionalisation of these practices among maintenance personnel would be the pre-requisite of the successful implementation of an effective AIMS. AIMS must be more than just a database of data and information, it shall support the assessment, analysis, forecasting and planning of maintenance works to achieve organisational Asset Management objectives by sustaining the required levels of service over the asset’s life cycle.

Components of an Asset Management System

An Asset Management System (AMS) should be distinguished from the Asset Information Management System (AIMS), the former is a larger eco-sphere which is an integration of 3 sub-components:



a. Framework

The Asset Management Framework (AMF) is a foundation of AMS. It is essentially a guiding blueprint on the implementation of Asset Management, tailored to the environment of which the implementing organisation operates in. It documents the objectives and policies of the Asset Management process, which may include an Asset Management Plan (AMP), which define the long term plan in attaining the above objectives. It also defines the supporting organisational structure and the roles and responsibilities of personnel in the AM supply chain. It specifies the approach, methodology, data requirements, skills sets and functionalities of an Asset Information Management System (AIMS).

b. Data

AM is a data knowledge process and the availability and quality of data has a great impact on the outcome of the whole AM process. Data can be classified into inventory, condition, work related, traffic/loading demand, economic/cost, environmental and constraints. Data need to be collected, analysed, modelled and reported in-lined with the AM process to produce the required output. Data will need to be stored in a database that is structured to facilitate the data access and usage. A robust data referencing system, such as a Location Referencing System (LRS) for linear assets needs to be developed for effective and efficient data retrieval and correlations.

c. Tool

The AM tools include data collection, processing and analysis systems such as a computerised Asset Information Management Systems (AIMS). The tools needs to able to make use of the data contained in the database to support the AM process as defined in the AMF. The output from AIMS will be an inventory register, condition and usage demand reports, trending and statistical analysis, work effects and condition modelling, work and cost optimisation, AM strategies and work programmes, budgeting and expenditure, performance and KPI reports.

All the above components of AMS are interrelated and needs to be aligned and integrated as an effective overall system. Would be AMS implementers will need to focus on the requirements of all 3 components and their relationships. A phased and structured approach should be adopted to evolve iteratively the requirements of all three components and allow time for organisational assimilation.

Key Requirements of an AIMS

An Asset Information Management System is an embodiment of the organisational processes in Asset Management. It needs to be provide functionalities beyond the simple storage and reporting of data in its database. It needs to support the AM process in line with the AMF. The following key requirements should be on top of the list of would be AIMS developers and implementers:

A. Supporting the full Asset Management Process

The following diagram shows a generic AM process and the various tools that can be included in an AIMS for each stage of the management process:

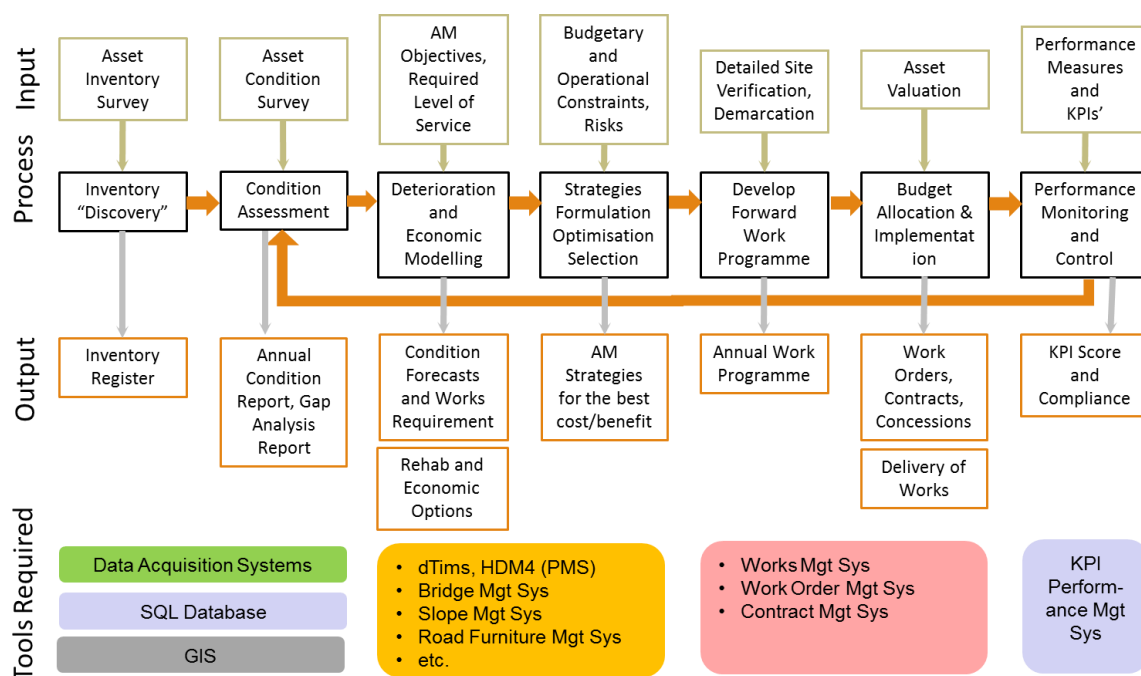


Figure 1 AM Processes and Supporting AM Tools

The 6 AM processes as described above can be grouped into the generalised functional layers:

- Data Management – inventory “discovery” and condition assessment.
- Analysis and Decision Support – deterioration/economic modelling, work requirements, performance gap analysis, prioritisation of work, strategies formulation and selection.
- Works Management – development of forward work programmes, budget allocation, work implementation.
- Performance Management – monitoring and controlling the performance of assets, operations and management.

Each of the above functional groups of AM processes can be supported by the various software tools and applications as shown above. Depending the actual application requirements of the implementing organisations, the examples given above may not be exhaustive or not essential. Some of these tools can be obtained from commercially available software packages while

others needs to be developed to specific requirements. Whatever the case maybe, it should be noted that all these sub-component systems will need to be integrated, if not at the system level, at least at the data level. This is to facilitate data and information to flow the component systems without much requirement for data transformation and reformatting.

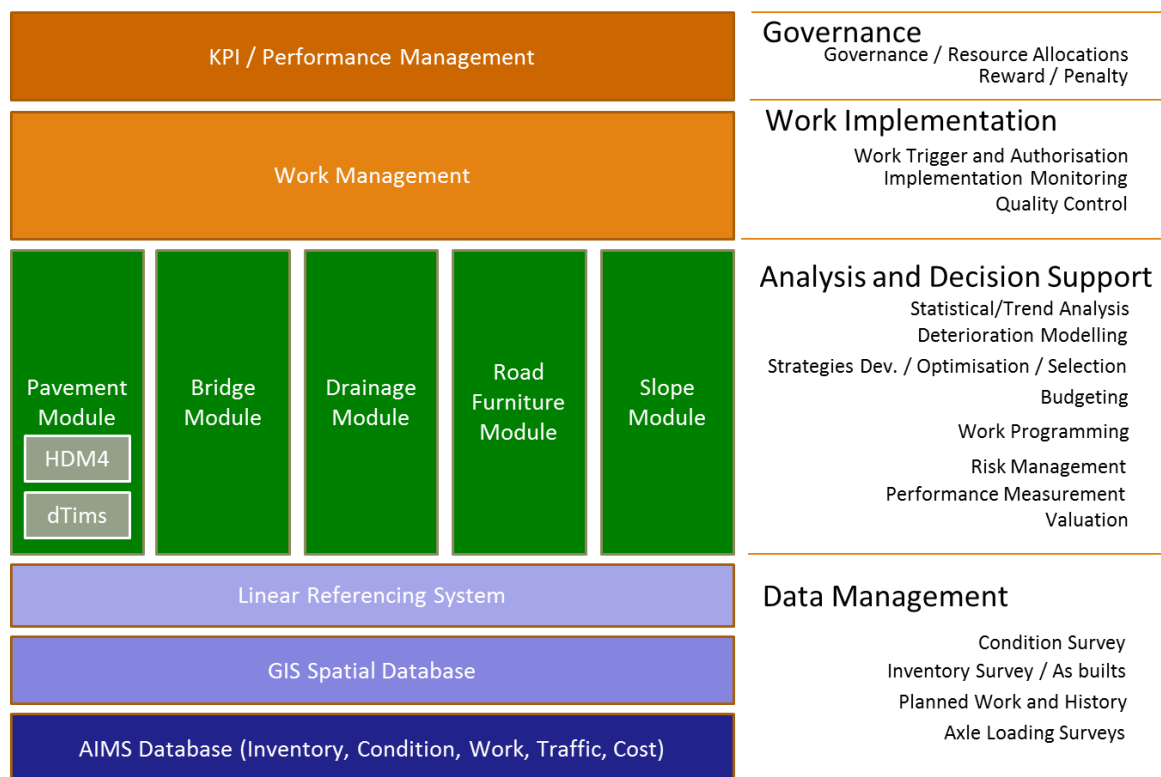


Figure 2 Functional Layers of an Integrated AIMS

❖ Data Management Level

The component systems at the Data Management layer manages the collection, compilation, referencing, storage and retrieval of data. In a corporate multi-user environment, these data will be stored in a relational SQL database. As the AIMS database will form the data and knowledge server to other functional layers, the robustness, availability, scalability and security features of the database software platform must be carefully considered.

The design of the database structure (schema) has a large impact on the performance of the system. In designing the database structure, the following factors must be considered:

- Multi-referencing of assets and events – by unique asset ID (e.g. bridge), by KM chainage referencing (e.g. work location from KM2.20 to KM2.53) and by coordinates system (e.g. GPS coordinates), the database will be able to retrieve data based on any one of the above methods.
- Time series of datasets – database will need to be able to record current information and the ability to relate to historic and future information for any asset inventory. This is crucial as data collected in different time continuum is used to develop trends and projections in the Analysis and Decision Support level (described later).

- c. Logical Integrity of data – as the information of any particular inventory will be assessed and updated from multiple sources, the database will need to provide automated data checks (e.g. there shall not be duplicate recording of same physical asset inventory).

AM data to be stored in the database can be classified as textual or spatial data. Textual data describes data that does not relate to location (e.g. IRI value). Spatial data describes information that depicts locations (e.g. alignment of a road).

These two types of data, though can be stored into a common AIMS database, will however need two different type of software systems to display and managed. While textual data can be easily processed and managed by a database application modules, spatial data is best managed by a GIS (e.g. ESRI's ArcGIS) software. The challenge is to integrate the two systems.

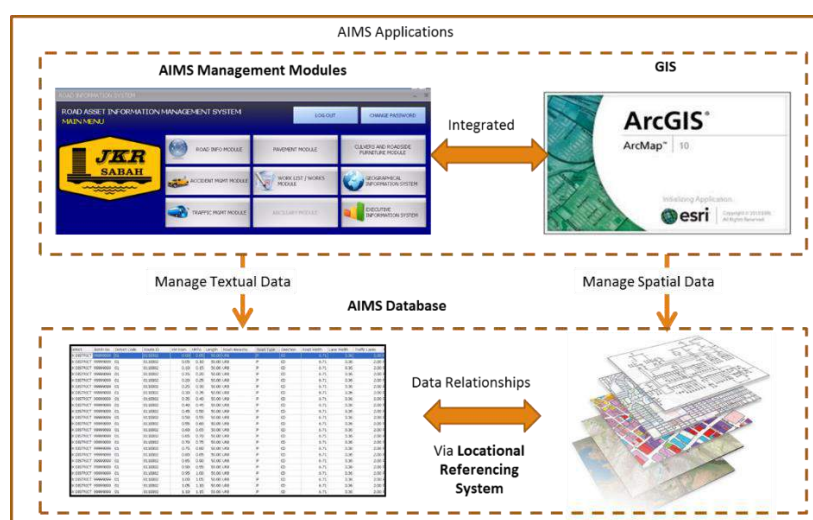


Figure 3 Integration of GIS and AIMS

At the database level both the GIS spatial database and the AIMS database resides as data in tables of the same database platform (e.g. MS SQL Server or Oracle). At the system level, these tables can be linked by unique key values in table fields. In linearly segmented data (such as rutting on pavement from KM5.2 to KM5.9 on Fifth Avenue), GIS provides a functionality to link the textual information (e.g. rutting value of 5mm) to the right location on the spatial data (e.g. alignment of the road).

This linking function requires the development of a Location Referencing System (LRS) in the database. An LRS is a feature class that describes location of objects or events on a linearly measured line with linear measurements (Kilometer value) from a reference station. A LRS for AIMS implementation should be a standard across the organisation to record the location of linear assets.

At the application level, both GIS and the AIMS modules would be integrated by a common web browser interface.

Data are collected from data acquisition systems such as laser-profiler, falling-weight-deflectometer, field GPS data collection devices, coring and lab testing, Lidar, GPS survey, weigh-in-motion, traffic counting, aerial surveillance and remote sensing. These data, are typically voluminous and tedious to enter manually into the system. AIMS will need to have

an automated data loading and logic checking facilities to import the data into the database and setup the required location referencing for the data.

❖ **Analysis and Decision Support Level**

The objective of application modules in this level is to turn data in the AIMS database into useful information for operations support and management decisions. This is carried out by the AM modules customised to manage the different classes of road asset including pavement, bridge, drainage (including culverts), road furniture, road corridor, tunnels and even facilities/building. The output from this level of application modules shall include:

- a. Gap analysis – current level of asset performance against required level of service.
- b. Work requirement analysis – what are the repairs/rehabilitation required to close the performance gap.
- c. Developing maintenance options and strategies.
- d. Model and predict the deterioration of assets under various projected traffic loading or usage and maintenance options / strategies.
- e. Applying constraints and carry out economic optimisation based on maximising benefit-cost over whole lifecycle or analysis period.
- f. Selection of AM strategy and development of funding requirements.

Processed data in the previous stage of the AM process is analysed for work/rehabilitation requirements and treatment options are generated. The conditions and level of service of assets over the whole lifecycle is analysed and predicted based on the various treatment options and scenarios. Budgetary and maintenance policy constraints are subsequently applied and the treatment option that offers the best benefit cost is selected and formulated in an optimised AM strategy, including the corresponding required funding. The AM strategy forms the basis for the development of a Forward Work Program which packages work requirements into projects that can be implemented.

Pavement Management module

The common tools used in analysing pavement for deriving optimised maintenance investment strategies are HDM4 and dTIMS. dTIMS is a full featured road asset management system that is commonly used in Australia, New Zealand, Canada and other developed Western Countries. The cost and skills required in the implementation of dTIMS are significantly higher than an all-in package solutions such as HDM4, which offers a lower start-up barrier to dTIMS. HDM4 is a standalone analysis tool that is developed by HDM4Global. sponsored by World Bank, it is used primarily in developing countries including Malaysia. HDM4 should feature as an 'analysis engine' that is integrated to the overall AIMS.

HDM4 is a very versatile and powerful analysis engine. However, the preparation of input data and the extraction of information output from HDM4 is tedious and poses a challenge to the less technically inclined asset manager. The process of preparing data input into HDM4 for a large road network, if done manually, is labour intensive and error prone. A 'pre-processor' is normally part of the pavement management module in AIMS to automate/facilitate the processing of data from the AIMS database into HDM4 for analysis. The primary function of the pre-processor it to reduce the intensity and volume of data that is required to be analysed by carrying out a process termed 'dynamic segmentation'. This essentially turns raw data collected in small interval cells of the pavement into

homogenous continuous sections of the pavement, termed ‘treatment lengths’ for deterioration and works optimisation analysis in HDM4.

On completion of analysis in HDM4, the results of analysis is further processed to generate the required AM reporting.

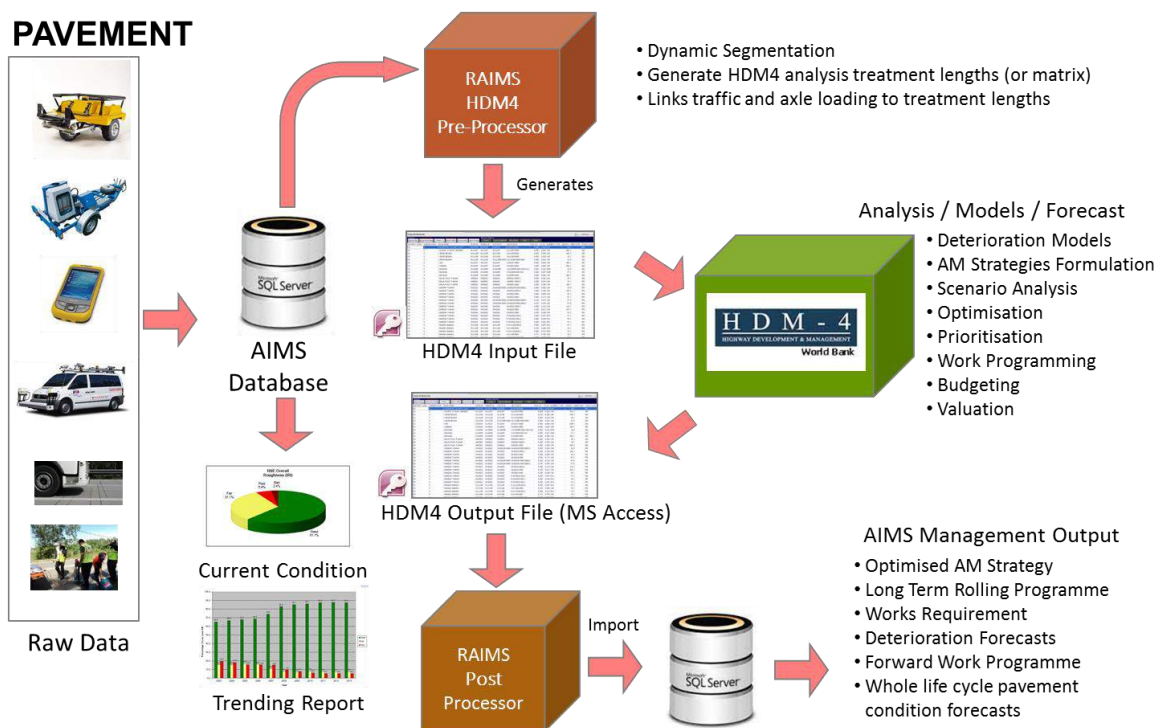


Figure 4 Schematic Data Processing Flow in AIMS Pavement Management Module

Bridge Management module

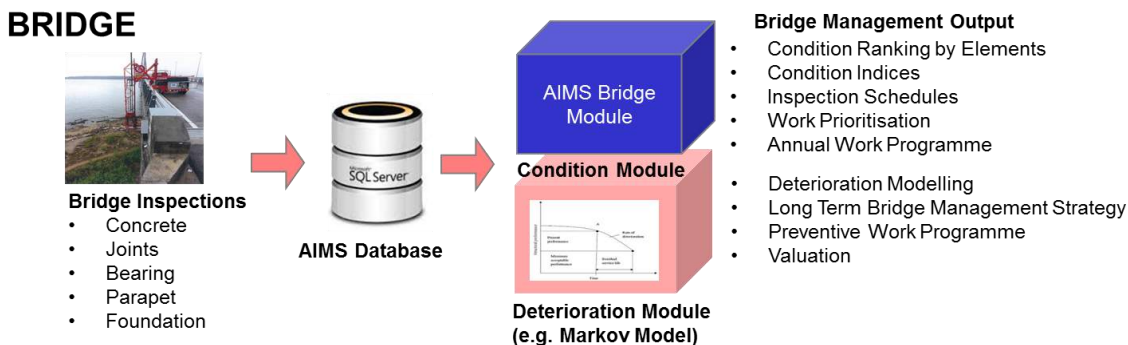


Figure 5 Schematic Data Processing Flow in AIMS Bridge Management Module

The primary objective of bridge management is to ensure safety of road users. In order to achieve the objective, earliest signs of deterioration or defects have to be captured through scheduled inspections and special investigation/testing carried out for each of the critical components of the bridge. The condition of all components of the bridge is assessed for severity and extend of defect. Ranking for condition and urgency for repair for each of the component is applied and an overall bridge index is calculated based on summation of weighted scores of these components. The functionality of the Bridge Management module of AIMS will have to be aligned with this data collection and condition assessment technique.

In a more advance application of Bridge Asset Management, a modelling or deterioration ‘engine’ can be developed in the module to provide the forecasting functionality to the module. This will enhance the long term preventive maintenance capability of the system.

The output from the Bridge Management module will be bridge repair/rehabilitation work programmes with a prioritised work lists according to urgency to restore safety and functionality of the asset. The module will have the capability to schedule inspection based on criticality of the component to the integrity and safety of the bridge.

Slope Management module

SLOPE

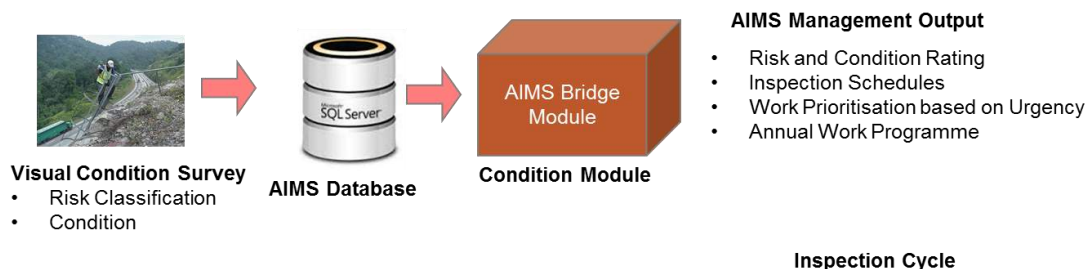


Figure 6 Schematic Data Processing Flow in AIMS Slope Management Module

The approach of Slope Management module for AMS can be based on a risk management approach. Slope assets are ranked according to risk of failure happening and consequential risk to road user. The risk of failure is dependent on the physical inventory attribute of the slope (inclination, height, type etc.) and environmental factors. These data will be part of the AIMS database that is to be extracted for evaluation of risk ranking. To inculcate a preventive maintenance approach to slope management, the frequency of inspection of slope assets is varied based on slope risk category as per the example below:

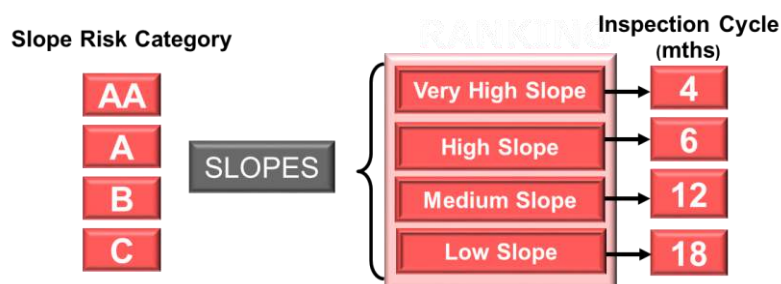


Figure 7 Assigning Slope Inspection Frequency based on Slope Risk Ranking

As there are numerous environmental factors specific to sites that will affect the deterioration of slope stability, a generic slope failure model is not normally used in AIMS to forecast future conditions and rehabilitation requirements.

Drainage and Road Furniture module

The approach to drainage assets and road furniture module is similar in that it is based on visual (or instrumented) survey of condition of assets and evaluating the following indices:

- a. Structural integrity index – the degree to which the asset is structurally sound.
- b. Serviceability index – the level of service of the assets compared to its design level.

- c. Urgency for work – the urgency to repair or maintain based on road user safety concerns or consequential damages due to unrepair.

DRAINAGE & ROAD FURNITURE

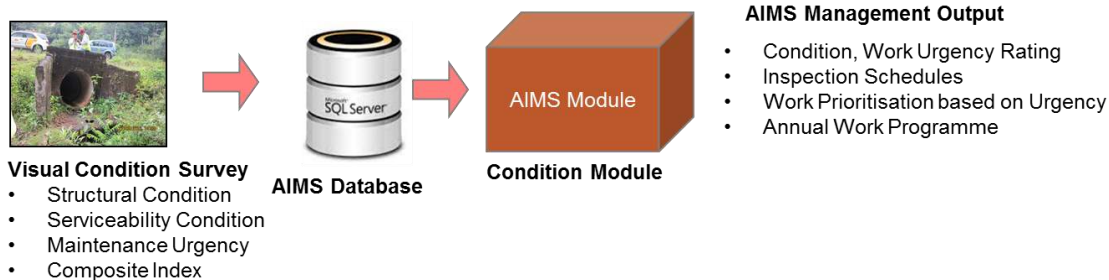


Figure 8 Schematic Data Processing Flow in AIMS Drainage and Road Furniture Management Modules

An overall index is developed based on weightages being applied to each of the above indices. Works are prioritised based on ranking of the overall index.

❖ Work Management Level

Work Management process involves the development of Forward Work programmes from the asset management strategies. These are planned and scheduled works triggered as a result of analysis carried out in the previous stage to conform to a selected asset management strategy. In addition, this stage of process will include ad-hoc or unscheduled works triggered from customer complaints, events happening such as damages caused by accidents, emergency responses such as floods or work due to significant importance (protocol). The common application tools are work order / maintenance contract management systems.

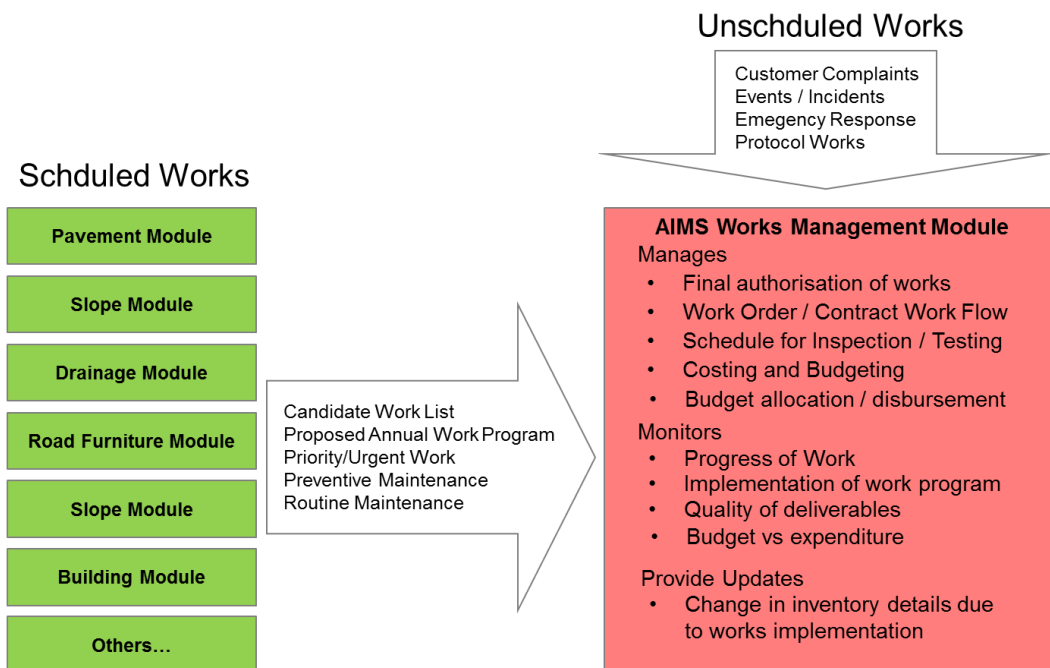


Figure 9 Schematic Data Processing Flow in AIMS Work Management module

The modules in the Work Management level of AIMS will support the following processes:

Management processes:

- a. Compilation of prioritised candidate work lists from asset management modules.
- b. Provides estimates on cost of works from Schedule of Rates or from tender/quotes.
- c. Manage and control the authorisation of work flow.
- d. Generation of work orders / specification of works / work documents.
- e. Manage budget allocations and disbursement.

Monitoring processes:

- a. Project execution, status reporting, issues management.
- b. Implementation of authorised work programmes.
- c. Budget and expenditure.

Updating process:

- a. Provide feedback update to AIMS database on inventory changes.
- b. Records work history.

❖ Performance Management Level

Performance Management is an integral part of the AM process. Performance Management provides the feedback required to the AMS for continual adjustment and improvement of asset management strategies, work programmes, condition assessment technique, work-triggering mechanism, analysis and modelling techniques, budgeting and funding of works.

The objective of the AIMS module is to support the Performance Management process by providing a tool to monitor the performance measures, or Key Performance Indices (KPI) as defined in the AMF.




Category	DURABILITY 	OPERATIONS 	MANAGEMENT 
Description	Determines the duration of service life of assets. Preservation of value of assets.	Determines the level of service (serviceability) provided to road users. Affects road users cost.	Determines the efficiency and effectiveness of the asset manager
Typical Indicator	<ul style="list-style-type: none"> • Structural Defects • Residual Life 	<ul style="list-style-type: none"> • Safety • Availability • Comfort • Convenience 	<ul style="list-style-type: none"> • Data Management • Reporting • Public Service • Response time to events
Measure	<ul style="list-style-type: none"> • Roughness • Cracking • Deflections • Rutting • Condition Index • Texture 	<ul style="list-style-type: none"> • Deformations • Siltation • Verge and Shoulder • Markings • Lighting • Traffic Management • Risk Ranking (slope) 	<ul style="list-style-type: none"> • Response Time • Data Update • Reporting • Emergency • Monitoring + Audit • Auditing

Figure 10 Categories of Performance Measure or KPIs

KPIs are measured as dashboards on a computerised AIMS. An example of a KPI dashboard is as shown in Figure 11.



Figure 11 Example of a AIMS Performance Management Dashboard

Dashboards displays are for management decisions support. It is used to monitor the performance of physical assets and the operation efficiency and management effectiveness of the Asset Manager. The KPIs are high levels summarised representations of required levels of services as defined in the AMF. The definition of KPIs and the methodology to derive the achieved value should be defined and agreed upon between stakeholders and documented in the AMF.

The Performance Management module in AIMS will need to be able to mine and extract performance related data from the AIMS database, compute according to the specified KPI computation methodology and display in the Performance Dashboard. In a more advanced version of the dashboard, drill down from high level KPIs to derive at the root cause / issues that contributed to the achieved KPI level can be incorporated.

B. System Architecture

AIMS should be a collaborative platform for all stakeholders in the AM delivery chain. The asset owner (authorities, agencies) and/or its appointed asset manager should assume ownership of the AIMS database and application system. They will be responsible for the operations and maintenance of the system. Data collection contractors, who carry out annual or periodic surveys of asset inventory and conditions will provide data in batches to the AIMS database manager and GIS staff to be input into the AIMS database. Data may also be updated directly into the AIMS database by the asset owner/managers own staff, these are normally visual condition assessment data (carried for the purpose of performance auditing) and works related information. Maintenance contractors or concessionaires will need to provide input in terms of work requirements (notification of defects) and progress information into AIMS. The asset

manager will carry out analysis on the data that is verified and uploaded by the database/GIS manager and produce the required analysis and reporting deliverables.

AIMS will need to be established on an internet/cloud based platform to enable the collaborative nature of AIMS among stakeholders as shown in Figure 12.

The system architecture of AIMS shall be a configuration that promotes:

- a. Sharing of AM related data within the stakeholders sphere

A networked or connected system architecture enables assessing and exchange of data and information among stakeholders. With the advancement in internet/web technology and processing capability of mobile devices, AIMS will be a common platform for sharing of vital information for the effective management of infrastructure assets.

- b. Collaborative work flow between owners, managers and works implementers

The process of AM needs to be conform to a defined and documented approach. This process shall be part of the organisation’s standard operating procedure in AM. All staff involved in the asset management process will have roles and responsibilities defined in information and knowledge management using AIMS. The use of database and internet technology enables more effective and efficient collaborative effort be made.

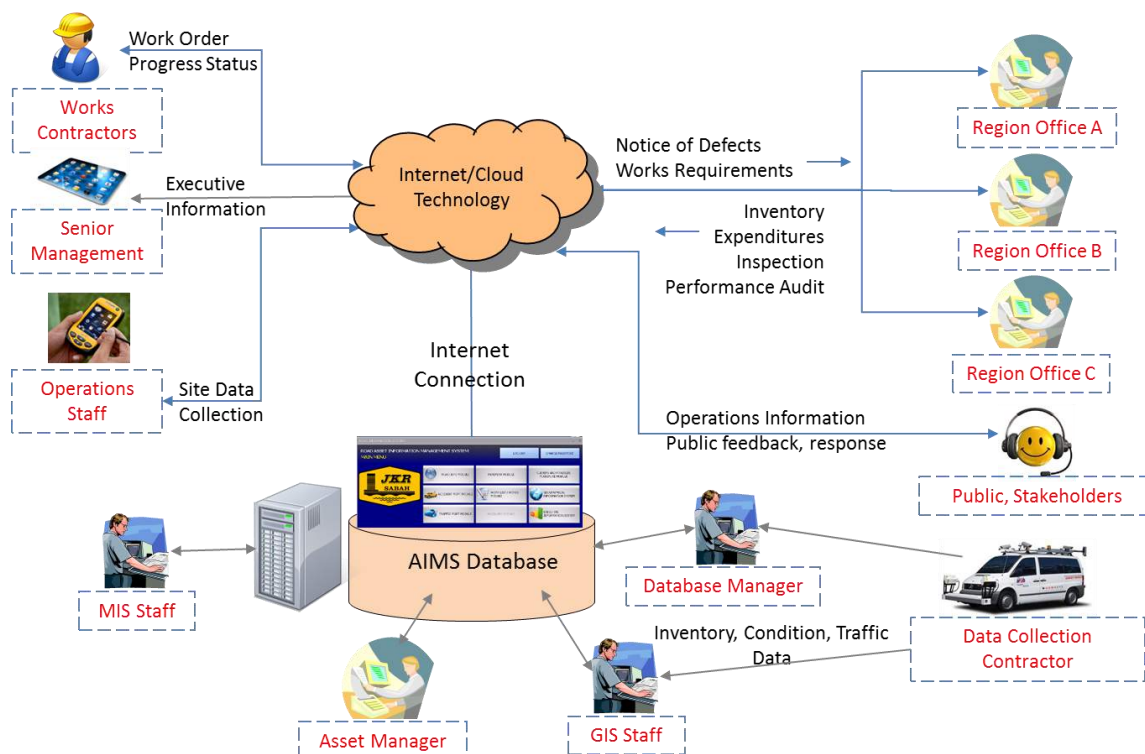


Figure 12 AIMS System Architecture to Enhance Accessibility and Usage

Conclusion

The success of implementation of an AIMS hinges on the understanding that AIMS' sole purpose is to support the Asset Management process. An implementing organisation without an Asset Management centric work processes will not be able to provide the framework for AIMS development and implementation. In addition, AIMS depends on quality data to generate the knowledge for the most cost effective decision in maintenance investments or spending. For AIMS implementation to be successful, funding for data collection should not only be required for the initial development stage, but operations and maintenance of AIMS in the long run. There are numerous cases of failed AIMS because there were insufficient funding for data collection and updating of the AIMS.

Organisations implementing AIMS will need to have a top down approach to its implementation. Senior management must understand and subscribe to the AM process and commit to implementing not just a computerised AIMS but an AMS framework as a whole. Funding for the continued development, adjustments, enhancement to the overall AMS system underpins the effectiveness and sustainability of the system. A champion from the senior management ranks should be identified and provide the drive and change impetus required in its implementation. The champion will be assisted by a knowledgeable industry implementer in providing advice in industry's best practices and pitfalls to avoid.

Organisation will need to adopt industry best practice in Asset Management and have all stakeholders (including contractors) to understand and subscribe to an accepted Asset Management Framework, which covers the following area:

- Objectives, KPIs, performance measures and required levels of service.
- Asset management process to be institutionalised.
- Data collection, processing and reporting requirements.
- Methodology for forecasting, modelling and optimisation.
- Establishment of a performance monitoring and management system.

Understanding that AIMS will be more than just an information system, it will need to assist the asset manager in all aspects of AM which includes:

- Condition assessment and gap analysis
- Works Planning
- Condition and economic forecasting
- Costing, budgeting and valuation
- Performance Monitoring
- Risk Management

All relevant stakeholders in the AM supply chain must be involved in the development, operations and usage of AIMS. AIMS will need to be developed on a common collaborative platform that all relevant stakeholders can participate in providing inputs and the knowledge support in return from AIMS.

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PAPER TITLE	Cost Analysis of Fiber-Reinforced Pavements		
TRACK			
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<p>KEYWORDS:</p> <p>Synthetic Fibers, Asphalt Concrete, Pavement Performance, Cost Analysis</p>
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<p>ABSTRACT:</p> <p>Synthetic fibers have recognized as a cost effective strategy when added to paving materials around the world. Fiber based asphalt pavement technologies generally require a larger initial investment than conventional asphalt concrete. However, benefits in long-term performance reduce the overall cost of the project when examined over its entire life cycle. The Mechanistic Empirical Pavement Design Guide (MEPDG) simulations showed that using the fiber-reinforced asphalt concrete (FRAC) significantly reduces the rehabilitation activities especially to mitigate alligator cracking such as crack sealing and patching when compared to conventional pavement. A Life Cycle Cost Analysis (LCCA) shows a nominal present net worth savings of approximately 17% with the FORTA fiber product. The analysis described in this paper shows that these savings can result in a dollar value difference of approximately \$35,000 per mile/lane over a 50 year analysis cycle or \$20,509 over a 40 year analysis cycle.</p>

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1. INTRODUCTION

Fibers can be used to improve the properties of the Asphalt Concrete (AC) mixtures through spatial networking, absorption and adhesion of asphalt. The first documented evidence of fiber reinforcement in asphalt concrete (AC) occurred in the early 1950s as a strategy to prevent reflective cracking in an asphalt concrete overlay (1). The current state of knowledge with respect to fiber-reinforcement in AC materials has made favorable conclusions regarding the use of fiber reinforcement in AC. In dense graded and open graded asphalt concrete mixtures, fibers have been shown to: alter the viscoelasticity (2); increase dynamic modulus and decrease rutting (3-9); improve moisture susceptibility (10); reduce reflective, thermal, and fatigue cracking (11-15); and enhance the overall mixture durability (16-18). To date, several fiber types have been used in AC mixtures and are categorized mainly as natural/non-synthetic or synthetic. The use of natural and non-synthetic fibers has provided more inconsistent results due in large part to their degradation under environmental exposure and high temperatures (19-21). Due to the relatively small quantities needed, the cost of using synthetic fibers in AC mixtures is normally low compared to the expected performance improvements if properly distributed within the mix. The most commonly used synthetic fibers are polymer fibers which includes polyester, nylon, polyethylene, polypropylene, aramid, and combinations thereof. Different polymers have different melt points, which need to be considered when adding fibers to hot mix asphalt. Reportedly, aramid fibers contract at high temperatures, which helps resist pavement deformation (3-5). The reinforcing capability of synthetic polymer fibers has been proven in many laboratory and field studies as promising and safe reinforcing elements (3-5, 22-24).

Tapkin showed a positive effect for the Fiber-Reinforced Asphalt Concrete (FRAC) using polypropylene fiber on Marshall Stability, flow values, prolonged fatigue life, resistance to rutting and reflective cracking (25). Wu et al. examined the dynamic modulus of three fiber-modified asphalt mixtures: cellulose, polyester and mineral fibers at dosages of 0.3%, 0.3%, and 0.4% respectively. The experimental results showed that fiber-modified asphalt mixtures had higher dynamic modulus compared with the control mixture (6). Qadir investigated the rutting susceptibility of FRAC mixture containing polypropylene fibers using the Wheel Tracking Device. The samples were tested at four temperatures; 40°C, 50°C, 55°C and 60°C. The polypropylene fibers were found to increase the Marshall stability by almost 25% and resist rutting more effectively at elevated temperatures (26). Tapkin reported that the addition of polypropylene fibers improves the behavior of the FRAC specimens by increasing the repeated creep loading lives by 5–12 times compared to control specimens (27). Research performed in Mexico and Texas has shown that the addition of polyester fibers in asphalt concrete pavements reduces reflective cracking (28, 29). Ma et al. 2006 compared the laboratory behavior of polyester FRAC to no-fiber-reinforced asphalt mixture using laboratory performance tests including high-temperature stabilization, low-temperature crack resistance, and fatigue resistance. The study indicated that polyester FRAC mixture has eminent pavement performance and economic benefits as expressway's surface layer compared to no-fiber asphalt mixture (30). However several studies have clearly showed the improved performance of AC mixtures using synthetic fibers, only limited studies are available to show their cost effectiveness.

Since 2006, Arizona State University (ASU) has been engaged in a research program to evaluate the performance benefits of synthetic polymer fibers. The fibers are a proprietary blend of collated fibrillated polypropylene and aramid providing a three dimensional reinforcement to the Hot Mix Asphalt (HMA). The polypropylene fibers are chemically inert, non-corrosive, and non-absorbent; whereas the aramid fibers have a high tensile strength, non-corrosive and have resistance to high temperatures. The physical characteristics of the fibers have been reported in other publications (3). The performance of the FRAC mixtures using the polypropylene-aramid fibers blend is assessed at ASU and other pavement laboratories using advanced material characterization tests for mainly stiffness, permanent deformation, and cracking evaluation. More details on the laboratory test results of the FRAC mixtures from several field and laboratory projects can be found elsewhere (31).

2. OBJECTIVES

The main objective of this paper is to investigate the cost effectiveness of using the polypropylene-aramid fiber blend with AC mixtures based on laboratory test results of 12 different mixtures tested mainly at ASU and other national and international pavement laboratories.

3. SUMMARY OF FRAC PROJECTS, TEST METHODS, AND RESULTS

Polypropylene-aramid blend fibers are a performance enhancing technology that has been used to improve the resistance of AC mixtures to permanent deformation and cracking. The technological capabilities of these fibers have been demonstrated in multiple laboratory and field studies over the years. This study includes twelve different projects from the United States and abroad where detailed performance assessment has been carried out. For many of these projects, AC mixtures were sampled during construction, transported to the laboratory, and evaluated using standard and advanced experimentation. Only one study, ASU Fiber Dosage Study, used exclusively laboratory mixtures and it was conducted to systematically evaluate parameters that could not be easily studied with field mixtures. The twelve projects considered in this study are listed below:

1. FORTA Boeing Project, Arizona, USA
2. FORTA Evergreen Project, Arizona, USA
3. FORTA Jackson Hole Airport Project, Wyoming, USA
4. FORTA Cranberry Township Project, Pennsylvania, USA
5. FORTA Montreal Project 1, Canada
6. FORTA Montreal Project 2, Canada
7. FORTA South Dakota Project, USA
8. FORTA Middle East Study, Iran
9. FORTA New Jersey Study, USA
10. FORTA Czech Republic Project, Czech Republic
11. FORTA University of Illinois Study, Illinois, USA
12. ASU Fiber Dosage Study, Arizona, USA

The materials studied in these projects represent different aggregate and asphalt types, gradations, contents (air void, aggregate, and asphalt), climate region, and mixing plant type. Table 1 includes a summary of test methods conducted mainly on AC mixtures from the 12 projects to assess stiffness, fatigue cracking, and rutting.

Table 1. Summary of Test Methods Performed on FORTA products

Mixture Test	Purpose	Project(s)
Dynamic Modulus Test		1-7,9,11, 12
Resilient Modulus Test	Stiffness/Material Response from Load	8
4-Point Bending Modulus Test		10
Hamburg Wheel Tracking Test		8,10,11
Asphalt Pavement Analyzer Test	Rutting Evaluation	9
Flow Number Test		1-3,5,6, 8,9
Beam Fatigue Test		1-3,9
Uniaxial Fatigue Test	Fatigue Cracking (Initiation)	4-6
Indirect Tensile Fatigue		8,10
Semi-Circle Bending Test		10,11

In total these studies reveal many positive benefits using 1lb/ton fibers on mix stiffness, fatigue cracking, and rutting as summarized below:

- Stiffness: The FRAC mixtures showed an overall average of 13% stiffness increase compared to the conventional mixtures.
- Rutting Resistance: The FRAC mixtures exhibited 4.7 times higher Flow Number and 1.9 times lower rutting magnitude compared to those of the conventional mixtures.
- Fatigue cracking resistance: The cyclic fatigue tests results showed that FRAC mixtures exhibited 3 times more fatigue life compared to the conventional mixture.

4. PREDICTION OF PAVEMENT PERFORMANCE USING MEPDG

The pavement performance of the conventional and the 1 lb/ton FRAC mixture was predicted using the MEPDG. For this analysis MEPDG version 1.1 has been utilized. The total simulation period was 50 years for the purpose of life cycle cost analysis (LCCA) divided into three individual simulation periods (10 to 20 years). Two main distresses were considered in this analysis; total rutting and bottom-up fatigue cracking (alligator cracking). The MEPDG simulations were conducted with respect to Arizona State calibration. For the fiber-reinforced pavement, the alligator cracking distress prediction model was calibrated to increase the fiber-reinforced pavement resistance against alligator cracking by three times compared to the conventional pavement (concluded based on the average laboratory

performance of 12 fiber studies). The dynamic modulus $|E^*|$ values used for the fiber-reinforced pavement simulations were also increased by 13% which is the average increase in the modulus due the use of fibers blend based on the results obtained from 12 fiber studies. The total rutting distress prediction model for the fiber-reinforced pavement was kept the same as the conventional pavement as there is not enough data to calibrate the model for fiber reinforced mixture. However, it is anticipated that the fiber-reinforced pavement would exhibit better rutting performance due to the increased stiffness of fiber-reinforced mixture. Table 2 shows the conventional and the FRAC $|E^*|$ values used in the MEPDG simulations where the $|E^*|$ values of a typical 19-mm conventional dense graded mixture was used and then was increased by 13% for the FRAC mixture. Table 3 shows the MEPDG inputs for both conventional and fiber-reinforced pavements.

Table 2. $|E^*|$ Values for Conventional and Fiber-Reinforced Mixtures

Temp. °F (°C)	Freq. Hz	Dynamic Modulus, MPa - ksi (Test Values)				Modular Ratio
		Fiber-Reinforced		Conventional		
		MPa	ksi	MPa	ksi	
14 (-10)	25	6,847	47,206	6,059	41,775	1.13
	10	6,313	43,528	5,587	38,520	1.13
	5	6,215	42,850	5,500	37,920	1.13
	1	5,631	38,822	4,983	34,356	1.13
	0.5	5,397	37,206	4,776	32,926	1.13
	0.1	4,760	32,812	4,212	29,037	1.13
40 (4.4)	25	4,736	32,654	4,191	28,897	1.13
	10	4,551	31,378	4,027	27,768	1.13
	5	4,286	29,548	3,793	26,149	1.13
	1	3,621	24,961	3,204	22,089	1.13
	0.5	3,322	22,905	2,940	20,270	1.13
	0.1	2,663	18,359	2,357	16,247	1.13
70 (21.1)	25	2,552	17,590	2,258	15,566	1.13
	10	2,223	15,326	1,967	13,563	1.13
	5	1,989	13,715	1,760	12,137	1.13
	1	1,454	10,023	1,287	8,870	1.13
	0.5	1,252	8,630	1,108	7,637	1.13
	0.1	858	5,910	759	5,230	1.13
100 (37.8)	25	1,141	7,865	1,010	6,960	1.13
	10	924	6,374	818	5,641	1.13
	5	774	5,337	685	4,723	1.13
	1	499	3,441	442	3,045	1.13
	0.5	407	2,805	360	2,482	1.13
	0.1	266	1,834	235	1,623	1.13
130 (54.4)	25	437	3,015	387	2,668	1.13
	10	332	2,287	294	2,024	1.13
	5	279	1,923	247	1,702	1.13
	1	195	1,349	173	1,194	1.13
	0.5	176	1,216	156	1,076	1.13
	0.1	147	1,009	130	893	1.13

Table 3. Inputs of MEPDG Simulations

Traffic Data		
Initial two-way Annual Average Daily Truck Traffic (AADTT)		2,500
Number of lanes in design direction		2
Percent of trucks in design direction (%)		50
Percent of trucks in design lane (%)		80
Operational speed, kph (mph)		96.6 (60)
Traffic Growth Factor		Comp. 4%
Climate Data		
Weather Station		Phoenix Airport, PHX
Latitude (degrees.minutes)		33.26
Longitude (degrees.minutes)		-111.59
Elevation, m (ft)		337 (1106)
Depth of water table, m (ft)		6.1 (20)
Mean annual air temperature, °C (°F)		22.92 (73.25)
Pavement Section Data		
	Material type	Asphalt concrete
Layer 1	Layer thickness, cm (in)	10 (4)
	Reference temperature, °C (°F)	21.1 (70)
	Unbound Material	Crushed Gravel
Layer 2	Thickness, cm (in)	20 (8)
	Modulus, kPa (psi)	172,369 (25,000)
	Plasticity Index, PI	1
	Liquid Limit, LL	6
	Unbound Material	A-6
Layer 3	Modulus, kPa (psi)	99,974 (14,500)
	Plasticity Index, PI	16
	Liquid Limit, LL	33

Preliminary MEPDG simulations were conducted on the conventional pavement section so that it would show a balanced performance with respect to rutting and fatigue cracking and to be close as possible to the failure criteria of rutting or alligator cracking after 20 years from the initial construction. The total rutting failure criteria was 0.75 inch and the alligator failure criteria was 25% of the of the pavement area. The MEPDG simulations were then conducted on the rehabilitated conventional pavement section for another 20 followed by maintenance activities and then 10 years which is the end of the LCCA analysis period. The MEPDG simulations were conducted on the fiber-reinforced pavement using the same conventional structure until failure criteria was reached. In this case, the total simulation period was also 50 years divided into three stages where the stage period was determined when the fiber-reinforced pavement performance (total rutting or alligator cracking) reached the same level the conventional pavement reached at the end of each analysis period. The future traffic for each simulation period was predicted using 4.0% compound traffic growth rate. Figure 1 and Figure 2 showed a comparison of predicted total rutting and alligator cracking respectively for conventional and fiber-reinforced pavements. It is observed that the rutting performance of the fiber-reinforced pavement is slightly better than the conventional pavement; however the alligator cracking performance of the fiber-reinforced pavement is substantially better than the conventional pavement. Table 4 includes a summary of the MEPDG simulations for conventional and fiber-reinforced pavements.

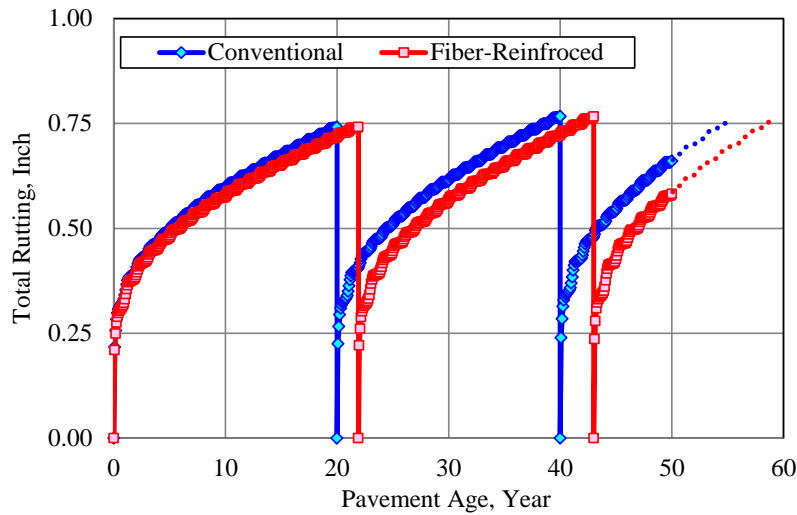


Figure 1. |E*| Prediction of Total Rutting for Conventional and Fiber-Reinforced Pavements

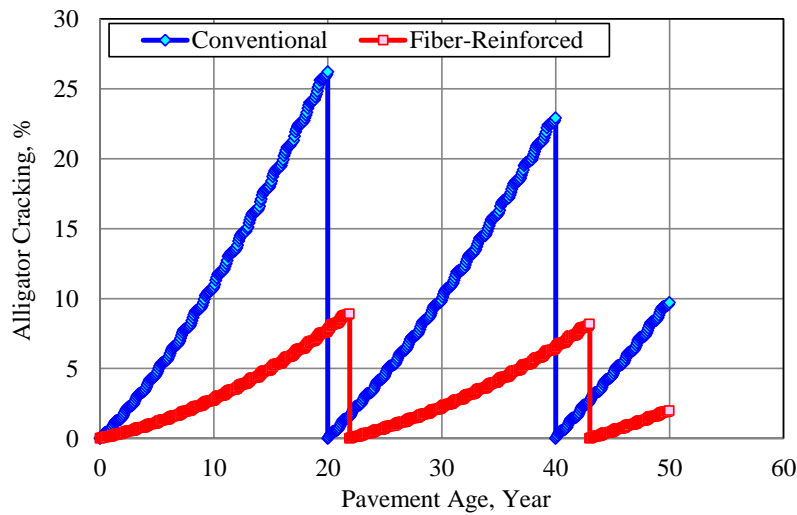


Figure 2. |E*| Prediction of Alligator Cracking for Conventional and Fiber-Reinforced Pavements

Table 4. Summary of MEPDG Simulations for Conventional and Fiber-Reinforced Pavement

Simulation Stage	Parameter	Conventional	Fiber-Reinforced
First	Simulation Period	20	22
	AADT at End of First Stage	5,500	6,000
	Rutting at End of First Stage, inch	0.74	0.74
	Alligator Cracking at End of First Stage, %	26.2	8.9
Second	Simulation Period	20	21
	AADT at End of Second Stage	12,000	12,500
	Alligator Cracking at End of Second Stage, %	22.9	8.2
Third	Simulation Period	10	7
	Rutting at End of Third Stage, inch	0.66	0.58
	Alligator Cracking at End of Stage, %	9.7	2.0
	Extended Surface Life after 50 Years, year (For salvage calculation)	4.75	8.25

5. REHABILITATION STRATEGY

The following decisions are made for the rehabilitation plan for both conventional and fiber-reinforced pavements:

- Crack seal is applied at 4% increment of alligator cracking
- Patching of alligator cracks is applied at 8% increment of alligator cracking
- Milling and asphalt concrete overlay are applied when total rutting is close to 0.75 inch
- The milling of the conventional pavement is 2.0 inches deep and 1.0 inch deep for fiber-reinforced pavement due to the higher alligator cracking for the conventional pavement
- The AC overlay thickness is 3.0 inches for the conventional pavement and 2.0 inches for the fiber-reinforced pavement as it has less amount of alligator cracking.

Based on the maintenance decisions, the rehabilitation activities for conventional and fiber-reinforced pavements are shown in Figure 3 and Figure 4 respectively. The predicted performance of both pavements showed that the use of the FRAC mixtures significantly reduces the rehabilitation activities especially to mitigate alligator cracking such as crack sealing and patching. Analysis was completed using 30, 40, and 50 year performance windows. Following national convention user costs (costs of delays, increased vehicle maintenance, impacts from pavement roughness on fuel consumption, etc.) were omitted from this analysis. In light of the impact of increased maintenance activities on these costs it is expected that this analysis results in a conservative assessment of the true life cycle cost benefits of FRAC mixtures.

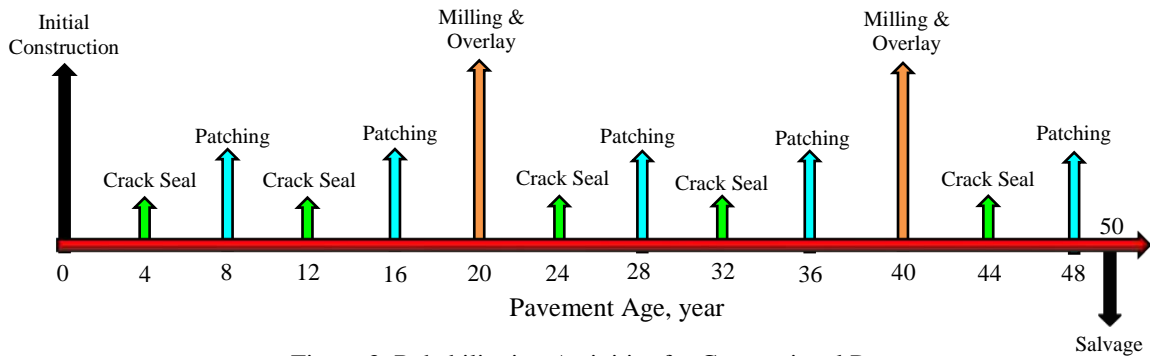


Figure 3. Rehabilitation Activities for Conventional Pavement

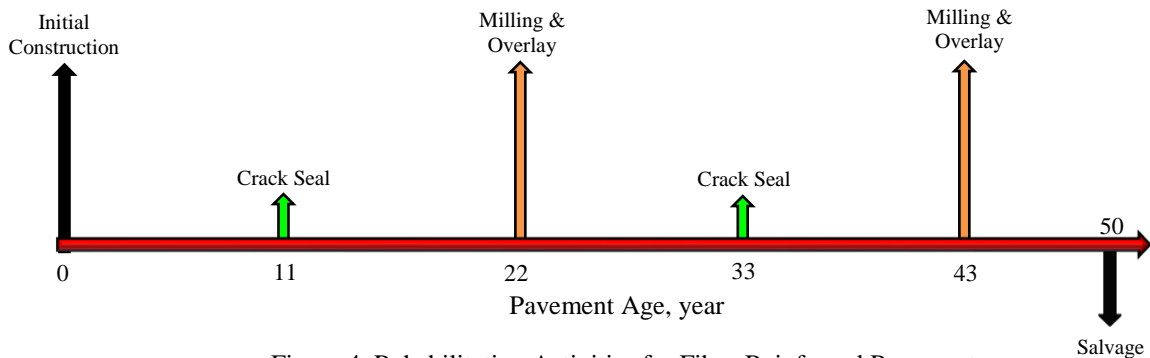


Figure 4. Rehabilitation Activities for Fiber-Reinforced Pavement

6. LCCA ANALYSIS

Updated costs for the applied rehabilitation activities were obtained according to the state of Arizona. The costs of the different construction and rehabilitation items are summarized in Table 5. The LCCA is conducted considering only one lane/mile for both conventional and fiber-reinforced pavement. The salvage value was determined based on the extended pavement service life after the LCCA period. The patching area was calculated by taking 33% of the cracked area and assuming one third of the alligator cracking area has severe cracks. The crack sealing length was estimated assuming there is one main fatigue crack under each wheel path and one transverse crack repeated every 15 ft. Table 6 and Table 7 include a summary of LCCA results of conventional and fiber-reinforced pavements respectively for the 50 year analysis period. Costs are assessed using both the net present worth, Equation (1), and the equivalent annualized cost, Equation (2). The net present worth converts all costs during the life cycle to current year dollars whereas the equivalent annualized method distributes the costs over the life time (accounting for the time-value

of money). Discount rates of 3-5% were used, which represent the conventionally suggested rates for LCCA. Results from all of the analysis cases are summarized in Table 8, as the difference in the PWC or EAC between the conventional and FRAC mixtures. In all cases the benefits of fibers are clear.

$$NPW = InitialCost + \sum_{j=1}^N R_j \left[\frac{1}{(1+i')^n} \right] - SalvageValue \left[\frac{1}{(1+i')^n} \right] \quad (1)$$

$$EAC = PWC \left[\frac{i'(1+i')^n}{(1+i')^n - 1} \right] \quad (2)$$

Where,

- i' = discount rate
- n = year of expenditure
- R_j = rehabilitation expenditure (single cost expenditure)

Based on the 50 year analysis period and the 4% discount rate, the following conclusions can be made:

1. The addition of the fibers at 1 lb/ton dosage reduces the net present worth by \$36,268 per mile/lane.
2. Increasing the cost of the fiber-reinforced mixture tonnage to \$72, the estimate reduction in the net present worth is \$33,462.
3. The savings in the net present worth due to the fiber usage is anticipated to increase if the user cost is considered due to the lower rehabilitation activities rate of the fiber-reinforced pavement compared to the control pavement. That means the user delays in case of the fiber-reinforced pavement is much less compared to the conventional pavement.

LCCA summarized in Table 8 showed that the cost differences change depending on the exact discount rate and analysis period considered, but in all cases the fiber-reinforced pavement was found to yield a reduced cost when compared to the conventional pavement.

Table 5. Costs of Construction and Rehabilitation Items

Item	Unit	Cost/Unit, \$
Conventional Asphalt Mix	ton	60.0
Fiber-Reinforced Asphalt Mix	ton	70.0
Crack Sealing	linear foot	0.35
Patching	square foot	2.6
Milling	ton	5.0
Asphalt Concrete Overlay	square foot/ inch	0.715

Table 6. Summary of LCCA for Conventional Pavement, 4% discount rate, (Lane/ Mile)

Activity	Time, year	Unit	Unit Cost, \$	Quantity	Total Cost, \$	Present Worth, \$
Initial Construction of 4 inches Conventional Mixture	0	ton	60.00	1,584	95,040	95,040
Crack Sealing	4	ft	0.35	580	203	174
Patching	8	square ft	2.60	1,670	4,342	3,173
Crack Sealing	12	feet	0.35	580	203	127
Patching	16	square ft	2.60	1,670	4,342	2,318
Milling of 2 inches	20	ton	5.00	792	3,960	1,807
Overlay of 3 inches	20	square ft	2.15	63,360	136,224	62,171
Crack Sealing	24	ft	0.35	580	203	79
Patching	28	square ft	2.60	1,670	4,342	1,448
Crack Sealing	32	ft	0.35	580	203	58
Patching	36	square ft	2.60	1,670	4,342	1,058
Milling of 2 inches	40	ton	5.00	792	3,960	825
Overlay of 3 inches	40	square ft	2.15	63,360	136,224	28,374
Crack Sealing	44	ft	0.35	580	203	36
Patching	48	square ft	2.60	1,670	4,342	661
Salvage	50	ton	60.00	765	(45,900)	(6,459)
Net Present Worth, \$						190,889
Equivalent Annual Cost, \$/yr						8,886

Table 7. Summary of LCCA for Fiber-Reinforced Pavement, 4% Discount Rate (Lane/ Mile)

Activity	Time, year	Unit	Unit Cost, \$	Quantity	Total Cost, \$	Present Worth, \$
Initial Construction of 4 inches Conventional Mixture	0	ton	70.00	1,584	110,880	110,880
Crack Sealing	11	ft	0.35	580	203	132
Milling of 1 inches	22	ton	5.00	396	1,980	835
Overlay of 2 inches	22	square ft	1.43	63,360	90,605	38,231
Crack Sealing	33	ft	0.35	580	203	56
Milling of 1 inches	43	ton	5.00	396	1,980	367
Overlay of 2 inches	43	square ft	1.43	63,360	90,605	16,777
Salvage	50	ton	70.00	1285	(89,950)	(12,657)
Net Present Worth, \$						157,624
Equivalent Annual Cost, \$/yr						7,198

Table 8. Summary of LCCA Results for NPW and EAC Methods for Analysis Periods and Discount Rates

Analysis Period (years)	Discount Rate (%)		
	3	4	5
Difference in NPW (\$/Lane/Mile)			
30	27,923	20,057	13,958
40	28,438	20,509	14,333
50	51,419	36,268	25,190
Difference in EAC (\$/Lane/Mile/Yr)			
30	1,085	934	765
40	1,105	955	785
50	1,998	1,688	1,380

7. CONCLUSIONS

In this study, the laboratory test results of 12 projects on FRAC and conventional mixtures were reported. The testing plan tended mainly to evaluate the materials stiffness, fatigue cracking, and rutting of conventional and FRAC mixtures. Based on the average results from 12 projects, the FRAC mixtures showed 13% stiffness increase, 4.7 times higher Flow Number, 1.9 times lower rutting magnitude, 3 times more fatigue life compared to the conventional mixture. Predicted pavement performance using MEPDG showed a better performance of the fiber-reinforced pavement especially against fatigue cracking which required less rehabilitation activities compared to the conventional pavements. Using discount rate ranges from 3-5% and analysis period ranges from 30 to 50 years, the LCCA detailed in this paper showed that using fibers can result in a saving in the net present worth dollar value ranges from \$14,000 to \$50,000 per mile/lane or a reduction in the equivalent annual cost ranges from 750 to 2,000 mile/lane/year. The estimated savings are expected to increase by calibrating the rutting model for the FRAC mixture in the MEPDG simulations and by considering user cost in the LCCA.

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PAPER TITLE	Automated detection of sealed cracks using 2D and 3D road surface data		
TRACK			
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KEYWORDS:

3D analysis; sealed cracks detection; pavement distress analysis; Laser Crack Measurement System; LCMS

ABSTRACT:

Reliable cracking data has proven difficult and expensive to obtain using cameras and video systems because of the lack of good automated 2D image processing crack detection algorithms. To solve this problem, 3D technology such as the LCMS (Laser Crack Measurement System) has been used to obtain automated reliable and repeatable cracking data.

The LCMS system has been widely used for automated crack detection on a variety of road surfaces (DGA, porous, chipseal, concrete) in over 35 different countries. While 3D techniques [1] [2] [3] [4] [5] have proven reliable at detecting open cracks these systems have not been used for detecting sealed cracks. These sensors however also often produce intensity (2D) images that are used to detect lane markings. Using this intensity (2D) data for the automated detection of sealed cracks has also proven unreliable because sealed cracks can sometimes be darker or brighter than the surrounding pavement in the images and tire marks and other features can also cause false detections.

This article will demonstrate that the accuracy of sealed crack detection can be improved by using both 2D intensity data and 3D texture information evaluated from the 3D data. To do this 3D texture evaluation algorithms are described and implemented in order to generate a complete texture map of the road surface. The intensity images are also processed in order to extract dark and light areas of the appropriate geometry (size and shape of sealed cracks). The combination of the results from both sets of processed data is then used to detect and validate the presence of sealed cracks.

Automated detection of sealed cracks using 2D and 3D road surface data

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1 INTRODUCTION

In order to optimize road maintenance funds and improve the condition of road networks, asset managers need detailed and reliable data on the status of the road network. 3D technology such as the LCMS (Laser Crack Measurement System) has been used to obtain automated reliable and repeatable cracking data on a variety of road surfaces (DGA, porous, chipseal, concrete). The LCMS is composed of two high performance 3D laser profilers that are able to measure complete transverse road profiles with 1mm resolution at highway speeds. The high resolution 2D and 3D data acquired by the LCMS is then processed using algorithms that were developed to automatically extract crack maps and severity. Also detected automatically are ruts (depth, type), macro-texture (digital sand patch) and raveling (loss of aggregates). This paper describes results obtained recently regarding road tests and validation of this technology for the detection of both sealed and unsealed cracks.

2 HARDWARE CONFIGURATION

The sensors used with the LCMS system are 3D laser profilers that use high power laser line projectors, custom filters and a camera as the detector. The light strip is projected onto the pavement and its image is captured by the camera. The shape of the pavement is acquired as the inspection vehicle travels along the road using a signal from an odometer to synchronize the sensor acquisition (see figure 1). All the images coming from the cameras are sent to the frame grabber to be digitized and then processed by the CPU. Saving the raw images would imply storing nearly 30Gb per kilometer at 100 km/h but using lossless data compression algorithms on the 3D data and fast JPEG compression on the intensity data brings the data rate down to a very manageable 20Mb/s or 720Mb/km. The critical specifications for the LCMS system can be found on table I. It is important to note that in addition to the 3D profiles the LCMS acquires the intensity of the reflection of the laser at each 3D point thus creating an intensity 2D image of the pavement while simultaneously measuring the shape. Figure 2 shows range (distance) image and intensity (2D) data acquired from the LCMS. A 3D image can be generated from the range and intensity data as shown.

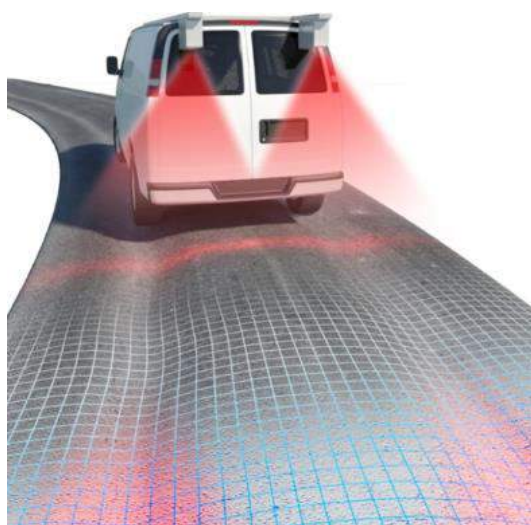


Figure 1. LCMS on an inspection vehicle.

Table 1. LCMS Specifications

Nbr. of laser profilers	2
Sampling rate (max.)	11,200 profiles/s
Vehicle speed	100 km/h (max)
Profile spacing	Adjustable
3D points per profile	4096 points
Transverse field-of-view	4 m
Depth range of operation	250 mm
Z-axis (depth) accuracy	0.5 mm
X-axis (transverse) resolution	1 mm

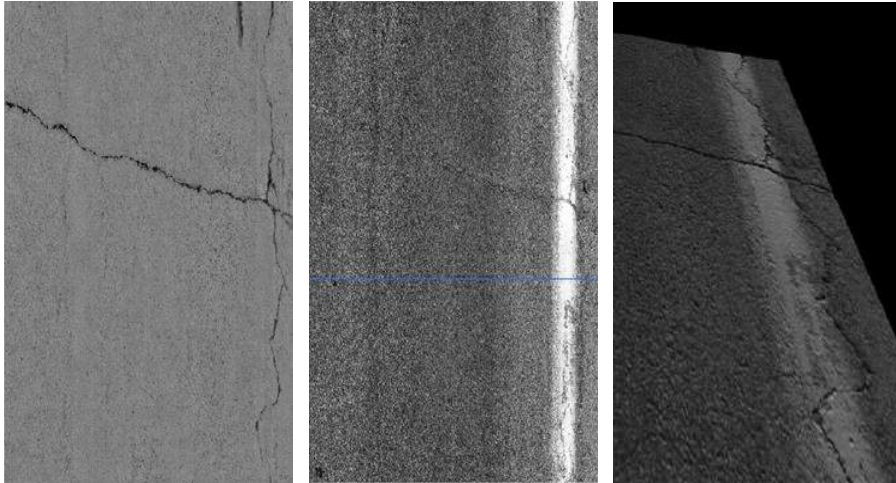


Figure 2. LCMS data type - Range (left) - Intensity (center) - 3D merged (right).

3 RANGE DATA

The 3D data acquired by the LCMS system measures the distance from the sensor to the surface for every sampled point on the road. The previous image (above left) shows a range data image acquired by the sensors. In this image, elevation has been converted to a gray level. The darker the point, the lower is the surface. In a range image the height can vary along the cross section of the road. The areas in the wheel path can be deeper than the sides and thus appear darker this would correspond to the presence of ruts. Height variations can also be observed in the longitudinal direction due to variations in longitudinal profiles of the road causing movements in the suspension of the vehicle holding the sensors. These large-scale height variations correspond to the low-spatial frequency content of the range. The first step in range analysis is to separate the high frequency content from the low one. This is performed using a specially designed filter. Figure 3b shows the result of this process. The low frequency part is what can be referred to as the mean surface. The high-frequency part clearly shows the presence of surface defects (cracks) and texture. Once separated, both frequency parts are then used as an input to the various feature detection algorithms the high frequency component is called the rectified range image as shown below.

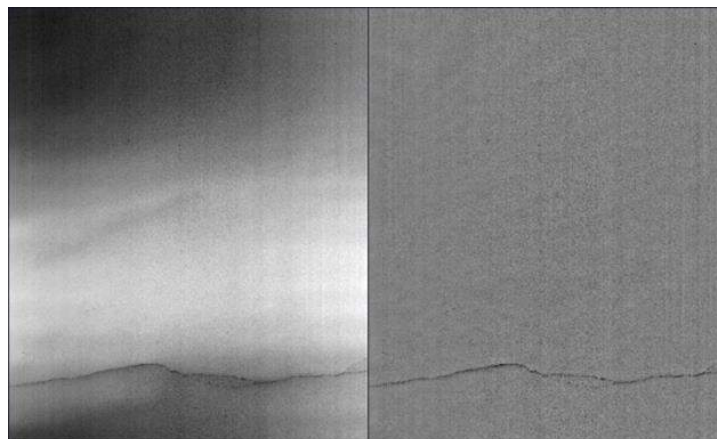


Figure 3. (a) Range (raw) and (b) Range (rectified) images

4 MACROTEXTURE

Macrotexture is important for several reasons, for example it can help estimate the tire/road friction level, water runoff and aquaplaning conditions and tire/road noise levels produced just to name a few. Macrotexture can be evaluated by applying the ASTM 1845-01 norm [6]. This standard requires the calculation of the mean profile depth (MPD). To calculate the MPD, the profile is divided into small (10cm) segments and for each segment a linear regression is performed on the data. The MPD is then computed as the difference between the highest point on the profile and the average fitted line for the considered portion. MPD is the only way possible to evaluate texture using standard single point (64 kHz) laser sensors. The LCMS however acquires sufficiently dense 3D data to not only measure standard MPD but also to evaluate MTD texture (mean texture depth) using a digital sand patch method (ASTM E965) [7] as shown on figure 4.

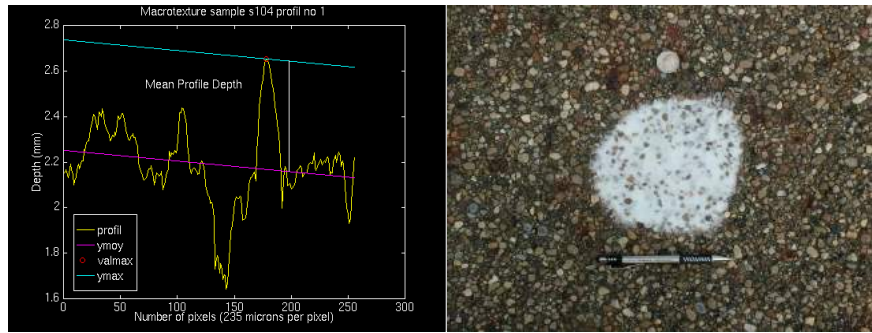


Figure 4. MPD vs sand patch.

The digital sand patch model MTD calculates the volume of the voids in the road surface that would be occupied by the sand (from the sand patch method) divided by a surface area. The digital sand patch method implemented allows texture to be evaluated continuously over the complete road surface instead of measuring only a single point inside a wheel path. The MTD results can be mapped into road surface texture image where the color corresponds to texture levels. In this example bleeding or flushing on the road surface corresponds to areas of very low MTD (0.2mm) shown in blue (figure 5).

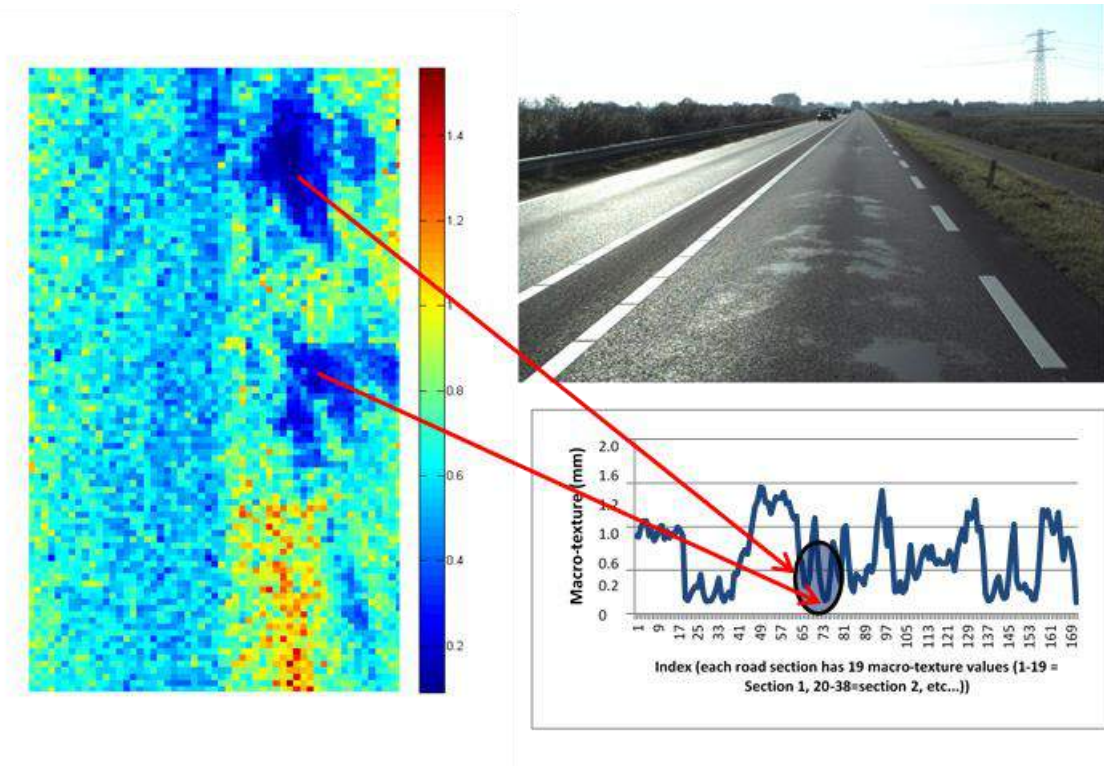


Figure 5. Bleeding

5 CRACKING

Although the basis of 3D crack detection is finding the crack points that are below the road surface, detecting cracks reliably is far more complex than applying a threshold on a range image. As mentioned previously the 3D data needs to be rectified to remove the effects of rutting and vehicle movements. Macrotexture is also a problem; road surfaces have very variable macrotexture from one section to the next and even from one side of the lane to the other. In general to detect a crack using 3D techniques the majority of the crack points must be below the average depth of the macro-texture of the road surface. For example, on roads with weak macrotexture we can hope to detect very small cracks which will be harder to detect on more highly textured surfaces. It is thus necessary to evaluate and to adapt the processing operations based on the texture and type of road surface. Once the detection operation is performed, a binary image is obtained where the remaining active pixels are potential cracks. This binary image is then filtered to remove many of the false detections which are caused by asperities and other features in the road surface which are not cracks on the pavement. At this point in the processing, most of the remaining pixels can correctly be identified to existing cracks, however many of these crack segments need to be joined together to avoid multiple detections of the same crack. After the detection process, the next step consists in the characterization of the cracks. The severity level of a crack is determined by evaluating its average width (opening) typically cracks will be separated in low, medium and high severity levels.

The LCMS system has been used to survey millions of lane kilometers over the past 6 years and has demonstrated its ability to automatically detect cracks [3]. Independent 3rd party users and DOTs have also evaluated the system [8] [9]. Typical results include repeatability tests that usually attain 95%+ in many different conditions.

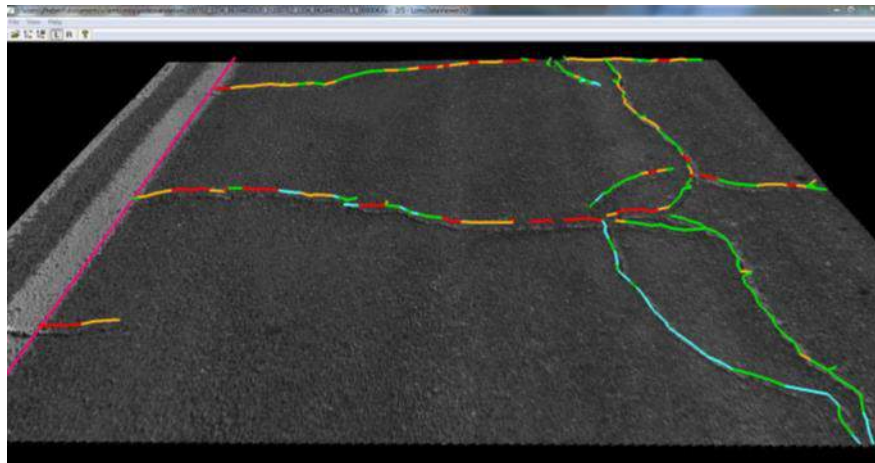


Figure 6. Example of crack detection results (severity = color code).

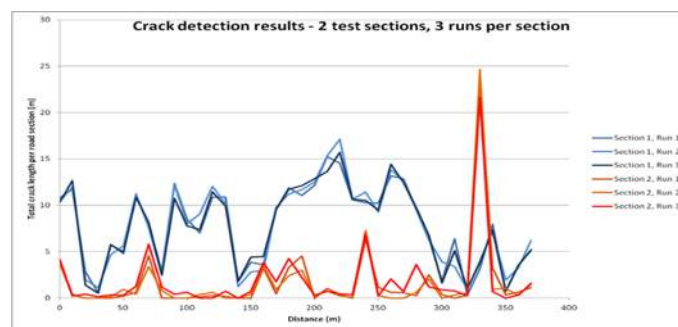


Figure 7. Repeatability results (3 passes) on two MTQ road sections.

6 SEALED CRACKS

The images (Figure 8) below demonstrate what the majority of sealed cracks look like, i.e. the sealed cracks are very dark in the images due to the black color of the sealant. This makes these cracks very visible in the intensity images (high contrast) and the fact that they are sealed makes them virtually 'invisible' in the range images compared to the unsealed cracks that appear as dark lines in the rectified range images.

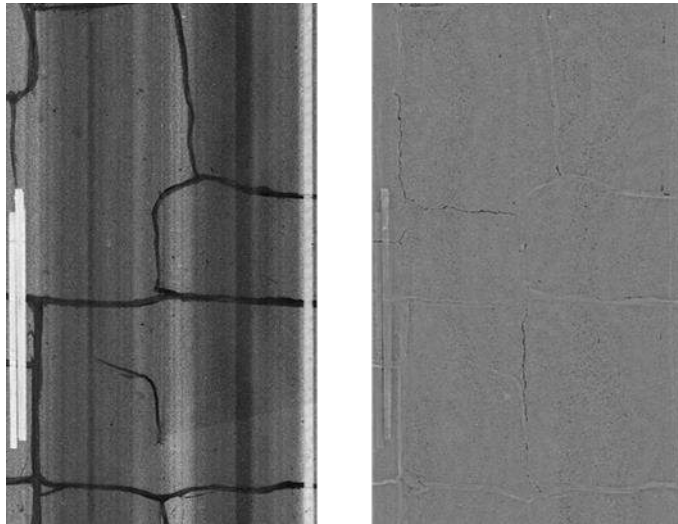


Figure 8. Example sealed cracks: Intensity image (left) Rectified range (right).

However, the next images (Figure 9a) shown below demonstrates that in some cases the sealed cracks do not appear dark, in fact their color can vary greatly ranging from black to shiny white. Thus intensity alone cannot be used as a reliable indicator of their presence.

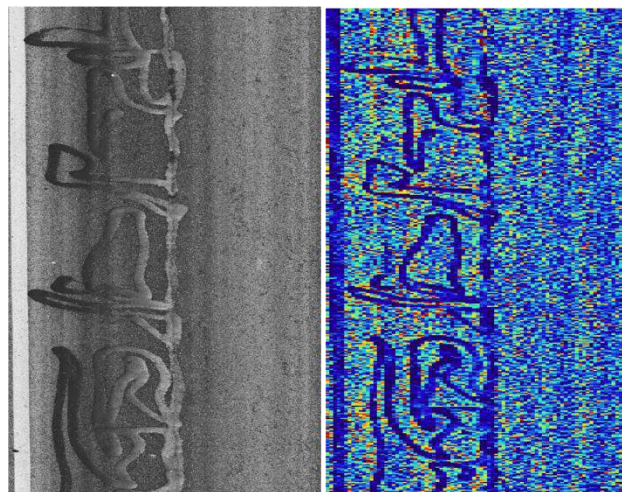


Figure 9 Example sealed cracks: a) Intensity image b) Texture map

Not being able to rely on color and since sealed cracks do not have a 3D depth associated with them as do unsealed open cracks we attempted to detect them using texture measurements as the surface of a sealed crack is very smooth (low MPD values) as the sealant used not only fills the crack but also the air voids of the asphalt texture. Figure 9b shows the texture color map of the roadsection with sealed cracks it is clear that these areas show very low texture MTD values (blue).

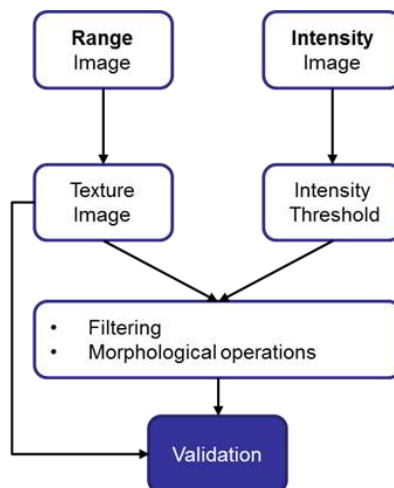


Figure 10. Main steps in the detection of sealed cracks

Figure 10 describes the main steps in the algorithm implemented. This algorithm uses in fact both intensity and range (texture) data to detect sealed cracks. The intensity image analysis algorithm looks for dark or light areas that are of different color than the surrounding background (road surface) and flags these areas as possible sealed cracks. As mentioned above the range data 3D texture is evaluated (MTD) and areas of low texture are detected as possible sealed cracks. All candidate areas are then evaluated as to their shape conformity to make sure they are crack shaped i.e. much longer than wide and with a width ranging up to a few centimeters. A final validation is done to make sure all remaining sealed crack candidate areas have both appropriate texture and color characteristics before being accepted.

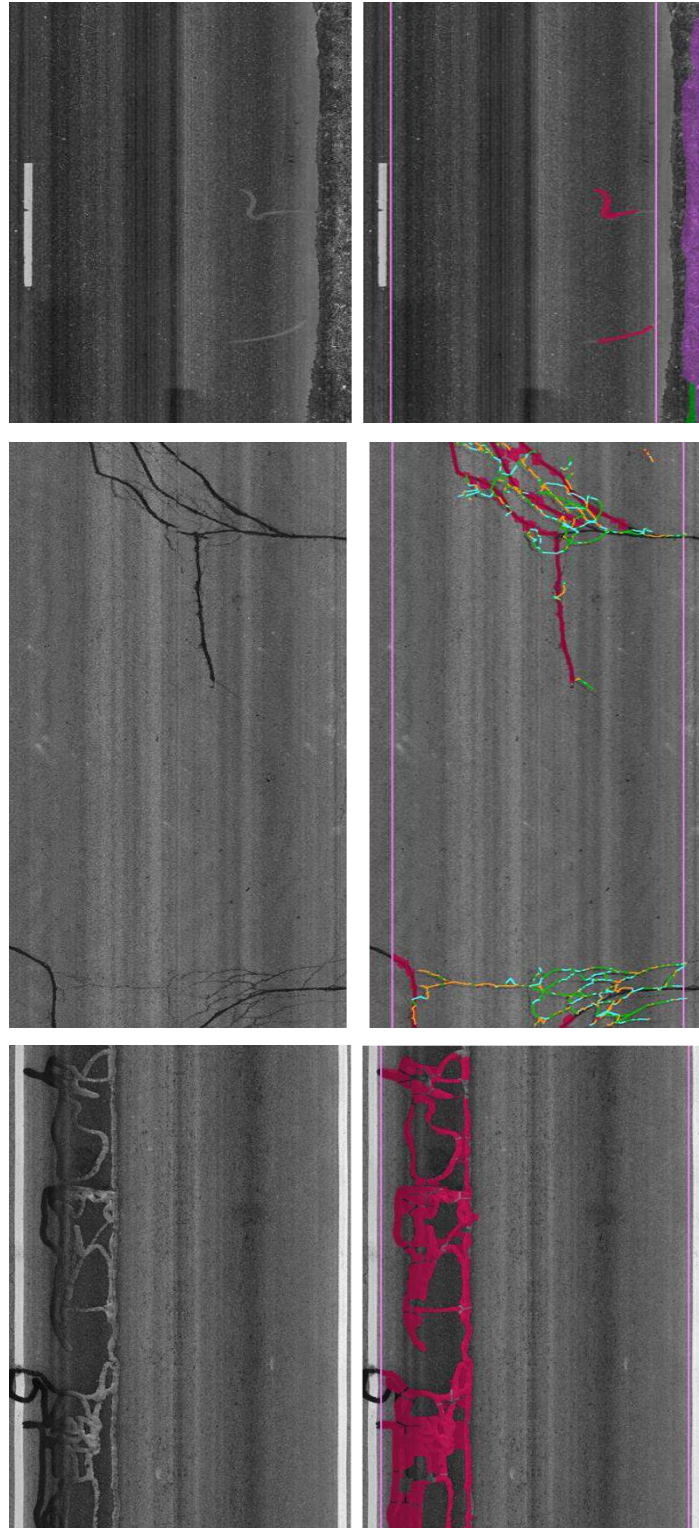


Figure 11. Three examples of sealed cracks detection on road surfaces (results in red)

7 CONCLUSIONS

Initial road tests on sealed crack detection were conducted with algorithms identifying only black sealed cracks based on intensity images only. These tests allowed us to acquire a set of 100+ road sections where the use of an intensity based algorithm alone for the detection of sealed cracks did not work. These examples were taken from different surveys covering several hundreds of kilometers of roads. With the algorithm described above using both 3D texture and intensity combined, over 90% of the sealed cracks missed by the initial algorithm were correctly detected with less than 5% false positives (visually it is never 100% certain in the images if a sealed crack is or isn't present, no on-site validation type ground truth was attempted).

Currently network level testing has been going on over the last six months with two different vehicles in operation in both New Zealand and the USA. No obvious or systematic cases of false or missing detections of sealed cracks have been noted. However, it has been noted that as the sealed crack ages and the sealant wears away and chipping starts. The chipping of the sealant and the texture of the road thus begin to appear increasing MTD (texture) values that can over time cause the sealed crack to be missed by this algorithm.

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PAPER TITLE	Weight impact tests of light-weighted barriers on expressways		
TRACK	Road Safety		
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KEYWORDS:

barriers, light-weighted, weight impact tests

ABSTRACT:

Three light-weighted barriers, namely, the ultra-high-strength fiber reinforced concrete barrier (UFC barrier), the hollow reinforced concrete barrier (hollow RC barrier), and the base concrete barrier (BC barrier) were developed .

For the practical use of three types of barriers developed, it was necessary to carry out evaluation tests to confirm the required performances according to the safety standards. Therefore, weight impact tests were implemented to simulate vehicle collision.

In the present paper, the results and discussions of the weight impact tests that were carried out for the three concrete barriers developed are reported.

Weight impact tests of light-weighted barriers on expressways

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1 INTRODUCTION

Steel barriers are often used to reduce the dead load of bridges. These steel barriers are hollow structures and are designed under the concept of being a sealed structure. However, corrosion due to infiltration of water was found in the interior of steel barriers.

As a result, steel barriers posts and base fixing bolts were corroded, reducing considerably their cross-sectional areas and load carrying capacity. This problem indicates that steel barriers cannot perform their function as protective fences. In addition, traffic regulation on the highway when repairing the steel barriers are also considered to be a problem and thus, steel barriers maintenance is considered to be a difficult work.

From the above, it was decided to replace these barriers by corrosion resistant light-weighted concrete barriers. However, such bridge barriers did not exist in the market. Therefore, three types of lightweight barrier, namely, Ultra High Strength Fiber Reinforced Concrete barrier (UFC barrier), hollow Reinforced-Concrete barrier (hollow RC barrier) and the Base Concrete barrier (BC barrier) were proposed and their development have started. In addition, weight impact tests were carried out as a performance verification for the developed bridge barriers.



Figure 1.Reduction of Cross-sectional Area due to Internal Corrosion of Steel Barrier Posts

2 CHARACTERISTICS OF THE PROPOSED BARRIERS

As there was no concrete barrier having unit weight close to that of steel barriers, new barrier structures having better maintenance properties, such as light-weight and improved corrosion resistance were necessary.

When steel barrier problems were re-evaluated it was found that one problem to be overcome was water. The condition inside the barriers was highly humid, because water drainage was not secured. Further, repeated dry-wet cycles occurred inside the barrier, contributing to the significant advances of corrosion.

So, with basic idea of a barrier structure having a concrete base, the following three types of barriers were proposed as replacement structures, and their standardization was carried out.

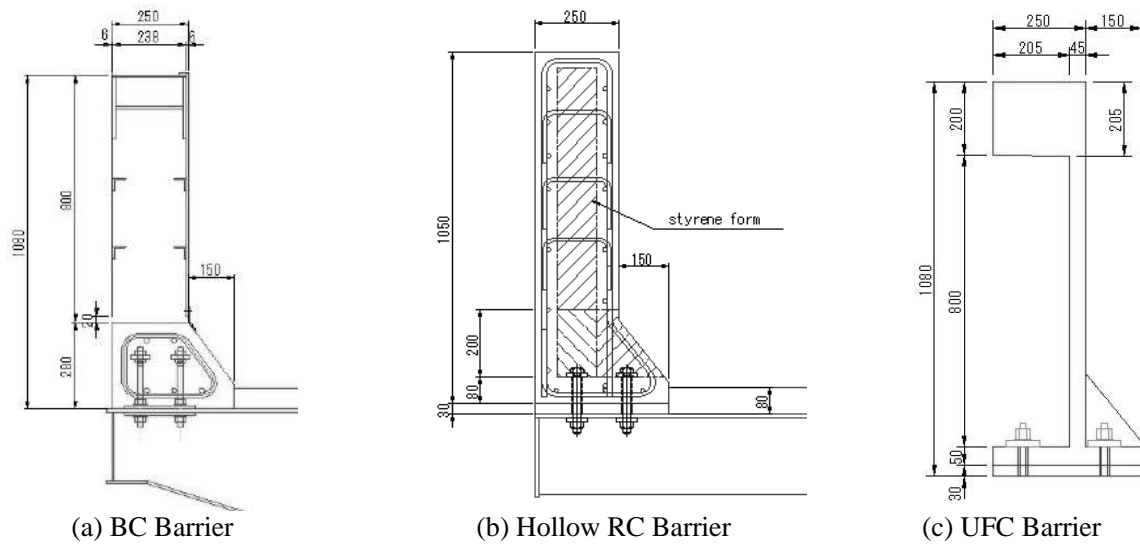


Figure 2. Proposed barriers

(1) BC BARRIER

Steel barriers are very good protection fences, but problems were found in their posts and post bases, which are their main elements of the steel barriers. Therefore, we considered solving these problems by taking advantage of the good characteristics of steel barriers.

To reduce corrosion progression in post base members, the elements were protected against corrosion by galvanization. Further, drainage of the structure was provided by making a space between the side plate and the barrier at the curb part. In addition, by installing the steel wall at the base concrete block above the pavement surface, infiltration of water from the road surface, which may contain anti-freezing agent, was prevented. As a result, corrosion factors could be reduced considerably.

By arranging transverse beams between posts, the resistance to the impact resistance is enhanced. Although there are corrosion risks, as there are steel parts remaining, the structure had improved corrosion resistance.



(a) BC Barrier wall provisional assembly



(b) BC Barrier wheel guard concrete part



(c) BC Barrier installation

Figure 3. BC Barrier

(2) HOLLOW RC BARRIER

To ensure corrosion resistance, light-weight concrete barrier was considered. To lighten the RC barriers, filling the structure with styrene foam was considered.

Hollow RC Barrier was a successful design in lightening the structure where styrene foam worked as internal formwork. In addition, ribs were also provided inside the barrier. Having reduced its weight, it is a fairly thin structure. Since the required covering depth cannot be secured, rebar rust protection is secured by using the epoxy resin rebar.



(a) Styrene foam



(b) Hollow RC Barrier inside of Styrene foam



(c) Completed Hollow RC Barrier

Figure 4.Hollow RC Barrier

(3) UFC BARRIER

For UFC barriers, as in the case of hollow RC barriers, to ensure corrosion resistance, light-weight concrete was considered. Barrier concrete material was changed and UFC was used.

Due to UFC material properties, rebars were unnecessary in UFC barriers and structure lightness could be obtained successfully. Since the UFC barrier does not use rebars, it is necessary to consider the barrier shape.

In order to obtain light-weight and high-strength, ribs were provided at the rear part of the barrier to ensure the required weight and strength.



(a) UFC Barrier front surface



(b) UFC Barrier back surface

Figure 5.UFC Barrier

(4) COMPARISON OF THE THREE BARRIER WEIGHTS

Table 1 shows the weight ratio of the three structures compared to that of a steel barrier of a certain section. The results of the calculations considering the weight ratio in a certain section confirmed the reduction of weight, once the calculated weight is close to that of the steel barriers.

However, it is necessary to confirm the girder strength against the increase of the barrier load, because many of the bridges were designed for live loads of old specifications.

Table 1. Comparison of the barrier weight

	Steel Barrier	BC Barrier	Hollow RC Barrier	UFC Barrier
weight	3.73kN/m	4.74kN/m	5.49kN/m	4.59kN/m
weight ratio	1	1.27	1.47	1.23

3 TEST METHOD

The purpose of weight impact tests is to confirm the condition that "the barrier collapses when the vehicle collides and the vehicle does not jump over the barrier" has been fulfilled or not.

In order to confirm it, weight impact tests were planned for the developed barriers. The advantage of this testing method is that it can be carried out at prices lower than that of impact tests using vehicle. In addition, the differences with the tests carried out with vehicles is that the weight does not break, and experiments can be carried out repeatedly until the specimen is destroyed, and failure shape of the specimen can be verified. In this test, the free fall hanging the weight collides with the barrier. The behavior and failure shape of the barrier is checked for the time of impact.

If the drop height of the weight of impact tests is 650mm, it is confirmed that the barriers is able to ensure the performance. Therefore, in the present paper it was decided to evaluate the behavior of the barriers for drop height of 400mm for small impact, and drop height of 650mm for standard impact.

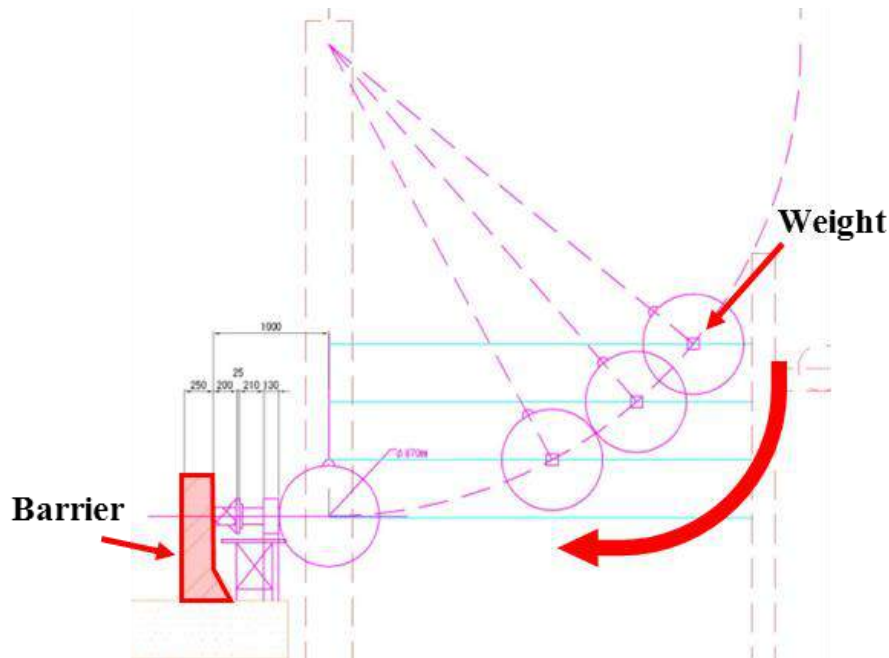


Figure 6. Schematic diagram of barrier impact tests

4 TEST RESULTS

The results of the tests are presented in the figure below, presenting the failure shape after the weight collision for each case.

Drop height of 400mm for RC hollow barrier

- A maximum of 20mm level difference at the impact surface has occurred with the cracks
- Cracks of about 2mm have occurred around the box vent for the barrier for joints

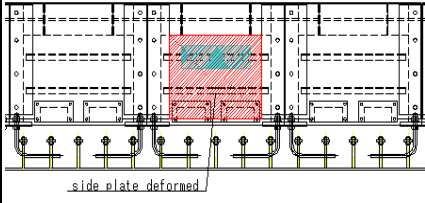
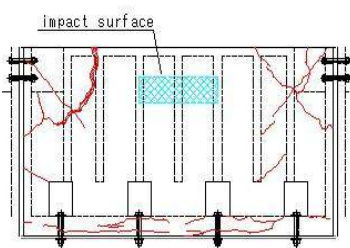
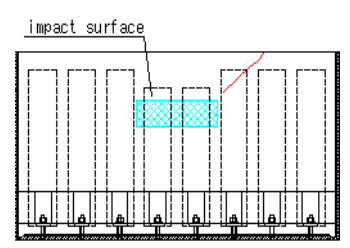
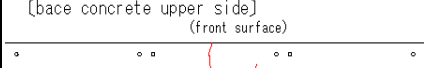
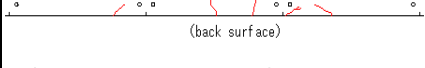
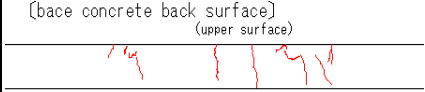
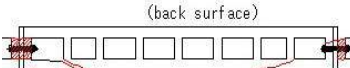
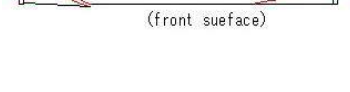
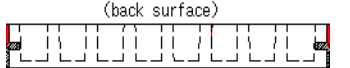
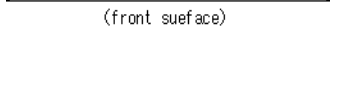
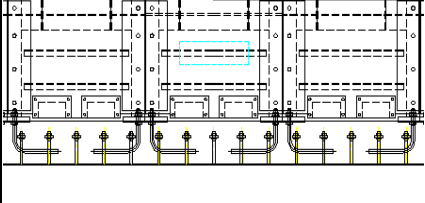
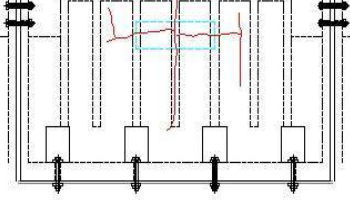
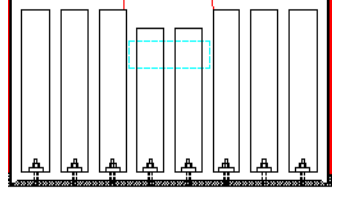



Drop height of 400mm for UFC barrier

- Cracks of about 0.1mm have occurred at the impact surface and its back surface

Drop height of 650mm for BC barrier

- The impact surface of the side plate deformed to a maximum of approximately 20mm.
- The concrete at the curb back and the top surface presented cracks of about 0.1mm and a level difference of about 1mm have occurred with the cracks.

Table 2 Shape after the weight collision

	BC Barrier	Hollow RC Barrier	UFC Barrier
front surface			
top surface	<p>(base concrete upper side) (front surface)</p>  <p>(back surface)</p>  <p>(base concrete back surface) (upper surface)</p> 	<p>(back surface)</p>  <p>(front surface)</p> 	<p>(back surface)</p>  <p>(front surface)</p> 
back surface			
After the impact tests	 <p>Drop height of 650mm</p>	 <p>Drop height of 400mm</p>	 <p>Drop height of 400mm</p>

5 CONCLUSIONS

To summarize,

- Three types of light-weighted and high-corrosion-resistance barriers were proposed
- The three barriers were successful in lightening the structures
- Impact tests were performed on the three types of the developed the barrier
- None of the barriers did not collapse through weight impact tests. However, experiments on RC hollow barrier and UFC barrier were not completed.
- In Drop height of 400mm for RC hollow barrier, cracks occurred with a maximum of 20mm of level difference.
- In Drop height of 400mm for RC hollow barrier, cracks of about 2mm around the barrier joint box vent have occurred.
- In Drop height of 400mm for UFC barrier, cracks of about 0.1mm occurred at the impact surface and its back surface.
- In Drop height of 650mm for BC barrier, impact surface of side plate deformed to a maximum of approximately 20mm. Cracks of about 0.1mm with a level difference occurred at concrete barrier curb back surface and the top surface.

It was confirmed that UFC barriers did not collapse at the time of vehicle collision and did not jump over the Barrier.

However, hollow RC barrier and BC barrier are schedule to proceed the impact tests.

When performance verification of the developed barriers are completed, improvements can be made by applying the experimental results to the developed barriers. We expect to complete the replacement works using the modified barriers with improved quality.

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KEYWORDS:

Road debris, Road kill, road worker, layout design, Safety

ABSTRACT:

Despite the government's efforts to control overloaded vehicles on the roads in South Korea, the number of road debris-induced accidents is increasing every year. Such accidents being an extremely likely cause of massive accidents, preventive measures should be established to protect the drivers as well as the road maintenance crews against safety hazards.

This study conducted surveys that targeted field professionals working for the road maintenance agencies. The results were incorporated into the layout design of road debris remover instrument which is customized to the road condition. The design involved examination and incorporation of instrument construct, road debris remover quantity, target and scope of removal, and other relevant information as identified through the surveys.

The proposed layout design is capable of maximally securing the safety of drivers and road workers through swift removal and handling of road debris. The advantages will likely ensure improvement of road service and traffic safety.

Layout Design of Korean-Type Road Debris Remover Instrument

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1. INTRODUCTION

Each year the Republic of Korea's Ministry of Land, Infrastructure and Transport (MOLIT) carries out its control and citing of overloaded vehicles that drive on the roads. The monitoring produces the citation of average some 30,000 to 40,000 cases including violation of load limits imposed by the relevant legislation. Despite the activities, road accidents involving debris are showing an increasing trend every year. Analysis of debris-related accidents over the past few years show the average annual accident rate of 41. Table 1 shows statistics on road debris on highways in South Korea.

Table 1. Statistics of road debris on Korean highways

Year	Number of road debris collected	Number of accidents related to road debris (death)	Number of reports on bad loading
2010	312,829	20(0)	50,964
2011	314,080	33(0)	65,800
2012	313,605	44(0)	87,070
2013	273,026	64(0)	83,527
2014	290,764	43(1)	60,789

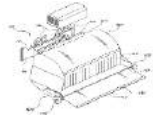

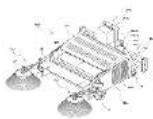
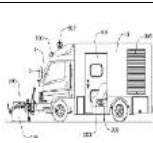
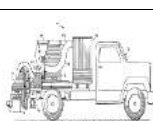
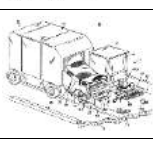
Source: Data from Parliamentary Land, Infrastructure and Transport Committee's regular audit of Korean Expressway Corporation

Currently, the type of road debris that pose the greatest danger to driver safety is estimated to be road kills such as roe deer. Similarly, rocks and stones, automobile components from accidents (shredded tires, bumpers, metals, etc.), beverage cans, glass bottles, boxes, and wood panels were found to threaten the road travel safety. Removal of road debris has been done mostly by manual cleanup of road workers upon receiving complaints. Additionally, sweeping machines are used to clean up after road traffic accidents. With the sweeping machines, the problem is their inadequacy in picking up larger debris such as bulky waste, cargo and dead animals. No particular device or equipment has been devised to remove such items. Any debris-related incident on highways is a potential hazard that can cause massive traffic accidents, including chain-reaction crashes. As such, debris-induced accidents should be subject to appropriate preventive measures. The purpose of this study was to create a layout design for debris remover equipment that aims to accomplish cleanup in swift and safe manners. Road maintenance agencies were contacted for surveys to be carried out. Results were incorporated into the layout design following reviews on the instrument's construct and on the capacity, targets and scope of removal.

2. LITERATURE REVIEW AND TECHNOLOGY TRENDS

Patent applications regarding road debris removal technology revealed a total of 13 such applications of which 7 cases were available for citation in South Korea. The corresponding number overseas was 354 in total. Four of them were found to be citable. Investigation of the applications' details showed that very few technology solutions existed which specifically addressed road debris removal, and that the majority of the applications simply utilized conventional cleaning equipment or machines to collect the road debris (Table 2).

Table 2. Patents related the road debris remover instrument

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

The AAA Foundation for Traffic Safety (2004) investigated danger associated with debris on North American roads (2004). The Colorado Department of Transportation (CDOT) carried out field tests on the Gator Getter™, a high speed debris remover instrument developed by CDOT. Valdes-Vasquez et al. (2014) conducted evaluation of the performance of the instrument. C.H. Yang et al. (2016) proposed directions of design concept in the development of road debris remover instrument customized to Korean roads.

3. SURVEYS

In developing a debris remover instrument that is customized to the condition of Korean roads, it is crucial to have feedback from personnel involved in actual road maintenance work. Keeping that in mind, the present author conducted surveys with some of the local highway agencies so as to identify the actual demand for debris remover instrument (Fig. 1).

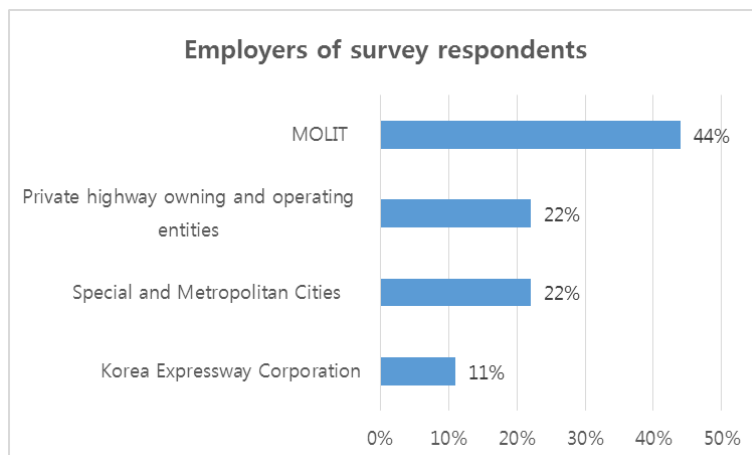


Fig. 1 Employers of survey respondents

Fig. 2 summarizes the frequency of debris removal work as performed by the highway maintenance agencies. As shown, the majority of these agencies (89%) responded that they handle at least one incident per day.

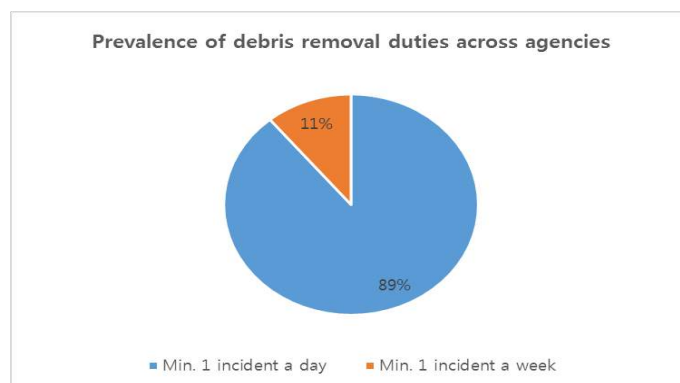


Fig. 2 Prevalence of debris removal duties across agencies

The types of debris items getting removed are presented in Fig. 3, where tire pieces (89%) and waste paper (89%) were found to be the major contributors. Dead animals, plastics, and auto accident debris followed in said order. The respondents also reported that most of the debris items are collected manually, with the remaining ones being picked up by sweeper machines.

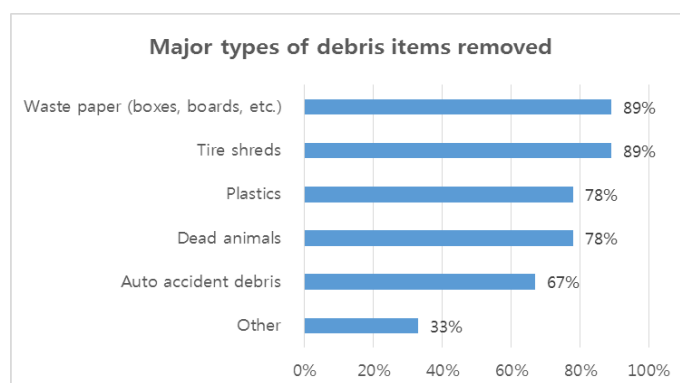


Fig. 3 Major types of debris items removed

As to the necessity of nighttime removal of debris, 67% of the respondents said 'No' (Fig. 4). The result indicates that the majority of debris removals are carried out during the day unless in an emergency. The survey participants advised night shifts, if any, would require related equipment/tooling and support to indicate such work is under way, e.g., safety signage, speed reduction, fluorescent markers for temporary lane changing, and surveillance cameras. Their input is considered to aim at preventing road blocking, rear-end crashes, etc. during removal operations.

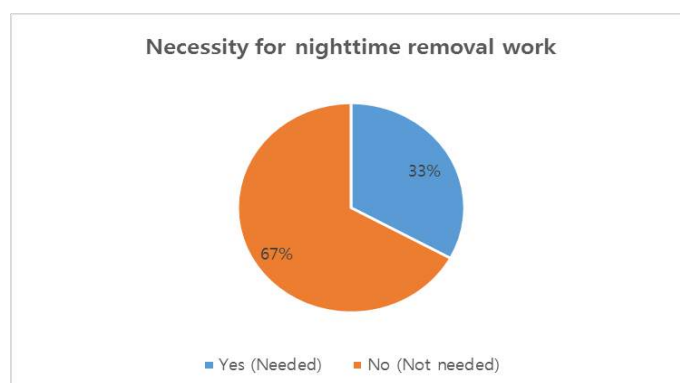


Fig. 4 Necessity for nighttime removal work

On removing debris using the instrument, 56% of the respondents answered yes to whether speed limits need

to be considered (Fig. 5). The most preferred maximum speed during removal work was indicated as ‘below 30 km/h’ - followed consecutively by ‘30-40 km/h’ and ‘80 km/h and above.’ The rating suggests debris removal is necessary for driving at not only low speeds but high speeds as well.

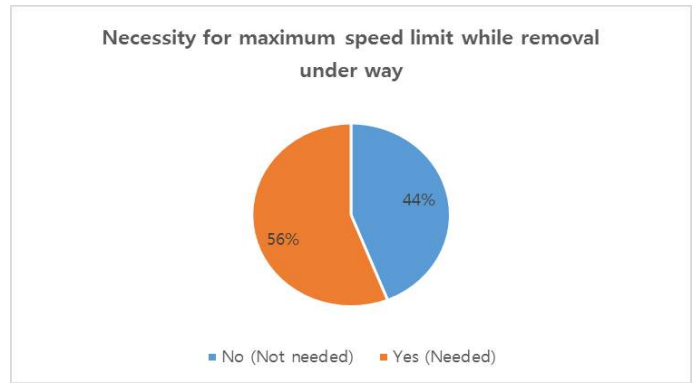


Fig. 5 Necessity for maximum speed limit while removal under way

The intensity of demand for debris removal was examined for road type. Fig. 6 shows the highest demand was identified with highways and expressways. In descending order, national highways in cities and non-urban areas, and roads in municipalities, counties and provinces followed.

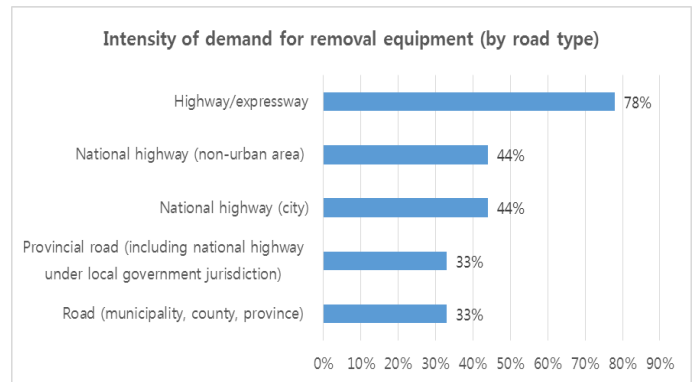


Fig. 6 Intensity of demand for removal equipment (by road type)

Fig. 7 summarizes highway agency-specific needs for purchasing the debris remover instrument. Across the agencies, the most prevalent answer in terms of purchase quantity was ‘2’ (44%) while 11% of the responding agencies answered ‘4 or more.’ Of note, some agencies said they were not considering the purchase because, as investigated further, of insufficient instrument drivers and the currently available instrument other than the one being proposed in this study.

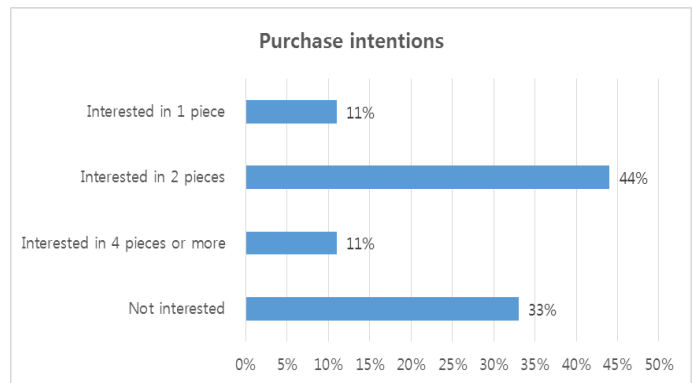


Fig. 7 Purchase intentions

The quantity of debris per removal most preferred by the respondents was ‘Less than 1 ton’ (78%) (Fig. 8). The second most preferred answers were ‘1 ton or more’ (11%) and ‘More than 3 tons’ (11%). Further, given the

responses on the quantity, the most favored answer for the removal tool capacity was 2.5 tons.

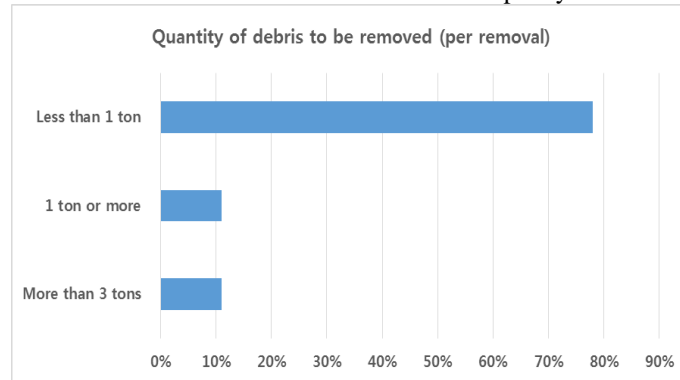


Fig. 8 Quantity of debris to be removed (per removal)

4. LAYOUT DESIGN

Based on the results of the surveys, Table 3 lists considerations to be made while designing and developing the instrument for removing road debris. A truck capable of carrying a 2.5 ton load was selected for the design development taking into account the preferred appropriate amount of debris per removal around 1 ton. Additionally, the vehicle’s construct and the types of debris items to be picked up were examined and then incorporated into the layout. The latter stages of the development will also incorporate travel speed of the vehicle, the total weight of debris per removal plus other factors.

Table 3. Considerations for tool development

Factors	Considerations
Vehicle type	Truck with large freight capacity (2.5 t)
Truck travel speed	Driving at 10 km/h or above while picking up debris
Per-pickup total debris weight (by type)	<ul style="list-style-type: none"> · Ordinary waste: max. 5 kg (approx.) · Carcass: 11 kg (approx.)
Debris amount per removal dispatch	Approx. 1 ton
Vehicle construct	Must be able to: <ul style="list-style-type: none"> · Collect both carcass and ordinary waste · Separate carcass from non-carcass debris · Satisfy relevant vehicle safety standards
Debris item	<ul style="list-style-type: none"> · Sand and pebble · Carcass, and debris remaining after auto accidents · Any and all debris obstructing traffic flow (plastics, wood chips, boxes)

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

Fig. 9 (plan view) and Fig. 10 (elevation view) illustrate the layout of the truck to pick up debris. In consideration of the diversity of debris, the vehicle’s frame height was raised by approx. 300mm so as to allow the mounting of the removing equipment underneath the frame. This design reflects difficulties securing vehicle safety certification which can often be encountered with front-mount applications. Also, difficulties picking up debris were considered with rear-mount apparatuses. Further, front-mount designs would limit the types of debris due to the set amount thereof to be removed. Significantly poorer vehicle safety during removal operation carried out at higher travel speeds was another disadvantage associated with front-mount designs. The rotating brushes at the front (Fig. 9) draw the debris in and push it underneath the frame. Once underneath, the debris will be transported onto the conveyor belt via underside collection tool and then to the freight space. For hygiene purposes, the design provides separate cargo

space for animal carcasses and ordinary debris items.

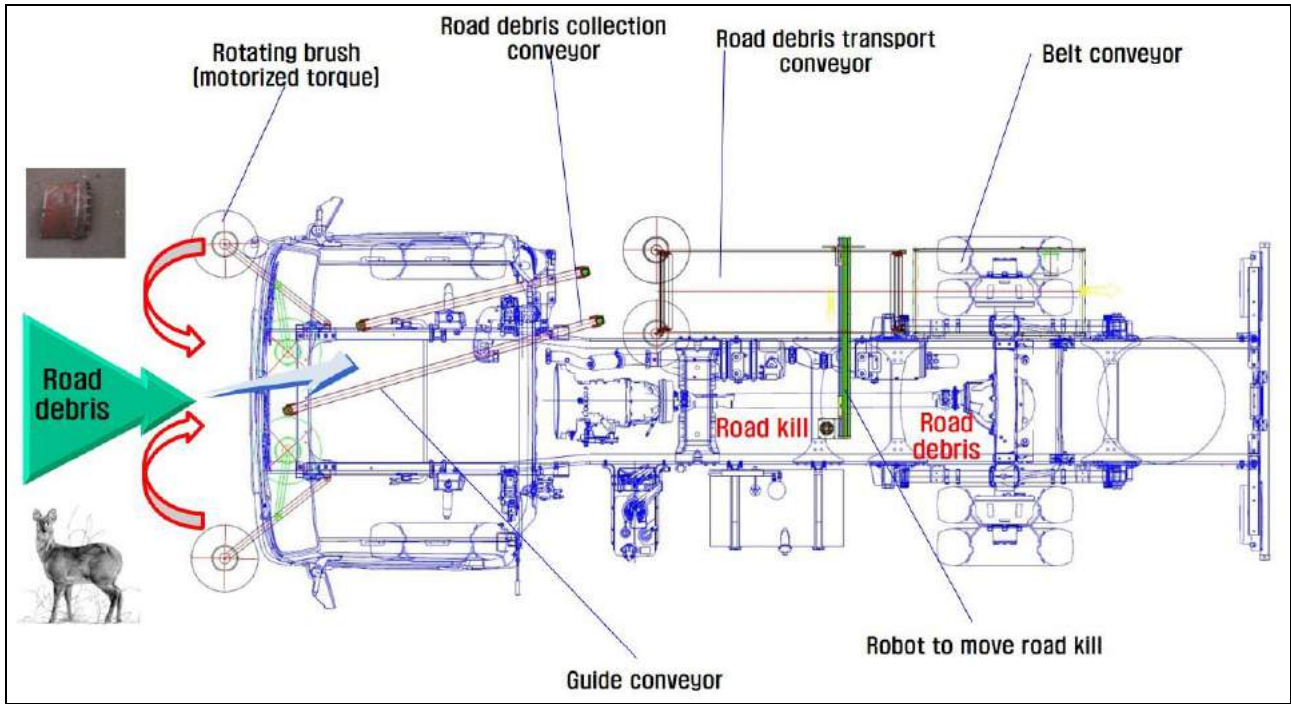


Fig. 9 Road Debris Remover Instrument (Plan View)

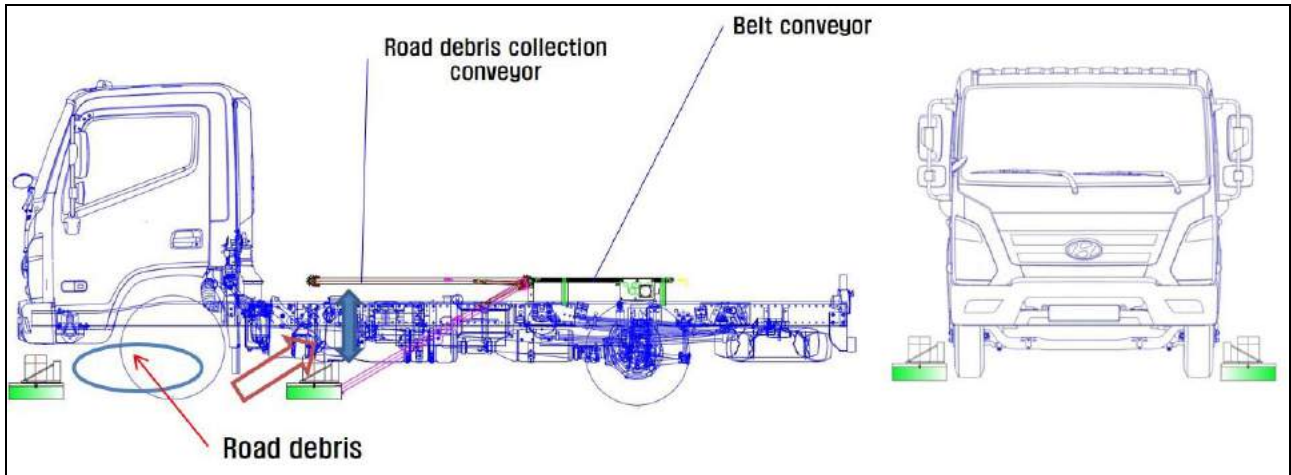


Fig. 10 Road Debris Remover Instrument (Elevation View)

5. CONCLUSION

Despite the government's efforts to control overloaded vehicles on the road in South Korea, the number of road debris-induced accidents is increasing every year. Such accidents being an extremely likely cause of massive accidents, preventive measures should be established to protect the drivers as well as the highway maintenance crews against safety hazards.

The purpose of the study was to propose a layout design for a debris removal tool that is suitable for the road condition in South Korea. To this end, surveys were conducted targeting the field professionals working for the road maintenance agencies. The results were incorporated into the instrument layout design. Although some local agencies are already using debris collection instrument, its safety performance is yet to be validated. For that reason, such instrument is being used only in a limited capacity, maintenance of specific roads or uninterrupted flow. The conventional instrument also poses possible hygienic problems in the latter stage of debris disposal due to the non-segregating approach being adopted whereby ordinary debris items are removed and stored along with wildlife animal carcasses. The current sweeping machines can only pick up relatively smaller objects such as metal scraps, glass, and debris remaining on the shoulder after automobile accidents. Contrastingly, the study proposed a design of the road

debris remover instrument that is capable of cleaning up even bulky animal carcasses, tire pieces, boxes, and wood panels. Such capability will likely enhance the work of the road maintenance. Further, the proposed instrument can accurately identify the type of debris present on pavements and can collect them swiftly. The speed and accuracy will likely offer greater efficiency in the secure of the road management and traffic safety.

ACKNOWLEDGEMENT

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Slope Maintenance Program is the key factor on choosing the Retrofitting Technology comply with the standard requirement laws for road user safety and comfortability.

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Abstract

Slope maintenance program is usually come up with the chosen solution of slope protection against landslides, including debris avalanche falls. This is probably an importance activity to be setup in advance in order to manage the stability of slope against failures occurrences to achieve the traffic safety and road user comfortability. Dealing with the evidence of avalanched debris material falls which is one of slope failure evidence, a number of chosen solutions have to be implemented either as the temporary solution and the permanent solution for maintaining functionally roads. An importance task is usually by implementing such kind of the retrofitting technology using the reinforced structures to support the increasing slope stability. Due to the limitation of the retrofitting budgeted cost and the other hand, the road network functionally have to be always maintained, hence the temporary retrofitting solution have to be priority chosen. The protecting avalanched debris falls is available and have been studied working well in order to perform road safety and road user conformability. Hence, Fence Defense and Catchment Area were effectively well implemented for temporary solution, especially to protect the occurrences of debris avalanche falls in the national Road in East Nusa Tenggara Island of Indonesia.

Key word: *slope maintenance, road user, importance task of implementing technology, traffic safety and road user conformability, avalanched debris falls, road network functionally and retrofitting technologies*

Introduction

The slope maintenance programme is likely a good practice to be implemented on carrying out the traffic safety and road user comfortability against landslide evidences due to the traffic will be blocked. The performance of the slope stability either natural and man-made slopes is usually implementing the retrofitting technology such as retaining walls and other reinforced strength structures to support the stability of slopes.

Basically, the maintenance activity is the programme for inspections and maintenance works to keep the slopes are in good condition with well-designed to achieve the traffic safety and road user comfortable. Hence, the inspections and maintenance works are recommended within road networks, and in the mountainous region the instability of slopes will be concerned due to can impact to the landslide evidence and come up with the blocked traffic occurrences. However, the slope have been designed comply with the currently geotechnical standards, the occurrences of landslide is still happened due to the number of many factor that influenced instability of slopes. As previously mentioned in this paper that the functionality road network was the main concern to be solved, therefore, one of the area have been studied which is as the national road networks of the link road connected between Ende City and Kelimutu village, exactly in Nuamuri Village, Kabupaten Ende at KM

46+000 have been experienced with the debris avalanched falls. The material falls have been recognized from the exposed rock on the outcrop slope and reported by Tribun News Kupang magazine on Thursday (3/3/2016). Based upon the record during study in 2008 can be also noted that the currently landslide was not the first evidence due to the similary landslide is commonly occurred at KM 13+000 and KM 76+000. Hence, along the road networks were identified a number of potential landslide can be occurs which is in Km 13+000 and KM 76+000 are critical due to the traffic can be blocked within uncertainty period of time. Regarding to this evidence, in 2008, the research by IRE have been implemented in those 2 (two) locations, using the temporary solution which is stressing to maintain the functionally road with performing the traffic safety and road user comfortable. This site location of road networks between Ende city to Maumere city have been noted there are many high slope with the outcrop rocks are un-well jointed and hanging on the slope outcrop, therefore potential to occurrences of debris avalanched falls into the roads will be impacted to the dangerous of road users. The occurrences of debris avalanched falls have been reported by Tribun News Kupang (2014) at KM 17 on Sunday (23/2/2014) and Pos Kupang magazine KM 46+000 on Thursday (3/3/2016). Hence, to come up with the solution, the principle retrofitting technologies were applied is by implementing a temporary solution to perform the traffic safety and road user comfortable besides the maintaining functionally roads.

Dependant upon the size of landslide evidence can be small or big scales, obtaining the retrofitting technologies were recommended by Institute of Road Engineering (IRE), Agency for Research and Development, Ministry of Public Works and Housing based upon the regulation laws on implementing the minimum standard of level services that to be adopted.

The idealism on problem solving of slope failures, there are 2 (two) options can be suggested, which is no 1 (one) bay protecting the slope failure evidence it self and number 2 (two) by implementing technology on suffering the road user from the debris avalanche falls. The first one is indeed as the structural retrofitting countermeasures which might be costly while 2 (second) is indeed less costly which is on supporting the functionality of the road networks against blocking current traffic flows.

National Road Situation in Trans Ende – Maumere, at East “Nusa Tenggara” Province

The occurrences of debris avalanched falls have been reported by local news paper either from Tribun News Kupang magazine (2015) and Pos Kupang magazine (2016) which have been previously noted by IRE (2008), in East “Nusa Tenggara” Province of Indonesia with the typical of the material slides contain with the avalanched debris combined with the fractured rocks, boulders mixed with sands and soils developed by volcanic breccias and metamorphic rocks were caused by un-jointed rock fragments.

During study on the implementation of the Slope Maintenance Programme by IRE in this National Link Road between Ende to Maumere in 2008, such as showed in Figure 1 are noted there are 2 (two) critical conditions with the occurrence huge avalanched debris falls which is each location are shown in Figure 2 for KM 13+000 and Figure 3 for KM 76+000.

Hence, Its can be stated refers from IRE research in 2008, the location of landslide at KM 17+000 and KM 46+000 have been identified as small potential avalanched debris falls. Within the following years such as showed on Figure 4 for KM 17 was reported by the Tribun News Kupang (2014) and Figure 5 for KM 46 was reported by the Pos Kupang (2016), those locations have been developed to become the huge landslide evidences due to changes on its material character manners as an impact of the environmentally changes.

On Figure 5 also showed the heavy equipment are ready for clearing up the Debris Flows which were always blocked the existing road and can be dangerous for road users against the safety and decreasing their conformability.



Figure 1. the Trans national Road between Ende – Maumere, East Nusa Tenggara Provinces, Indonesia (IRE, 2008)



Figure 2. Avalanched Debris Falls at KM 13+000 Ende, trans road Ende – Maumere, East Nusa Tenggara Provinces (IRE, 2008)



Figure 3. Avalanched Debris Falls at KM 76+000 Ende, Trans Ende – Maumere East Nusa Tenggara Province, Indonesia (IRE, 2008)



Figure 4. Landslide Evidence at KM 46+000 Ende, East Nusa Tenggara Island, Indonesia (Pos Kupang, 2016)



Figure 5. The Evidence of landslide at other side of KM 17+000 Ende during road widening progress, East Nusa Tenggara Province, Indonesia (Tribun News Kupang, 2013)

Main Principle of the Implementation of the Retrofitting Technology against the Debris Avalanched Falls.

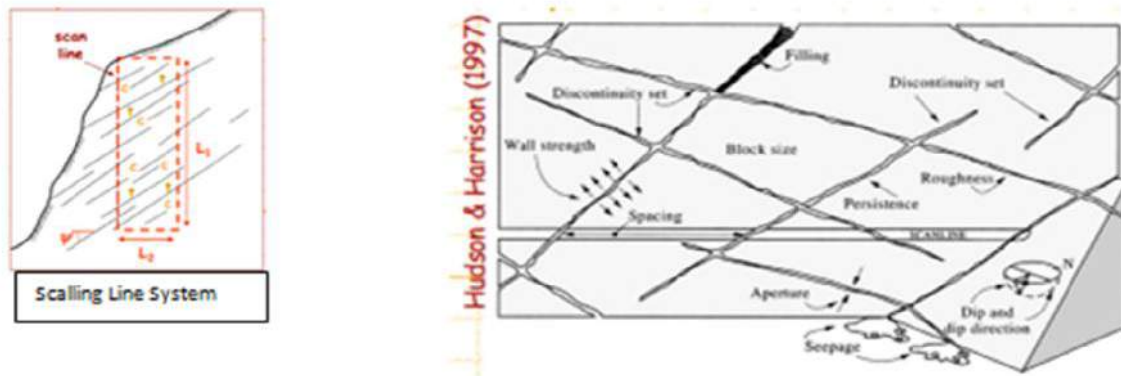
There were previously noted that the implementation of the Retrofitting Technology with less costly even might be temporarily solutions which is mainly based upon the concerning on full filled the regulation laws to the requirement to obtain the standard minimum of road level service which is meant on managing the potential of blocking road traffic flows. The regulation laws are stated as The Indonesian Government Regulation Law no 22, 2009 and the Ministry of Public Works no. 11, 2009.

Refer to the research results regarding to the retrofitting technology against those regulation laws (IRE, 2008), there are a number of retrofitting technologies can be adapted which is principally concerned to the traffic safety and road user conformability, such as:

- 1) Technology on protecting the avalanched debris falls using shotcrete technology which is suitable for potential debris avalanched falls by filling the gaps between the crack rock fragment joint and changes on material character manners due environmental changes which are usually intended to the increasing degree of weathering
- 2) Technology on managing the avalanched debris falls, which are containing Fence Defense Technology and Catchment Area Technology which are intended suitable for small to medium dimension of debris avalance material sizes.
- 3) Technology on protecting the avalanched debris falls using Rock Netting techniques which is intended for the small dimension of debris material sizes

The techniques before starting on choosing the suitable retrofitting technologies, based upon the IRE research (2008) are by measuring the outcrop rocks, such as by measuring the jointed patterns by determining the value of RQD (Rock Quality Designation). The measuring RQD is stated the ability of slope on performing stand up time alone during period of time without any doubt. The other method on measuring the rock quality designation (RQD) can

be done using the Scaling System as they have closed relationship correlations, as showed bellow on Figure. 6. If the measuring scaling is between thos values then can be interpotated.



Measurement of Joint Cracks	20 / meter	Measurement of RQD Value (%)	< 25
	11 / meter		25 - 50
	5 / meter		50 - 75
	2 / meter		75 - 90
	1 / meter		> 90

Figure 6. Measurement of RQD values using the Scaling System Approach on the Outcrop Rock Slopes in meter length (IRE Research, 2008)

The result from measuring the RQD (Rock Quality Designation) on the outcrop rock slope Surface in each location in 2008 by IRE showed in Table 1 for 4 (four) locations were categorized as the dangerous risk for safety and un-comfortable for road user due to occurrences of the Scaled value from 2018 to 2016, which is road traffic flows can be blocked within the mater of time, especially in raining season and can be also impacting to the road safety and the user road conformability.

Table 1. Measurement of the Scaling and RQD on the Outcorp Rock Slopes

Location	Outcorp Condition	2008	2010	2014	2016
KM 13	Jointed Cracks/m length	11	20	20	20
	RQD values	25 - 50	< 25	< 25	< 25
KM 17	Jointed Cracks	2	5	11	20
	RQD values	75 - 90	50 - 75	25 - 50	< 25
KM 46	Jointed Cracks	2	5	11	20
	RQD values	75 - 90	50 - 75	25 - 50	< 25
KM 76	Jointed Cracks	11	20	20	20
	RQD values	25 - 50	< 25	< 25	< 25

Hence, referring to the result on measuring the RQD value based on the Scaling Method Approach are shown that the location at KM 13 and 76 have been decided to be immediately retrofitted in advance if compared to the location at KM 17 and KM 46. Further, referring to the period of time in relation with the RQD value of KM 17+000 and Km 46+000 are dropped significantly as a result of its change in character manners due to environmental changes.

The implementation of the retrofitting technology types against the occurrences of Avalanched Debris Falls at KM 13+000 and KM 76+000

Referring to the regulation law by Indonesia Government and the implementation of regulation law on road building refers to the standard minimum road services, hence the priority on the adapting retrofitting technology using “Fence Barrier and Catchment Area” or “Buffer Zones” were concerned in order for improving against the traffic safety and road user conformability.

Based on the research results by IRE, there were 3 (three) typical technologies have been implemented in Full Scale Study, (IRE, 2008) and showed on the following figures as a Buffer Zone Techniques on bellow using Fence Defense and Catchment Area Technologies as showed on Figure 7.



Figure 7. Buffer Zone Techniques (IRE, 2008)

The types of those retrofitting technology on above are believed will be not costly and effective if compares to the typical countermeasure technologies based upon the aims on increasing strength to improve the slope stability against the avalanched debris falls, such as anchoring and grouting techniques and as well as “shotcrete”..

Based upon the monitoring program in 2013 in order to recognizing the performance of the retrofitting technology using either Fence barrier and Catchment Area, have been known that the local Government through the EINRIP (Eastern Indonesia National Roads Improvement Project) programme in 2013, has adopted one of those technologies at KM 17+000 using Fence Barrier by modifying with “Earth Retaining Wall Structure”, showed on Figure 8. However the type of Fence Barrier is differ but on principle is still similar for protecting the avalanched debris falls and the following concerned should be noted:

- 1) The material debris can be dumped on the specific area between the buffer zone construction and the slope surface and might be can develop the New Slope with gently surfaces. The dumped material can be regularly thrown away if necessary or can be left to develop the gently slope.
- 2) The water might be trapped behind the buffer zone construction should be able to drain out therefore
 - a) The buffer zone or buffer zone constructions should have the facility to drain of the trapped water or having the enough permeability
 - b) The function as the Earth Retaining Structures, the facility such as the weep holes have to be included to drain out the trapped water.



Figure 8. Implementation of Fence Defense as Earth Retaining Structure at KM 17+000 by local Government Authority (IRE, 2013)

Further Action to Defining the Performance of Retrofitting Technology on carried out the Avalanched Debris Falls

Regarding to the retrofitting technologies using Fence Barrier and Catchment Area have been implemented by IRE in 208, to fulfill and comply with the regulation laws on achieving the minimum standard level of servicing therefore, should have also fulfilled to the road safety and road user comfortability. Hence, the maintenance and repairing works then can be set up as part of inspection programme within the passage of time.

Regarding to the slope stability on these location which have been distinguished 4 (four) locations with the high potentially rock slope failures, hence, the maintenance activity should be prepared along the road for each site location, KM 13+000, KM 17+000, KM 42+000 and KM 76+000, within the inspection programmes (Layman's, 2006):

- (a) Routine Maintenance Inspections, which can be carried out by any responsible person with professional geotechnical knowledge. This is
- (b) Engineer Inspectors for Maintenance programmes should be carried out by a professionally-qualified geotechnical engineer,
- (c) Regular Check of Buried Water-Carrying Services should be carried out by a specialist leakage detectiocontractor for the fence barrier structure which might be performed as earth retaining structure walls.
- (d) Regular Monitoring of Special Measurement should be carried out either by a firm company, or individually with a special expertise in particularlu to detecting the performance. Hence, the monitoring programmes to be set-up for determining either short-term or long term stability of the slope. If the retaining wall structure has becoming implementd, to be noted that the rating values have to be relies in specific measurement due to the liable on detecting the retrofitting technology performance that might be less effective within the passage of time.

Conclusion and Recommendation

- 1) The requirement on building and managing the Road Networks comply on urging the statement of standard minimum level of services against road safety and road user conformability, the investigation of the avalanched debris fall can be rates and grouped if necessary in order on choosing the suitability retrofitting technologies.
- 2) The measuring the Rock Quality Designation (RQD) is necessary by the measurement of Jointed Cracks of Rock Pattern using Scaling System then can be useful to define the Suitability Retrofitting Technology comply to accommodate the evidence of Avalanched Debris Falls occurrences.
- 3) The types of retrofitting technologies using the buffer zone techniques are as a suitable technology in order to effectively maintain the road network to be safety and conformable for road user and otherwise un-costly compare with the reinforced technologies.

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PAPER TITLE (90 Characters Max)	Performance Based Key Performance Indicators for Expressway Concessions in Malaysia		
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KEYWORDS: Key Performance Indicator, Performance Based

ABSTRACT:

The paper outlines the study undertaken for the introduction of performance based KPIs for monitoring which emphasizes the Malaysian Highway Authority's governance role while at the same time will encourage expressway Concessionaires to adopt a preventative approach to the maintenance of their assets. Efficiencies in terms of maintenance and upkeep of the expressway assets can be gained if all Concessionaires are monitored under appropriately similar KPIs and employ compatible asset management systems and procedures in line with their respective Concession Agreements.

1 Background

The Malaysian Highway Authority (MHA) was incorporated by the Malaysian Act 231 (Highway Authority of Malaysia (Incorporation) Act 1980). The Authority was initially established for the supervision and execution of the design, construction, regulation, operation and maintenance of inter-urban highways with the objectives of:-

- i. Providing a fast, safe and efficient transportation system on a national scale;
- ii. Connecting all major towns and their surrounds for economic, cultural, social development and national unity; and
- iii. Enabling an effective inter-urban public road transport system throughout the country.

All Malaysian tolled expressways are managed through the Design Build Operate and Transfer (DBOT) model where concessions were given to private companies to finance, build, operate and collect tolls for a stipulated duration ranging from 30 to 50 years. The Concessionaires are obliged to operate and maintain their expressway facility based on the agreed Concession Agreements.

In 2013, Concessionaires under MHA each employ their own maintenance management systems and procedures. Key Performance Indicators (KPIs) are not fully established in their Concession Agreements resulting in dissimilar performance targets being adopted by concessions in the course of maintaining their assets.

In general, MHA recognizes that efficiencies in the governance role and upkeep of the expressway assets can be gained if all Concessionaires are under appropriately similar KPIs and employ compatible asset management systems and procedures in line with their respective Concession Agreement obligations. These objectives will be achieved by a move to a performance based process that:-

- Minimises auditing and inspecting tasks undertaken by MHA of the expressway networks and moves it to a monitoring / governance role;
- Obligates expressway Concessionaires to maintain their networks including identification, recording, programming and reporting of works to meet agreed minimum service levels; and
- Encourages expressway Concessionaires to:-
 - adopt a preventative approach in the maintenance of their assets to minimize larger failures which may result in longer operational distress or interruptions; and
 - adopt contemporary asset management principles in optimizing maintenance activities in a KPI achievement based framework.

1.1 Introduction

Opus International (M) Bhd and Ace Vector Sdn Bhd (Opus-AVSB) was appointed by MHA to provide the consultancy services for the development of Key Performance Indicators (KPIs) required to support the governance role of MHA within the framework of the Concession Agreements signed with the Highway / Expressway Concessions. The constituents of the highways/expressways covered under this scope of services comprise pavement, slopes, bridges, drainage culverts and tunnels (inclusive of related mechanical and electrical) assets. The SMART Tunnels are not part of the scope of this study.

The scope of services is broken into distinct phases to enable MHA to meet its objectives. The total duration of the project was 24 months comprising of Stage 1 (Phase 1 to 4) and Stage 2 (Phase 5). The distinct phases are shown below and further description is provided in the following sections.

Table 1.1: Summary of Project Stages

Stage	Phase & Description
1	Phase 1 – Project Scope Phase 2 – Monitoring Framework Phase 3 – Performance Measures Phase 4 – Supplementary Documentation
2	Phase 5 – Roll Out (Pilot Program)

Phase 1 - Project Scope

This phase focused on discussions for mutual understanding of the objectives between MHA and the Consultant's team. In this phase, the scope of services was defined by the Consultant in terms of objectives, methodology and work programme taking into account the kick-off discussion with the Client.

Phase 2 - Monitoring Framework

The objective of Phase 2 was to derive a monitoring framework for the Client to adopt. The monitoring framework allows the Client to enhance its governance role in managing the concession agreements and reduce the amount of on-site activities such as the detailed inspection and identification of defects they are currently required to undertake.

Phase 3 - Setting of Minimum Service Level

The objective of Phase 3 will be to assess and recommend minimum performance levels or service level values for each of the previously identified measures.

The derivation of the level of services will be based upon the following aspects.

- Best practice elsewhere;
- Current requirements of the Concessionaires;
- MHA's current requirements (e.g. MHA Manual M1, Expressway Maintenance System, Maintenance Manual and Guidelines (Civil Works), September 1996); and
- Other factors (in discussion with MHA).

Consideration will also be given to the situation where an expressway network has fallen below the derived or agreed performance values, the manner that the network could be brought up to the required standard, including the time frame to do this. These aspects will take into account the terms of the prevailing Concession Agreements.

Phase 4 - Supplement to Existing Guidelines and Documentations

Appropriate supplementary documentation to the MHA's Expressway Maintenance System Civil Works Guideline and Manual (M1) and the Expressway Maintenance System Civil Works, Guideline and Manual (Response Time) will be identified upon the performance measures and service level values being determined and agreed.

Phase 5 - Roll Out

A pilot program to be undertaken on appropriate stretches of expressway to assess the most appropriate manner for implementation of the Key Performance Indicators.

2 Current Highway/Expressway Network

There were 29 expressway concessions under the scope of the study as shown in **Table 2.1** and **Figure 2.1**. The network comprises of the expressways traversing from north to south of Peninsular Malaysia, i.e. the North-South Expressway and from east to west, i.e. the East Coast Expressway. The majority of the expressways (20 nos.) are situated within or around the capital city of Kuala Lumpur as shown in **Figure 2.2**. The length of the expressways ranges from 5km to 823km.

The characteristics of these 29 expressways vary from large numbers of steep high slopes to no slopes and some with high proportions of structural elements. The pavements of these expressways are a mix of flexible and rigid.

Table 2.1: Highway/Expressway Concessions

No	Route No	Expressway	Length (Km)	Abbreviation
1	E1&E2	North South Expressway	823.0	NSE
2	E3	Malaysia Singapore Second Crossing	44.7	LINKEDUA
3	E5	Shah Alam Expressway	34.5	KESAS
4	E6	North South Expressway Central Link	48.0	ELITE
5	E7	Cheras Kajang Expressway	11.7	CKE
6	E8	Kuala Lumpur - Karak Expressway	60.0	KLK
7	E8	East Coast Expressway	169.0	ECE
8	E9	Sungai Besi Expressway	16.7	SBE
9	E10	New Pantai Expressway	19.6	NPE
10	E11	Damansara - Puchong Expressway	40.0	LDP
11	E12	Ampang Kuala Lumpur Elevated Highway	7.9	AKLEH
12	E13	Kemuning - Shah Alam Highway	14.7	LKSA
13	E14	Johor Bahru Eastern Dispersal Link	8.1	EDL
14	E15	Butterworth - Kulim Expressway	17.0	BKE
15	E17	Butterworth Outer Ring Road	12.1	BORR
16	E18	Kajang SILK Highway	37.0	SILK
17	E20	KL - Putrajaya Expressway	26.0	MEX
18	E21	Kajang Seremban Expressway	44.3	LEKAS
19	E22	Senai-Desaru Expressway	77.0	SDE
20	E23	Western KL Traffic Dispersal System	26.0	SPRINT
21	E25	Kuala Lumpur - Kuala Selangor Expressway	31.0	LATAR
22	E26	Southern Klang Valley Expressway	51.7	SKVE
23	E29	Seremban - Port Dickson Highway	22.7	SPDH
24	E30	North Klang Straits Bypass	17.5	NKSB
25	E33	Duta Ulu Kelang Expressway	18.0	DUKE
26	E35	Guthrie Corridor Expressway	25.0	GCE
27	E36	Penang Bridge	13.5	PNB
28	E37	East West Link Expressway	17.0	METRAMAC
29	E38	SMART Tunnel	5.0	SMART

Note: Tunnel in SMART is not in the scope of the study



Figure 2.1: The Highway/Expressway Network



Figure 1.2: The Highway/Expressway Network within /around Kuala Lumpur

2.1 Concession Agreements

The Concession Agreements were reviewed in terms of the following subject matters.

- Asset management
- Asset condition (Data Collection)
- Level of service/Performance measures
- Key performance indicators
- Routine maintenance
- Monitoring
- Non-conformance
- Reporting
- Systems and processes

In general, while there were differences between the CAs with regard to funding, financing and toll rates, the observed differences in terms of maintenance requirements between the earlier concession agreements are in terms of the preparation of maintenance manuals. After 1991, all CAs were referred to the MHA's M1-Maintenance Manual and its subsequent publications.

3 International Best Practices

The objective of researching and reporting current international best practice in the field of performance monitoring of road networks is to give MHA an insight into what might be achieved through the introduction of a performance monitoring regime.

Performance based specifications were introduced to manage road assets approximately 20 years ago. Australia, The United States and then New Zealand and a number of other countries recognized the benefits in terms of the management of their road assets and started moving from traditional forms of contracting to performance contracting. This review and the lessons learnt and will give MHA a sound basis upon which to make decisions about moving the management of tolled expressways in Malaysia to a performance based monitoring system.

The countries and projects reviewed were:

Table 3.1: Countries Reviewed in International Best Practice Study

Item	Country	Projects
1	New Zealand	NZ Performance Specified Maintenance Contracts (PSMC), Western Bay of Plenty, Auckland Motorway Alliance, Lyttleton Tunnel
2	Australia	Hybrid PSMC New South Wales, Queensland Motorway Road Franchise
3	Canada	Ontario, British Columbia, New Brunswick Public Private Partnership (PPP)
4	United Kingdom	Highways Agency's Asset Support Contracts (ASC)
5	India	World Bank Output Performance Road Contracts (OPRC)
6	Turkey	Trans European Road Network (Tunnel)

3.1 Lessons Learnt from International Best Practice

Over the years various countries have put in place performance based contracts many lessons have been learnt. Often the lesson has been drawn from trial and error as road controlling authorities strive to achieve value for money while keeping road users safe and comfortable. Key lessons learnt are described in the following sections.

3.1.1 Roles

Worldwide the maintenance and asset management of road works is split between two or three key parties. Usually they are:

1. Road Controlling Authority (client)
2. Consultant (asset manager)
3. Contractor

Each brings different expertise to the team. In order to recognize this expertise and provide clear responsibilities it is important that separation is achieved between each of the roles.

In general the contractor is best at and suited to implementation, that is carrying out the physical works on the network. It is important that they do things right by using the correct standards of workmanship. The consultant brings asset management skills into the mix as well as long term understanding of asset characteristics and performance over time. They need to ensure the right treatment has been selected in the right location. The road controlling authority, or client organization, has valuable connections to politicians and the community. They are in a good position to translate the preferences and demands of political and community bodies into desirable contract outcomes.

This separation of roles can be illustrated against an asset management continuum as shown in the following illustration.



Figure 3.1: Asset Management Continuum

Inefficiencies will occur when a clear separation between the roles is not achieved. From experience duplication of effort results and the true value from each party's expertise is marginalized.

3.1.2 Culture Shift

In each of the countries that have moved from traditional method specified maintenance to performance based management a significant culture shift, across the whole industry has been necessary.

Just as the road controlling authority struggles to keep out of the detail the contractor has often struggled to think and act proactively. They have been used to being told what to do, where and when. Their challenge is to take responsibility for short term programming of works, selecting cost efficient work techniques and organizing resources as effectively as possible.

Often the change to performance management comes with a change to the procurement or contractual arrangements between parties. Relationship based contracts are favoured worldwide to extract maximum value from the parties involved. In this arrangement working cooperatively, with risk sharing and best for network decisions being made, people involved need to cast aside old relationship roles and embrace a more open working environment.

3.1.3 Service Levels

In the early days of performance based management it was thought a road network should and could be maintained in perfect condition at all times. Through experience this has proven to be an extremely difficult aspiration that has also been expensive to achieve. It is now fairly well acknowledged that a network can never be 100% perfect all the time. This is particularly so since many defects have to be evident before they can be fixed. For example – a missing or damaged sign, a damaged length of guardrails.

A level of service is set to recognize the varying needs of a particular part of a road network. Factors varying a level of service often relate to traffic volumes, location (urban/ rural), and geotechnical conditions. It is important that three areas are addressed when setting a service level.

1. Road user safety and comfort – those using the road need to feel safe and to enjoy a comfortable ride – particularly if they have a choice of routes to use
2. Durability (or resilience) – the road has to last with minimal intervention in its life time to keep it usable. The asset has to be maintained in an optimal condition and managed to minimize whole of life costs
3. Information flows – a successful performance managed road network relies on good flow of information between all parties. However, a poorly chosen level of service that requires excessive reporting will draw effort away for the core functions of the management team. Ensure the level of services reflects a desired outcome and is realistically measurable

Another big challenge in setting service levels is to avoid setting a level that is greater than actually required. Experience has proven that capturing ‘what usually happens’ on a network into ‘what must happen’ incurs a cost increase. Good practice has proven to be to define key objectives or Customer Service Levels, and then set performance measures to achieve these.

3.1.4 Monitoring

A number of international best practice examples include response times for rectification of identified defects. While the concept of response times is not hard to comprehend, monitoring of response times has proven to be time consuming and complex. Response times generate what can be considered a ‘two pass’ monitoring and auditing regime. The first pass identifies the defect. The second pass is made at the end of the specific response time to ensure compliance with the given time period for rectification.

The two pass system is made complicated by the varying response times on each defect. This can be illustrated by considering two separate defects within say 50 m of each other. The first a pothole with a 24 hours response time while the second a missing sign with a 7 day response time. In order to assess true compliance with the response time the auditor would be required to return to the defect site after 24 hours for the pothole and again after 7 days for the sign replacement. Monitoring of response times becomes a full time occupation.

The solution to address both issues was to change the monitoring regime to a ‘one pass’ system and a level of allowable defects. A move to defects per unit of length was adopted. Returning to the example of the pothole and missing sign within 50m of each other the one pass regime would require the auditor to record the location and nature of each defect and then continue to the next similar defect and record that until the audit was completed. The audit data would then be assessed against the performance measures which would state

the allowable number of potholes over a certain length, or the allowable number of missing signs over a given length of network. No further requirement to go back out onto the network until the next audit. An assessment of compliance is achieved in 'one pass' thus reducing the contractors self-auditing, or monitoring time and similarly reducing the road controlling authorities auditing time.

Compliance results are generally repeatable if the site inspections are undertaken within similar time periods.

3.1.5 Measuring

Human nature appears to compel us to take action if we know we are going to be assessed or monitored. Similarly in a performance based environment what gets measured gets done. Or more importantly 'what does not get measured does not get done'. Knowing this human tendency it is important to set performance measures that actually reflect what the road controlling authority wants.

Ultimately these performance measures need to relate to the levels of service which have been derived from an understanding of what the customer (road user) wants. The challenge is to set performance measures that encompass or drive desirable behaviour and results without having to specify excessive numbers of measures.

It is also ideal to have a mechanism to change or modify performance measures throughout the contract period. In the New Zealand performance models this mechanism is achieved through the Management Board. Successful performance measures are ones that are agreed by all parties. It is therefore important that each party bring to the table their concerns relating to the way performance measures have been set in order to make improvements.

3.1.6 Conformance

Three strikes and you are out has been a typical instrument for ensuring conformance with performance measures in a number of performance based contracts. This simply means that when the contractor fails to comply three times with performance measures the contract is terminated. It sounds harsh and in reality it is such a huge step to take to terminate that the 'three strikes and you are out' has rarely been used.

Despite the difficulties in implementing this conformance framework it remains critical to the success of a performance based contract to have some form of conformance, or non-conformance framework. Consequences of not meeting performance measures remain influential drivers in contractors taking responsibility and undertaking what they are being paid to do.

The solution developed to date has been called 'the non-conformance bucket'. Essentially this assigns a weighting to each type of non-conformance and allows the contractor to build up to a number of non-conformances before a financial penalty is triggered.

Network conformance is measured each month via the following methodology.

1. The contractor audits 10% of his network
2. Each non-conformance identified, on any asset, has a predetermined value. Each defect is weighted to reflect significance
3. The identified non-conformances are 'placed' in the bucket containing the at risk money
4. In the month if the volume of liquid fails to reach the top of the bucket and overflow then the network conforms. Too many individual non-conformances and the liquid overflows and the contractor loses some at risk money

If the road controlling authority identifies a non-conformance not previously identified by the contractor the weighting is greater than for the same non-conformance has the contractor identified it.

This bucket system recognizes that a network cannot realistically be 100% compliant at all times and gives the contractor flexibility to manage resources, plant and materials efficiently.

4 Monitoring Framework

The Monitoring Framework sits within a wider set of inspection regimes. A three tier inspection regime is most commonly adopted in performance based contracting. They are:-

Patrolling – the Contractor’s day to day inspection of the network. Patrolling aims to identify defects that are then programmed for rectification. Frequency and inspection methodology are usually at the discretion of the Contractor but he must ensure his patrolling regime enables him to meet the target levels of service.

Monitoring – undertaken by the Contractor to assess compliance against target levels of service. Asset inspections are undertaken in accordance with the details of individual performance measures but commonly compliance is reported on a monthly basis.

Auditing – this activity is undertaken as part of the governance role. It seeks to confirm that the level of compliance reported through the Contractors monitoring process is accurate. It usually involves the selection of a random audit length in the network and an independent assessment of compliance in the audit lengths.

Levels of service are set to define minimum values to which the Concessionaire must manage their asset condition to. However, he is able to make decisions about when and where to invest in his maintenance works. His investment decisions will be based on his understanding of asset condition through condition data collection and his calculation of whole of life costs for various treatment solutions and the consequences of not meeting the targets.

Evidence from New Zealand, India and Canada is that the ‘non-conformance bucket’ is driving desirable behaviours and that assets are being maintained at the ‘correct’ level of service which had been set. Success of the bucket system will be assisted by the ability of the Principal to place an ‘at-risk’ payment in the bucket initially.

While a financial penalty system has been introduced on a small number of the Expressway Concessions, it is understood there is very little, if any, attempts to make similar changes to all of the existing Concession Agreements (CA) for Expressways in operation. The amendments made to the several CAs mentioned were mostly due to a revision in scope mostly in terms of additions to the lengths of individual Concession network. Without financial consequences, the ‘non-conformance bucket’ would rely on ‘name and shame’ to drive Concessionaires to comply with performance measures. While ‘name and shame’ may not be as powerful as a financial penalty it is highly likely that Concessionaires would be motivated to be reported as operating as a compliant network.

4.1 Recommendations for MHA Monitoring Framework

The following recommendations were made based on the findings as follows:-

4.1.1 Concession Maintenance Patrolling

The Concessionaire is responsible for identifying defects, programming repairs to meet performance measures (and response times where applicable) and for carrying out the repairs. In order to identify defects for rectification they need to patrol their network. The form the patrolling takes will vary from asset to asset and will be dictated by the nature of the performance measures. Whatever form patrolling takes, the frequency is the responsibility of the Concessionaire.

Pavement OPMs, some Street Lighting and Tunnel performance measures may require patrolling on a daily basis. This is likely to take the form of a vehicular drive over identifying and locating defects. These records are then used to program rectification works.

Where constant evaluation of compliance is required, for example air quality within a tunnel, the Concessionaire may opt to mount permanent equipment from which adjustments to ventilation can be made from real time data.

For assets such as culverts, bridges, street lighting, tunnels and slopes periodic site inspections will be required, at times under temporary traffic management. Again inspection records are compiled and defects requiring rectification are fed into the works program.

Pavement Durability Performance Measures require the Concessionaire to patrol the network using special equipment (usually high speed data collection). This will be undertaken on a less frequent basis and most Concessionaires will opt to collect such data to meet condition monitoring and reporting requirements (typical - annually). However, where a Concessionaire feels they need to build their forward works program with more frequent collection of pavement data they may opt to undertake high speed data collection at a frequency greater than that required for their own condition monitoring purposes.

Management Performance Measures do not in themselves require any form of patrolling but the Concessionaire will need to put in place a comprehensive program of reporting requirements in order to ensure compliance.

4.1.2 Concession Monitoring

The objective of Concession Monitoring is for the Concessionaire to determine compliance against the target levels of service. Compliance is then reported to MHA on a regular basis. As each concession is already in place MHA do not have a great deal of flexibility to introduce financial penalties through the Non Conformance Bucket. However, the principles of the Non Conformance Bucket do not vary a great deal from the concept of the current MHA performance assessment system. Therefore the proposed methodology draws on both the current system and the Non Conformance Bucket specifically for MHA to introduce into all Concessions.

4.1.3 Non Conformances

The proposed Monitoring Framework is not built on the identification of defects but the identification of non-conformances. A non-conformance is when the number of defects exceeds the target level of service. Drawing away from the identification and recording of individual defects allows the Monitoring Framework to cover all the asset groups and include the network wide performance measures, such as the Pavement Durability Performance Measures (PDPM) and the Management Performance Measures (MPMs).

Non Conformance by Asset Group

The effect on an expressway and the impact on a road user of non-conformances from the different asset groups can vary significantly. Comparison of non-conformances from different asset groups is therefore difficult. To overcome this and avoid comparisons being made it is proposed that non-conformance is assessed and reported by asset groups.

The relevant performance metrics are initially developed from the review of existing information and international best practice. Further insight was gained from the major stakeholders i.e. MHA and the Concessionaires via the workshop sessions carried out. Preliminary performance measures arrived at were evaluated via site sampling data collection to obtain a snapshot of the current Level of Service and to ascertain the practicality of the measures proposed during the implementation phase. Discussions were also held with MHA to streamline the study with MHA's objectives and aspirations. Further refinements were made based on the results obtained during Stage 2 (Pilot Program) of the project where the proposed performance measures were tested against the rigours and demands of tolled expressways in operations.

4.1.4 Recommended Performance Measures

A total of 47 performance measures are recommended comprising as follows:-

Table 4.1: Recommended Performance Measures

Description	Notation	Number
Management Performance Measures	MPM	6
Pavement Durability Measures	DPM	6
Pavement Operational Measures	OPM	10
Culvert Operational Measures	CPM	3
Slope Performance Measures	SPM	5
Bridge Performance Measures	BPM	5
Tunnel Performance Measures	TPM	9
Street Lighting Performance Measures	LPM	4
Total		47

Recommended Performance Measures and their relevant Levels of Service were derived after taking into account of the current and international practices, MHA Standards, manuals and guidelines, site sampling data collection, Pilot Program findings and discussions and suggestions by MHA. Further refinements undertaken were incorporated into the Supplemental Document.

4.2 Monitoring

Various methods will be engaged to identify non-conformances. The following monitoring methodologies were proposed:

Table 4.2: Compliance Monitoring Methodologies

Asset Group	Monitoring	Frequency of Monitoring
Pavement - Durability	Automated data collection	Annual
Pavement – Operational	Visual inspection	Monthly
Street Lighting	Structural – visual Functionality – visual Functionality - light meter	Annual (or as specified) Monthly Annual or as specified
Culverts	Functionality - visual inspection Structural integrity - record of inspection	Functionality – Monthly Structural integrity – annual (or as specified)
Bridges (Inspections)	Record of inspection	Annual (or as specified)
Tunnels	Air quality – automated Structural – record of inspections Lighting – light meter	Air quality – continual Structural – annual (or as specified) Annual (or as specified)
Slopes	Risk assessment Condition assessment	As required Annual (or as specified)

4.2.1 Weightings

Within each asset group weightings will be applied to each different non-conformance. The weightings will be assigned to reflect important factors such as road safety. As in the bucket non-conformance system described, it is proposed that the new MHA system also increase non-conformance weighting for non-conformances occurring in multiple consecutive months. In each consecutive month a repeat non-conformance attracts higher and higher weighting. A non-conformance identified by MHA but not previously identified by the Concessionaire will also attract a higher weighting.

4.2.2 Compliance Achievement Level

Currently MHA set compliance achievement for Periodic Maintenance defects at 88% and Routine Maintenance defects at 100%. This recognises that for periodic type maintenance the expressways are not going to be in ‘as constructed’ states 100% of the time. However, it is likely that stakeholders do expect a perfect network and equally likely those Concessionaires will not be in a position to physically provide such a perfect network given the dynamic nature of the asset. There are two ways to balance stakeholder expectations against what a Concessionaire can realistically achieve:

1. Set achievable target levels of service – setting the target level of service at perfection is costly, to the point where some concessions may not be viable. In addition to this many asset defects cannot be treated until they are visible. There are always going to be periods of time where the asset is not perfect.
2. Set compliance against target levels of service to < 100% - a high target level of service is set but a certain level of non-compliance is allowed to enable the Concessionaire to manage cost and effort effectively

It was recommended that recognition is maintained of the concept that the expressways cannot be maintained in a perfect condition; that dynamic factors such as asset age, traffic loading and climatic conditions will continue to influence the condition of each asset.

4.2.3 Proposed Compliance Monitoring & Methodology

The Supplemental Documentation describes the monitoring methodology to be employed, the relevant non-conformance weightings, measurement of compliance and reporting based on the performance measures and levels of service recommended for each asset class.

Since the Malaysian expressway networks have different lengths, normalization is introduced to some of the recommended performance measures so that the compliance performance can be compared. This is usually in the form of a %, based on length. Without the normalisation, if the total numbers of non-conformances are reported instead, longer networks will be at a disadvantage since they will have the higher likelihood of being more 'non-conforming'. Normalisation however, is not always required i.e. tunnel air quality – it is either compliant or not, irrespective of length (and if reported by tunnels it would be irrespective of the number of tunnels in the same network).

4.2.3.1 MHA Auditing

One of the key objectives of the project is to draw MHA away from an operational role (identifying defects and assessing compliance) to a governance role. Once the Concessionaire takes responsibility for assessment of compliance MHA can transfer effort to an auditing role. Essentially they will be checking that each Concessionaire is reporting compliance accurately.

4.2.3.2 Current MHA Practice

MHA currently undertake a comprehensive defects inspection each month and compile a performance achievement report for each Concession on a six monthly basis. In brief the following process is undertaken:-

- Month 1 – drive over inspection of 100% of network undertaken jointly by MHA and Concession. All routine and periodic defects recorded.
- Months 2, 3, 4, 5, 6 – drive over inspection of 100% of network undertaken jointly by MHA and Concessions. All new routine and periodic defects recorded. All defects from the previous month(s) that have been fixed recorded as 'rectified'. Any defects from the previous month(s) that have not been fixed are recorded as 'not rectified'.

Of particular note in relation to this performance assessment is that it is undertaken by MHA and not the Concessionaire. It is also restricted to the assets and maintenance activities defined as either routine or periodic maintenance. In order to move MHA toward a governance role three significant changes to the current monitoring framework need to be considered:-

- The Concessionaire must be responsible for the assessment of conformance against the performance measures;
- MHA moves from detailed monitoring of defects to auditing on non-conformances; and
- The performance assessment should cover all performance measures.

4.2.4 Non Compliance Penalty (Rewards & Consequences)

Once implemented the non-conformance system will provide MHA with a clear understanding of compliance across each concession. However, in itself it will not necessarily drive desired behaviours from the Concessionaire. An incentive to comply or a penalty for noncompliance should be considered. As each concession is signed, incentives need to be identified within the bounds of the concession agreements.

4.2.4.1 Concession Extensions

Extension of a concession period could be sufficient incentive for a concession to maintain compliance with levels of services. Concessions invest significant amounts in terms of resources and equipment. Being able to extract further value from equipment and keep resources employed for maximum time frames may be attractive to Concessionaires.

Consideration of extending the years of a concession in proportion to the consistency of compliance against levels of services can be considered. With this, the greater the consistency of compliance, the longer the extension can be considered.

4.2.4.2 Name and shame

A powerful non-financial incentive for a Concessionaire to comply is comparison of performance against competitors. It is not in a contractor's interest to be known as a supplier who fails to meet a client's objectives, or who is not performing as well as competitors. The consequences of being 'shamed' are significant in the long run as the reputation of the contractor is affected. Contractors may then struggle to

secure further work with the client, or if the performance data is available to the wider industry they may jeopardise opportunities with other clients.

'Name and Shame' can be used in conjunction with an incentive scheme, such as the contract extensions described previously, or a penalty scheme, such as the calling of the Maintenance Bond for non-compliance as described in the next section.

4.2.4.3 Maintenance Bond

Consideration could be given to linking the calling of the maintenance bond (common to all the concession agreements) when non-conformance reaches a predefined level. The maintenance bond is currently called when a concession fails to rectify defects identified by MHA. It is rarely invoked and somewhat discretionary.

If an objective framework were developed to provide both Concessionaire and MHA with a mechanism for calling the bond when the level of non-conformance reached a maximum allowable level (perhaps with a time factor) it may well act as a sufficient consequence to minimise cases of non-compliance against levels of service.

4.2.5 Pavement Management and Performance Monitoring Systems

Information Communication Technology (ICT) plays a vital an important role in the management of highway assets. MHA has long realised the importance of road asset management systems with the implementation of Total Expressway Asset Management System (TEAMS) for pavement management and Highway Audit Quality System (HAQS) for Concession performance monitoring system. A system study and review for both systems were undertaken and areas for improvement for the integration of the KPI Compliance were identified.

5 Summary

The development of the KPIs for the Malaysian Expressway Concessions was undertaken in two stages as follows:-

- Stage 1 - Development of the KPIs and KPI Monitoring Framework; and
- Stage 2 - Pilot Program

The various components essential for the success of the full implementation of the KPIs to all Concessions under MHA purview are illustrated as follows:-

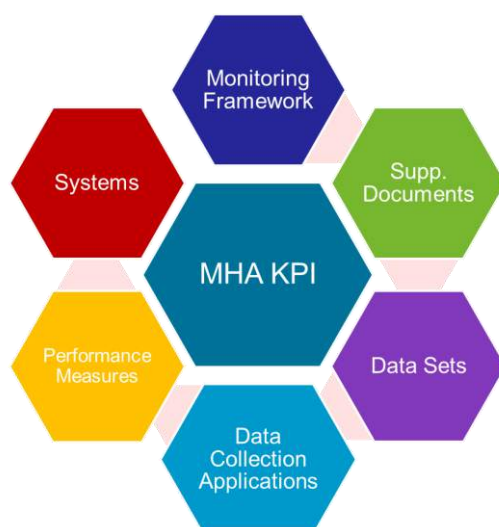


Figure 5.1: Essential Components for Implementation of KPIs to All Concessions

5.1 Essential Components for KPI Implementation

5.1.1 Performance Measures

Performance measures and their respective Levels of Service (LOS) need to be established based on the current practice and compared to internationally accepted practices. They need to be 'tested' for appropriateness and practicality in terms of data collection and reporting which will facilitate the buy – in from both MHA and Concessions.

5.1.2 Monitoring Framework

The Monitoring Framework need to be properly documented outlining all of the processes involved that are to be employed. This includes the auditing process, the parties involved and the reporting aspects.

5.1.3 Supplemental Document

This technical document provides the necessary details of the performance measures involved and their respective Levels of Service required. The means of how the data is collected, the frequency of measurement, the equipment to be used (if required) and evaluation of compliance are to be defined in the document.

5.1.4 Data Collection Applications

Depending on the frequency of data to be collected, vast amount of data are to be collected and reported every month. The usage of data collection applications especially in the field would be very useful when the collected data is to be used for further processing and reporting. Usage of data collection applications (using Smart phones and computer tablets) will simplify the process of data recording compared to the manual paper forms. The data can be automatically updated to a database.

5.1.5 Database Systems

It is crucial that database systems are to be in place when huge amount of data would be expected considering the number of Expressway Concessions operating and soon to be operating in Malaysia. The type of data includes the inventory and type of assets, raw monthly audit data, asset condition reports, compliance reporting etc. The use of computerized database systems will facilitate the management and monitoring of data to ensure ease of reporting due to standardization and in real time, if web based systems were to be used.

Since MHA is currently using HAQS as a reporting database, it is of utmost importance that the existing HAQS to be enhanced to include KPI Monitoring where all maintenance related data and reporting can be updated into the system and the extraction of documented data could be possible from the existing HAQS database.

5.1.6 Data Sets

With larger data sets and longer data cycle, further data mining processes can be employed to derive trending and predict the Concessionaire's maintenance and asset performance. Further refinements of related processes can then be undertaken accordingly to ensure that the expected levels of service would be met.

5.2 Recommendations

Based on the findings of the study, it was recommended as follows:-

- the KPI Monitoring Framework and reporting is to be implemented to all Concessionaires under MHA purview;
- more training sessions would be warranted for Concessionaires who were not involved in the Pilot Program for data and compliance reporting;
- sufficient time need to be allowed for Concessionaires to become fully conversant with the Compliance Reporting procedures and requirements;
- to undertake data collection, analysis and trending for all Concessions for between 12 – 24 months to ensure consistency;
- the Supplemental Document is a 'live' document and it can be refined as necessary;
- to consider further improvement in data collection using data collection applications (Smartphones or computer tablets); and

- to consider enhancing the current MHA HAQS for integration of the KPI Monitoring Framework.

The asset management approach of implementing KPI for the Malaysian Expressway Concessions would ensure the standardization of maintenance criteria of expressway assets where reporting of performance would be streamlined. To ascertain success, collaborative effort between MHA and Concessions would be warranted.

The management of the Concessions via performance reporting would set the asset management of Expressways in Malaysia by MHA on a new threshold with benefits as follows:-

5.2.1 MHA

- Reinforces and enhances the governance role for the maintenance and upkeep of the expressways and Concessions.

5.2.2 Concessions

- Encourages innovation in maintenance approach
- Provides flexibility of how assets and funds are to be maintained and managed based on a set KPIs
- Promotes a sense of responsibility of Concessions for the maintenance of their respective assets.

6 Conclusions

Performance based specifications were introduced to manage road assets approximately 20 years ago. Government and road controlling authorities internationally recognized the benefits in terms of the management of their road assets and started moving from traditional forms of contracting to performance contracting.

The relevant performance metrics were initially developed from the review of existing information and international best practice. Further insight was gained from the major stakeholders i.e. MHA and the Concessionaires via the workshop sessions carried out. Preliminary performance measures arrived at were evaluated via site sampling data collection to obtain a snapshot of the current Level of Service and to ascertain the practicality of the measures proposed during the implementation phase. Discussions were also held with MHA to streamline the study with MHA's objectives and aspirations. Further refinements were made based on the results obtained during Stage 2 (Pilot Program) of the project where the proposed performance measures were tested against the rigours and demands of tolled expressways in operations. A total of 47 performance measures and their related levels of service were recommended, and these performance measures are deemed critical to the governance role of MHA and become the Key Performance Indicators (KPI) for the monitoring of the Malaysian Expressway Concessions.

In conclusion, the study to date had achieved its objective in terms of developing a performance based monitoring framework which will emphasize MHA's governance role while at the same time will encourage expressway Concessionaires to adopt a preventative approach to the maintenance of their assets. Efficiencies in terms of maintenance and upkeep of the expressway assets can be gained if all Concessionaires were to be monitored under appropriately similar KPIs and employ compatible asset management systems and procedures in line with their respective Concession Agreements. The overriding objective is to have a safe, well maintained and appropriately preserved expressway networks which road users use by choice.

Currently, MHA is undertaking all of the recommendations made for the implementation of the KPIs for the Concessionaires since 2015. It is envisaged that with the advent of 2 years data to be obtained in 2015 – 2016, further refinements would be made in terms of the assets involved, the means of data to be collected and reported and also the KPIs themselves for further improvement.

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ABSTRACT:

The theme of Road Safety is often related to other engineering issues, both of transportation and structural nature. Ensuring or increasing a suitable level of safety would bring the Road Network Manager to deal with several levels of problems.

In Brazil, in the States of Minas Gerais and Goiás, the Road Network Manager MGO is planning to strongly increase the safety of the BR-050 highway, by enlarging the road platform and bringing the number of lanes to 4 in every kilometer of its total length of 437 km. This project would double, in half of the length of the highway, the number of lanes.

The paper will present the main transportation and structural issues faced by MGO, with a huge effort of conservation, refurbishment, reinforcement and enlargement of the existing structures and infrastructures, in order to deliver a cost-effective intervention of the whole road network branch.

Experiences of improving road safety in Brazil

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1 INTRODUCTION

The theme of Road Safety is not only related to vehicles and road regulation, but it is often also related to specific engineering issues, both of transportation and structural nature.

In fact, Road Safety depends on the condition of the infrastructure, on its geometrical characteristics, on its conservation state and level of safety. This not only helps to increase comfort and safety of road users, but decreases also the risk of damage and loss of operability in case of disaster events like earthquake, floods and others (Franchetti et al., 2008).

Therefore, increasing and ensuring a suitable level of safety is a challenge that every Road Network Manager has to face, dealing with several levels of problems of technical, functional and economical nature.

In this paper it is presented a real case of application of Road Safety Improvement Planning in Brazil. In particular, in the States of Minas Gerais and Goiás, the Road Network Manager MGO Rodovias is planning to strongly increase the safety of the BR-050 highway.

This will be achieved by enlarging the road platform along all the highway and bringing the number of lanes to 4 in every kilometer of its total length of 437 km. This project would double, in half of the length of the highway, the number of lanes.

In this paper the main transportation and structural issues faced by MGO Rodovias are presented and discussed.

2 FRAMEWORK

Starting from 2014, MGO Rodovias is the Road Network Manager of BR-050 (GO / MG), a 436.6 km Brazilian highway that runs from the intersection with BR-040, in Cristalina (GO), until the border between Minas Gerais and São Paulo states, in the municipality of Delta (MG).

Currently, the segment under concession has:

- 191.85 km of two-lane road in Goiás;
- 26.65 km of four-lane road already duplicated by the concessionaire, still in Goiás;
- 218.1 km of four-lane road with central separation in Minas Gerais;
- 22.8 km of side roads,
- 1.3 km unpaved.

In its extension it covers 9 municipalities, 5 in Goiás (Cristalina, Ipameri, Campo Alegre de Goiás, Catalão and Cumari) and 4 in Minas Gerais (Araguari, Uberlandia, Uberaba and Delta).

The highway is one of the main lines connecting the Federal District (with the Capital City of Brazil, Brasilia) and São Paulo, cutting important municipalities of agro-industrial activity and wholesale trade.



Figure 1. Map of BR-050 Highway.



Figure 2. BR-050 Highway.

2 PLANNED WORKS OF ROAD SAFETY IMPROVEMENT

The works of structural and operational improvements to be made by MGO Rodovias during the contract will have a direct impact on the quality of road transportation in Goiás and Minas Gerais, with the improvement of the BR-050 on security aspects, fluidity, economy and comfort. They are considered essential for the modernization of this regional infrastructure.

In 30 years of concession, MGO Rodovias will invest \$ 3 billion in BR-050, and 1.5 billion in the first five years of the concession. Of this total, R \$ 2.0 billion will be for structural interventions in the pavement and functional and operational improvements, and R \$ 650 million specifically for enlargements / duplication of the highway.

Franchetti is an engineering company that operates in the fields of civil, transportation and environmental engineering. The structural engineering work undertaken by Franchetti mainly involves existent strategic and important works that require a rapid assessment of safety and degradation.

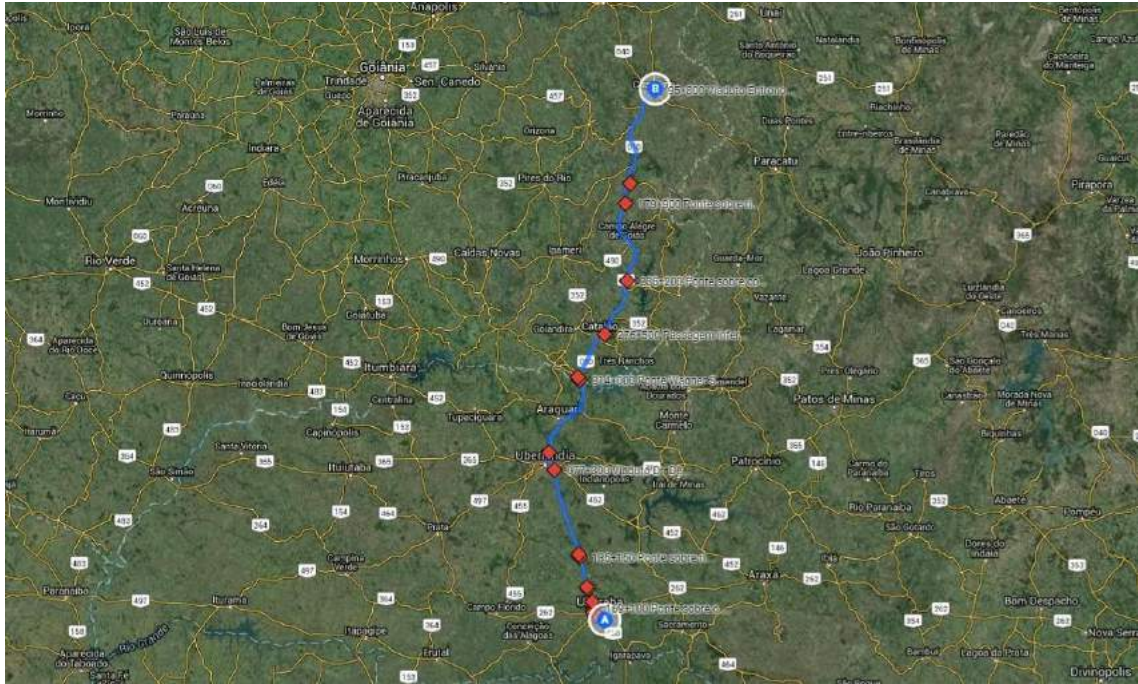


Figure 3. Bridges to be refurbished and enlarged along BR-050



Figure 4. Some examples of the bridges to be upgraded

Franchetti is in charge for the verification and design of the refurbishment and enlargement of 27 bridges along the BR-050. The activities included in-depth initial inspections of the bridges, assessment of their condition state,

analysis of the actual safety condition of the structure, design of refurbishment, reinforcement and enlargement of the bridges.

The bridges were all in reinforced concrete, with only two of them in pre-stressed concrete. The lengths of the bridges span from 20 m to 170 m. 14 of them were bridges on water stream, usually of small width, with the exception of the bridge Wagner Estelita Campos, between the states of Minas Gerais and Goiás, across the river Paranaíba with a width of over 150 m. The other bridges are crossing over other roads and railroads.

2 PRELIMINARY ACTIVITIES: INSPECTIONS AND TESTS ON MATERIALS

As a preliminary activity, a campaign of in-depth inspections was carried out, focused on the assessment of conservation state of the bridges.

The activities provided:

- Visual inspection with damage assessment, degradation assessment and condition state definition;
- Campaign of destructive and non-destructive tests on materials, included concrete core boring, carbonation and chemical tests, steel corrosion and tensile stress test, Schmidt hammer tests, pull-out tests.

A total amount of 570 tests was performed.

The tests helped the designer to identify the structural characteristics and conservation state of the bridges in order to define their residual capacity and therefore design the interventions.



Figure 5. Some examples of test on materials

3 STRUCTURAL AND ROAD REQUIREMENTS

The main structural and road geometry requirements related to the planned works were the following:

1. Structural refurbishment and reinforcement to increase the load classification of the bridges (from TB-36 to TB-45);
2. Structural and road enlargement to increase the road capacity.

3.1 STRUCTURAL REFURBISHMENT AND REINFORCEMENT

The bridges along the BR-050 highway, like many bridges in Brazil, were designed with the code NB6/60, that prescribed a truck load on bridges of 360 kN of weight (TB-36).

Starting from the code NBR-7188/84, and then with other revision in the following years until the last one in 2013, the vehicle load on regular bridges passed from 360 kN to 450 kN (TB-45). This brought the bridge to have an increment on nominal load of 25%, plus some correction coefficients that take into account dynamic effects, number of lanes and defects on the road surface.

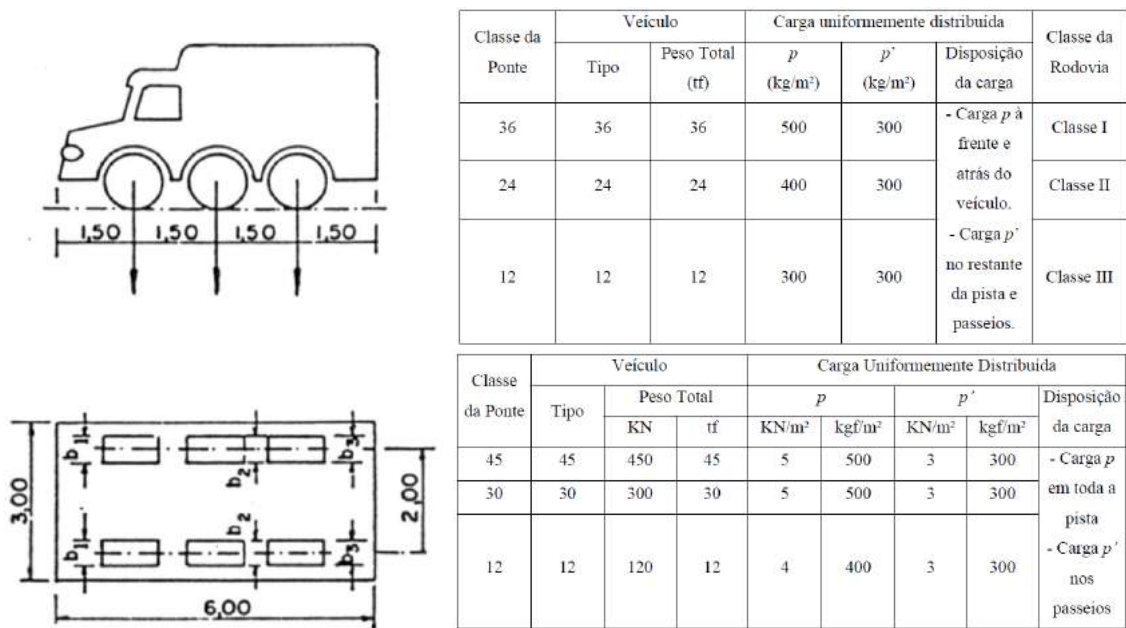


Figure 6. Comparison between TB-36 and TB-45 load characteristics

Given the need for Road Manager to enlarge the bridge, it was also the necessary to upgrade them to the new code.

3.2 STRUCTURAL AND ROAD ENLARGEMENT

The principal goal of MGO Rodovias, following the indication and the planning of ANTT (National Agency of Land Transportation), was to enlarge the road platform in every kilometre of the concession, in order to achieve the following:

- At least four lanes (two lane in north direction and two lanes in south direction);
- Total width of 13,66 m (13,16 m in Minas Gerais);
- Lane width of 3,60 m;
- Emergency lane of 3,00 m (2,50 m in Minas Gerais);
- Presence of 1,50 m wide crosswalk;
- Internal shoulder of 1,00 m.

The initial width of the existing bridges was from 10 to 14 m, needing in some cases an enlargement of more than 3,5 m.

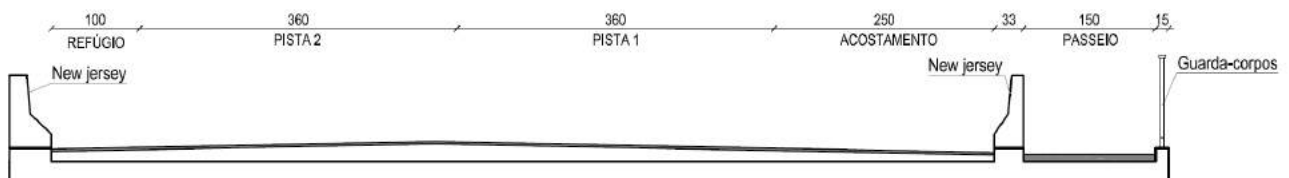


Figure 7. Road enlargement characteristics

4 DESIGN OF INTERVENTIONS

The interventions were guided by the following principles:

- They had to be cost-effective;
- They had to preserve the main structure of the bridge, without any deep modification on the structural behaviour;
- They had to be as simple as possible to apply, without complex procedures that involve skilled figures not always present in the territory;
- They had to leave the road always open during the works, at least in alternating directions. This is important mostly in the segments of the road with only two lanes, since BR-050 is the only connection between north and south regions in this area, without any alternative road.

4.1 REINFORCEMENT INTERVENTIONS

The reinforcement interventions were designed, starting from the previous guidelines, in 4 typologies:

- To increase the negative moment resistance of the deck, the thickness of the slab was increased, with additional reinforcement;
- To increase the positive moment resistance of the deck, concrete beam section were widened, with additional reinforcement and grouting;
- When additional reinforcement was needed, or when geometrical condition didn't allow the application of concrete widening (slab deck), FRP reinforcement strips were used;
- When strong additional reinforcement was needed, post-tension was added to the beam.

In the case of bridge Wagner Estelita, a specific solution had to be designed to substitute the existing concrete piers on the water with new piers, always without interrupting the traffic on the road. This was necessary because the existing piers had lost the bearing of the ground at foundation level, due to scour of the water under the piers.

Together with the principal interventions described above, other procedures were applied in order to have the bridges fully upgraded.

For example, on most of the bridges, the old bearing, constituted by steel or lead plates, were substituted with new modern bearings, like confined elastomeric bearings with controlled degree of restraint.

In order not to change the original static scheme of the bridge and create unexpected solicitation states, the new bearing scheme had to be as similar as possible to the old one, assuming the columns with only metallic plate on the top as fixed bearings and the short columns (usually at the extreme sides of the bridges) as mono-directional bearings, since a "freyssinet" type hinge was originally built at the bottom of them.

The new bearing system provides a couple of fixed columns (usually the ones at the centre of the bridge) and two or more mono-directional bearings at the sides.

The substitution of the bearings is provided through the use of metallic cantilevers fixed at the top of the columns, and with hydraulic jacks to lift the bridge. For every column a variable number between 2 and 4 jacks is provided. All the jacks are connected to a central control device, in order to lift the bridge homogeneously.

After the bearing substitution, the metallic cantilever are disassembled and used on another bridge.

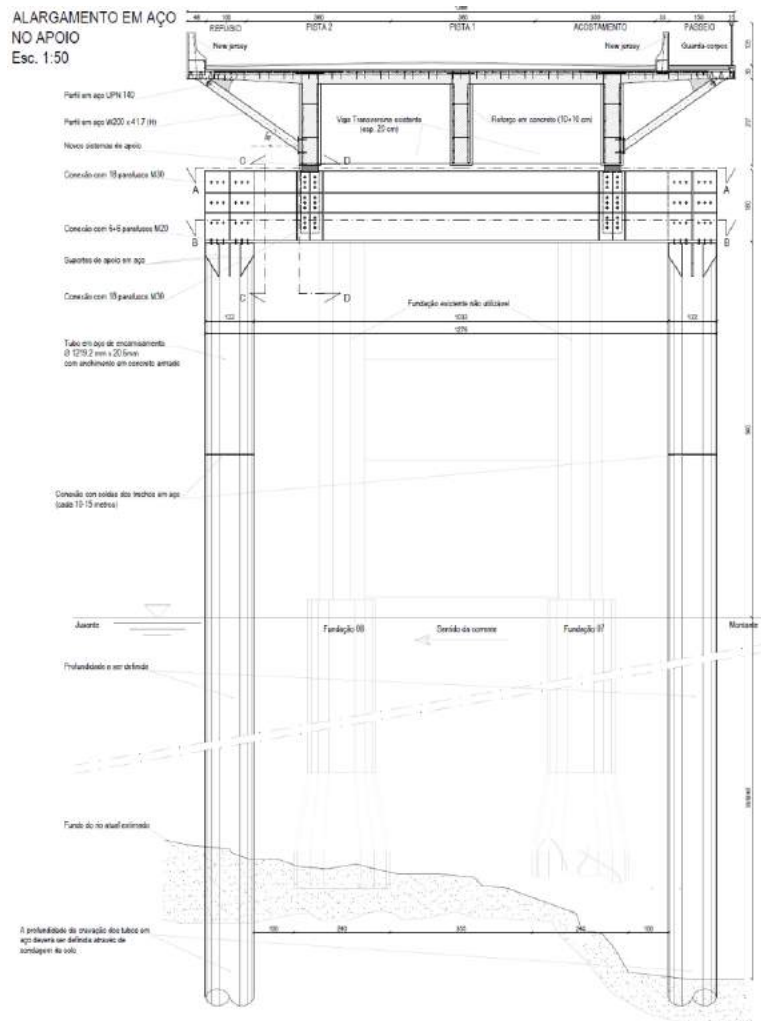


Figure 8. Wagner Estelita Campos bridge: substitution of existing piers with new piers



Figure 9. Bearing substitution

4.2 ENLARGEMENT INTERVENTIONS

The enlargement interventions were designed, starting from the previous guidelines, in the first attempt by avoiding additional foundation and piers. Where possible, this allows quicker interventions and reduced costs:

- Where the four road lanes were separated in the middle, usually there was enough space to allow symmetrical enlargement of the bridges, suitable for structural needs. In these situations, a system of cantilever transversal beams, made on steel or concrete, were designed, connected to the longitudinal beams; the existing columns and foundations were verified with the new loads;
- Where the four road lanes were not separated in the middle, another type of intervention was designed, usually with an additional beam and column by one side, or additional deck slab.

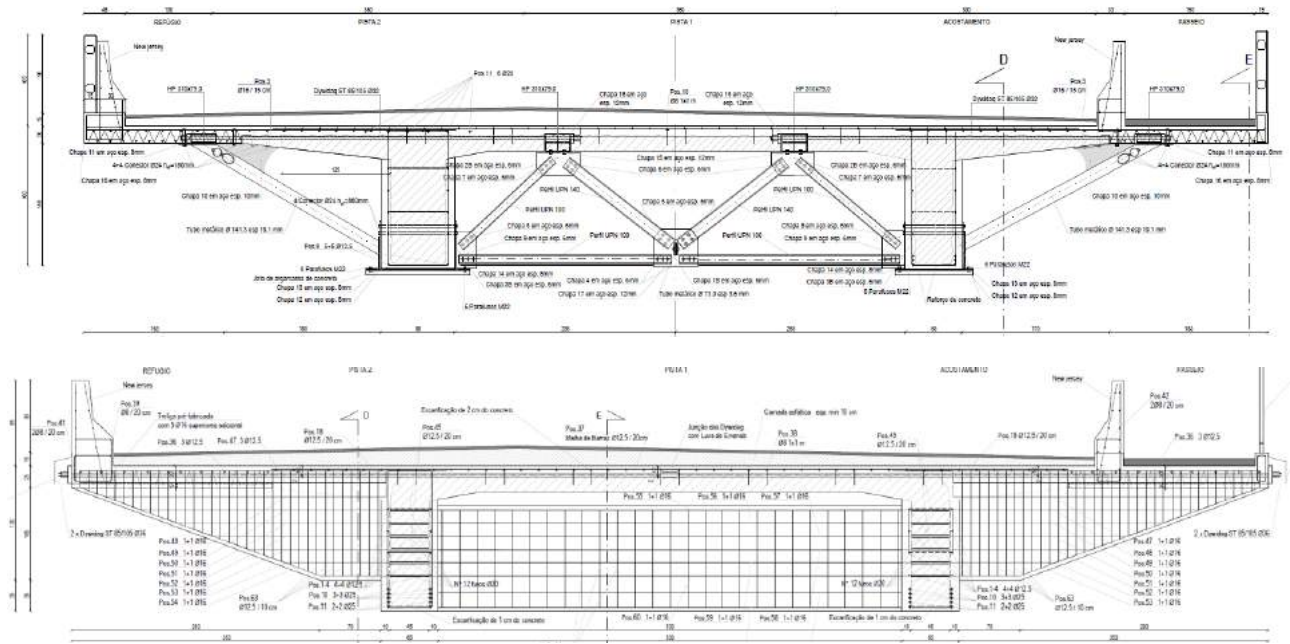


Figure 10. Symmetrical bridge enlargement: solutions with steel frame and concrete cantilevers

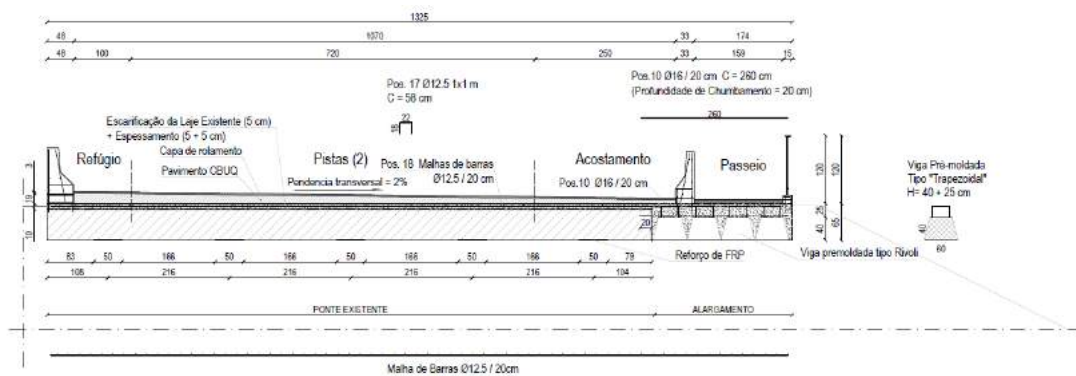


Figure 11. Asymmetrical bridge enlargement: solutions with slab deck

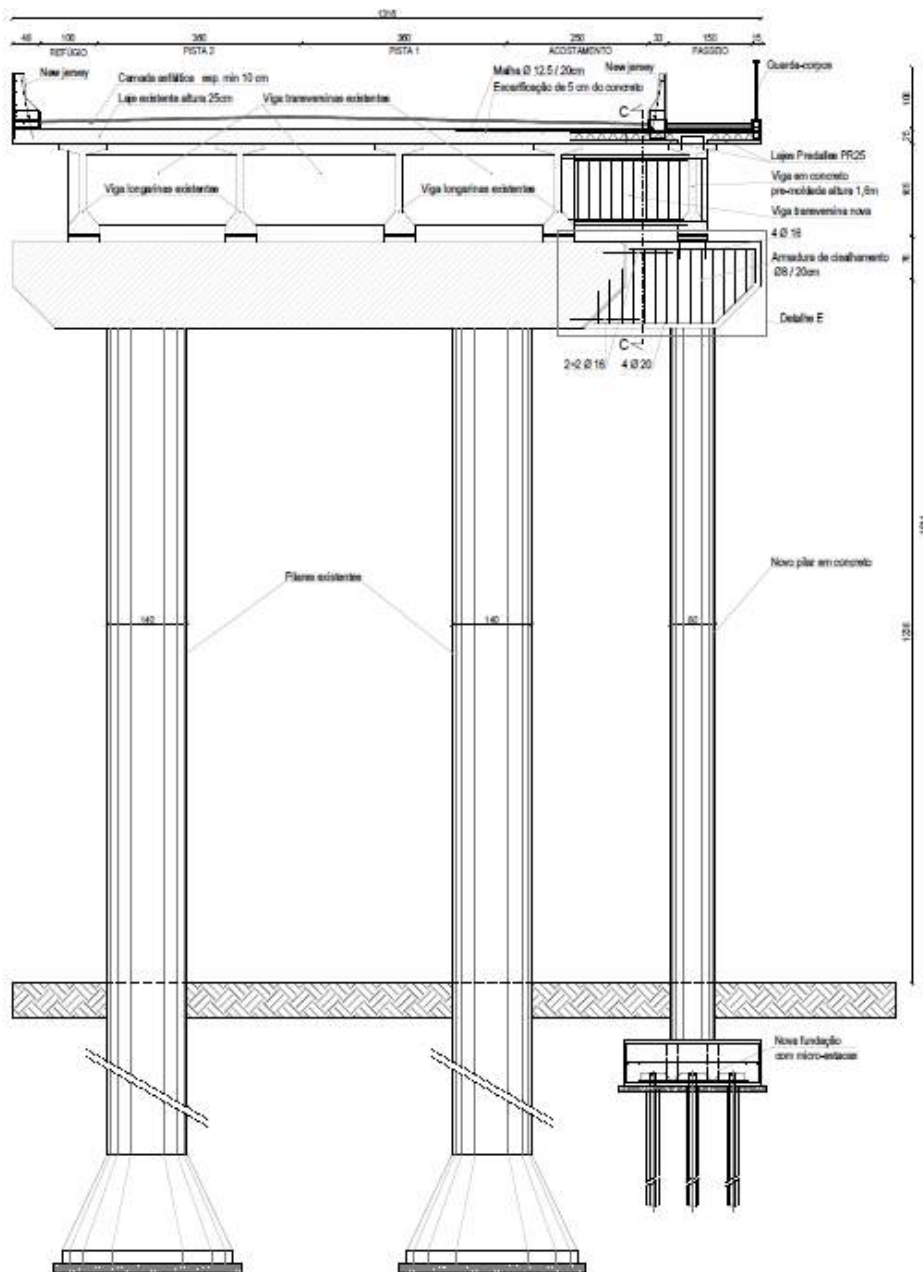


Figure 12. Asymmetrical bridge enlargement: solutions with beam deck

4.3 ROADWAY SAFETY

Together with structural interventions, also roadway safety issues were taken into account.

The width of the lanes was not increased with respect to the previous situation, but additional safety measures were introduced, like:

- Emergency lane, not present before the intervention;
- Internal shoulder, not present before the intervention;
- Physical separation between road platform and crosswalk through concrete barrier (new jersey).

The choice of concrete new jersey is related mostly to the construction practice of the area, and to the lack of steel barriers suppliers.

The bridges object of the analysis are not crossed often by pedestrians during the day, since most of the structures are far from inhabited areas. However there are some people that stop on the bridges for long periods of time (mostly fishers).

The previous crosswalk configuration was not sufficient to ensure the minimum level of safety for pedestrians, since the crosswalks were constituted by narrow bands (from 50 to 80 cm wide) not physically separated from the road platform but only elevated of 10-15 cm with respect to the road.

The countermeasures that were provided, even if not fully completed, started to give the first outcome, as seen from the following graph. The monthly number of events in the BR-050 roadway is showing a decrease in the recent periods.

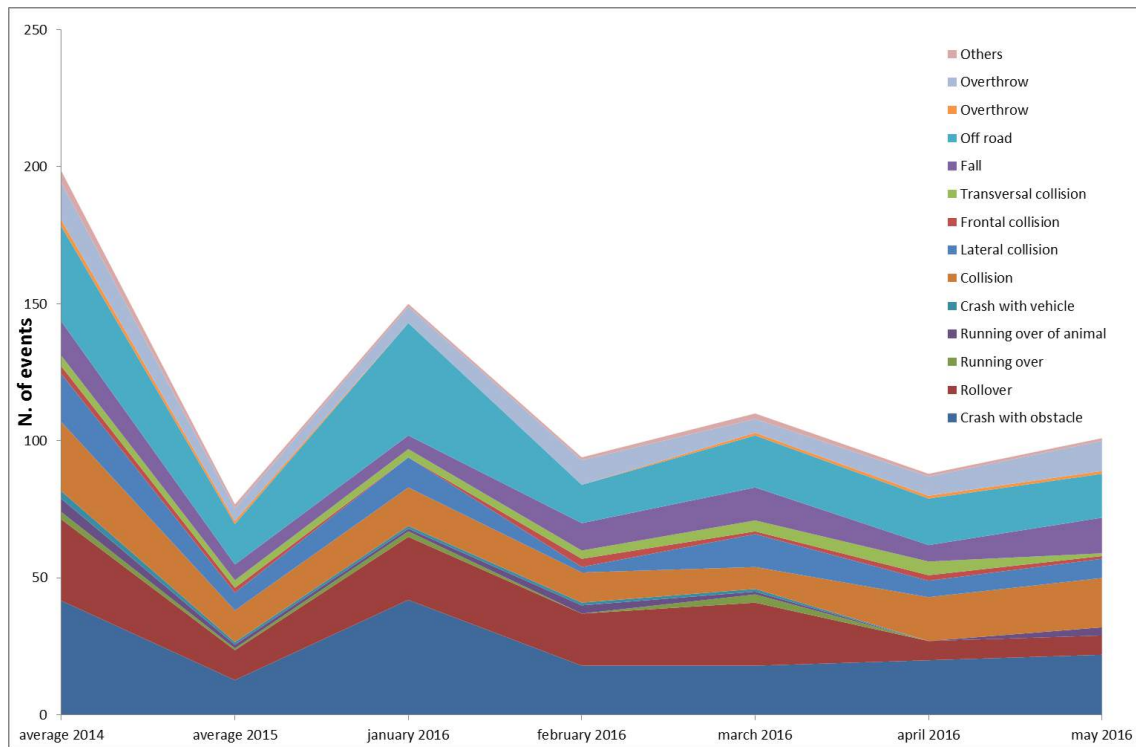


Figure 13. Number of monthly events in the past months

The safety barriers of the bridge were built in concrete, due mostly to the construction practice of the area, and to the lack of steel barriers suppliers. Also the barriers along the roads, when present, were mostly in concrete. In this way the transition between bridge and road is not affected by the change in barrier typology.

4.3 LIFE CYCLE ANALYSIS

Given an estimation of the cost of intervention for the reinforcement and enlargement of the bridges, a really rough and preliminary analysis was performed in order to have a rough order of magnitude (ROM) of the Return of Investment (ROI) of the rehabilitation intervention.

The estimation was made by comparing the cost of intervention for some bridges to the cost of construction of a bridge with the same length at the side of the first one. This happens when the existing road has a width of only 2 lanes, and the design road has a width of 4 lanes. In this situation, the existing bridge is reinforced and enlarged to fit the emergency lane and the crosswalk, while a new bridge is built next to the old one for the others 2 lanes.

For example, for Wagner Estelita Bridge, we have:

- Cost of rehabilitation of existing bridge: 4'500'000 US\$
- Cost of construction of the new bridge: 7'500'000 US\$
- Cost of demolition of the existing bridge: 500'000 US\$

With these values we have a ROI of 0,78, index of good investment.

5 CONCLUSIONS

The paper presented the main issues faced by MGO with the goal of refurbishment and enlargement of BR-050 bridges, in the general planning of road safety improvement.

A huge effort of conservation, refurbishment, reinforcement and enlargement of the existing structures and infrastructures was made, in order to deliver a cost-effective intervention of the whole road network branch. The final result was a series of structural solutions that allowed minimum cost of intervention and time-effective actions.

6 ACKNOWLEDGEMENTS

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MGO Rodovias – Internet site (www.mgorodovias.com.br).

PAPER TITLE	Pilot study of permeability of pervious concrete in conjunction with ultrasonic pulse velocity and X-ray computed tomography		
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KEYWORDS:

Permeability, Pervious Concrete, Ultrasonic Pulse Velocity, X-ray Computed Tomography

ABSTRACT:

The research objective of the pilot study was to study the permeability of pervious concrete in conjunction with ultrasonic pulse velocity and X-ray computed tomography. The research scopes cover to batch three different pervious concrete, to cast concrete cylinders, to assess the mechanical properties of 28-day moist-cured compressive, to evaluate dynamic modulus of elasticity conducted by ultrasonic pulse velocity, and permeability virtually determined by X-ray computed tomography. The potential research outcomes explore preliminary results in mechanical properties of hardened pervious concrete and predictive correlations between dynamic modulus of elasticity and permeability nondestructively evaluated. The pilot study is still an ongoing project and more outcomes will be unveiled in in the near future.

Pilot study of permeability of pervious concrete in conjunction with ultrasonic pulse velocity and X-ray computed tomography

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1 INTRODUCTION

Pervious concrete pavement (Tennis et al., 2004) has been found a well-designed tool in recharging groundwater, reducing stormwater runoff, and meeting U.S. Environmental Protection Agency (EPA) stormwater regulations. Typically, the porosity or the interconnected porous structures are designed to meet between 15% to 25% in hardened pervious concrete and the flow rates for water running through are 0.34 cm/s or 200 L/ m²/min (480 in/hr or 5 gal/ft²/min). The benefit of use of pervious concrete may help on getting credit point in the U.S. Green Building Council's Leadership in Energy & Environmental Design (LEED) Green Building Rating System (Sustainable Sites Credit 6.1) which eventually possible winning the LEED project certification. In general, the density of pervious concrete is about 1600 to 2000 kg/m³ (100-125 lb/ft³); typical compressive strength can develop from 3.5 to 28 MPa (500 to 4000 psi).

The research objective was to develop a pilot study and assess pervious concrete cylinders with three different mix proportioning in conjunction with ultrasonic pulse velocity and X-ray computed tomography. The research scope was to utilize the local siliceous aggregates obtained in southern Taiwan to mix and proportion pervious concrete mixtures. Fresh and hardened concrete properties, permeability, and voids were of the major research interests in this pilot study.

2 EXPERIMENTAL PROGRAM

Details of materials and mixing proportioning used, fresh and hardened concrete tests performed, and ultrasonic pulse velocity and X-ray computed tomography tests executed are as follows:

2.1 Materials and mixing proportioning

ASTM Type I cement (ASTM C150-16e1) was purchased by a local contractor and mixing water was the tap water provided on campus. Coarse aggregates identified as the siliceous aggregate were supplied by a local contractor and it was a blended type with the 12.5mm of Nominal Maximum Aggregate Size (NMAS), 2.64 of bulk specific gravity (ASTM C127-15), and 1.35% of absorption (ASTM C127-15), respectively.

Table 1 shows the mix proportioning of three different pervious concrete mixtures, namely P1, P2, and P3 mixtures. P1 mixture contains one fractionated coarse aggregate that was retaining on the 4.75mm (#4) sieve and passing through 9.5mm (3/8 inch) sieve; P2 mixture also comprises another fractionated coarse aggregate that was retaining on the 9.5mm sieve and passing through 12.5mm (1/2 inch) sieve; P3 includes the aforementioned blended type of 12.5mm coarse aggregates without fractionation process. It has to be noted that three pervious concrete mixtures were carefully designed and identically proportioned by weight. The water to cementitious ratio (w/c) was selected to 0.30. Concrete cylinders were cast and later brought to cure in the limewater tank for 28 days curing process, shown in Figure 1. It has to be noted that only a very few amount of superplasticizer was added to improve the workability of pervious concrete mixtures.

Table 1. Mixing proportioning of pervious concrete mixtures (by weight)

Mixtures	W/C	Coarse aggregates (kg/m ³)	Cement (kg/m ³)	Water (kg/m ³)	Superplasticizer (kg/m ³)
P1	0.30	1530	340	100	2
P2	0.30	1530	340	100	2
P3	0.30	1530	340	100	2

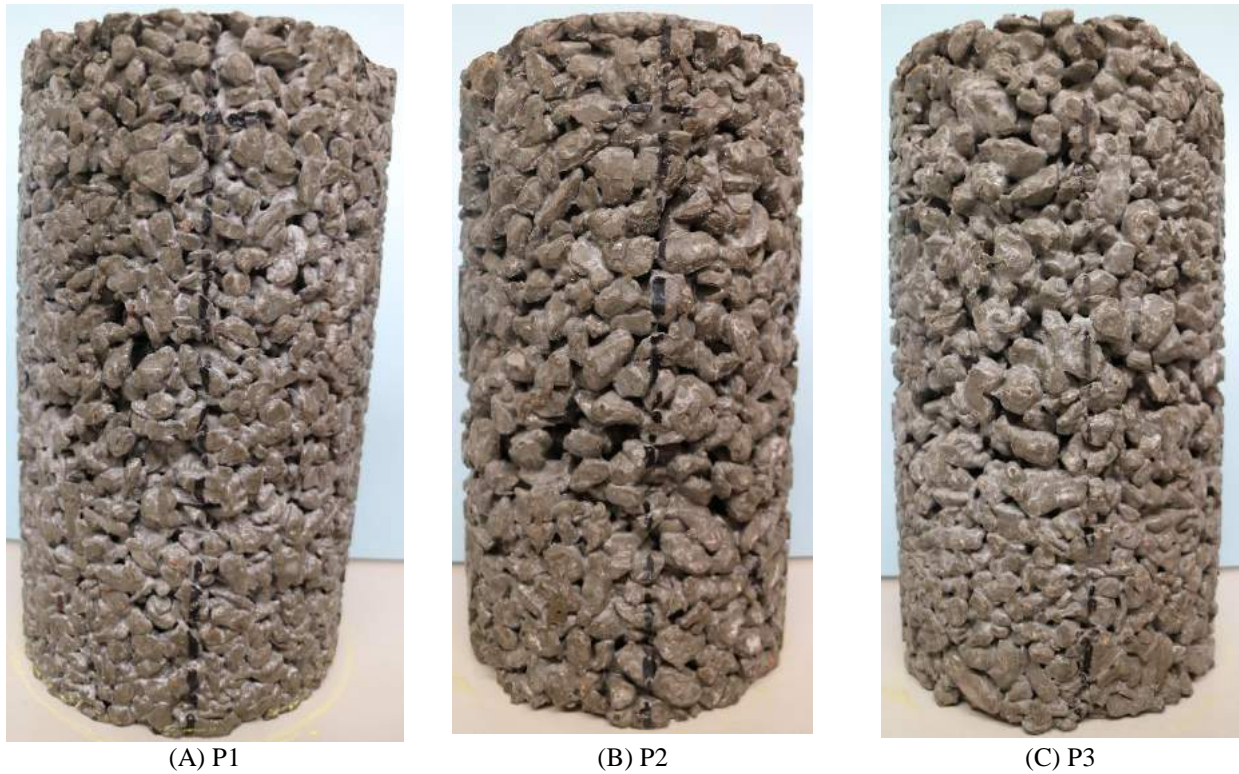


Figure 1 (A)(B)(C) Three pervious concrete cylinders (Photos taken by Ping-Ni Hou).

2.2 Fresh and hardened concrete tests

Unit weight test (ASTM C138-16) of fresh concrete and compressive strength (ASTM C39-16b) test for hardened pervious concrete cylinders were performed. Figure 2 (A) and (B) show the scheme of the unit weight and compressive strength test in the materials testing laboratory located the Department of Civil Engineering of National Kaohsiung University of Applied Sciences.

The percentage of interconnected air voids inside of three pervious concrete cylinders were also assessed. The hardened concrete cylinder was measured the volume (V) and weighted in water tank (W_1); the cylinder later brought out of water tank, placed to drain, and weight in ambient and surface dry condition (W_2). Equation (1) indicates the percentage of interconnected voids or porosity ($P_{a_interconnected}$) as follows:

$$P_{a_interconnected} = \left[1 - \frac{(W_1 - W_2) / \rho_w}{V} \right] \quad (1)$$

Where the density of water generally assumed $1.0 \text{ (g/cm}^3\text{)}$.

2.3 Ultrasonic Pulse Velocity test

Ultrasonic Pulse Velocity (ASTM C597-16) test was utilized to evaluate the velocity of ultrasonic pulse running through three hardened pervious concrete cylinders nondestructively, shown in Figure 3. The pulse velocity (V) was a longitudinal stress ultrasonic waves propagating through a pervious concrete cylinder to relate with its dynamic modulus of elasticity (E_d), dynamic Poisson's ratio (μ), and density (ρ). Equation (2) shows the aforementioned parameters and relationship as follows:

$$V = \sqrt{\frac{E_d(1-\mu)}{\rho(1+\mu)(1-2\mu)}} \quad (2)$$



Figure 2. Scheme of unit weight and compressive strength test (Photos taken by Ping-Ni Hou).

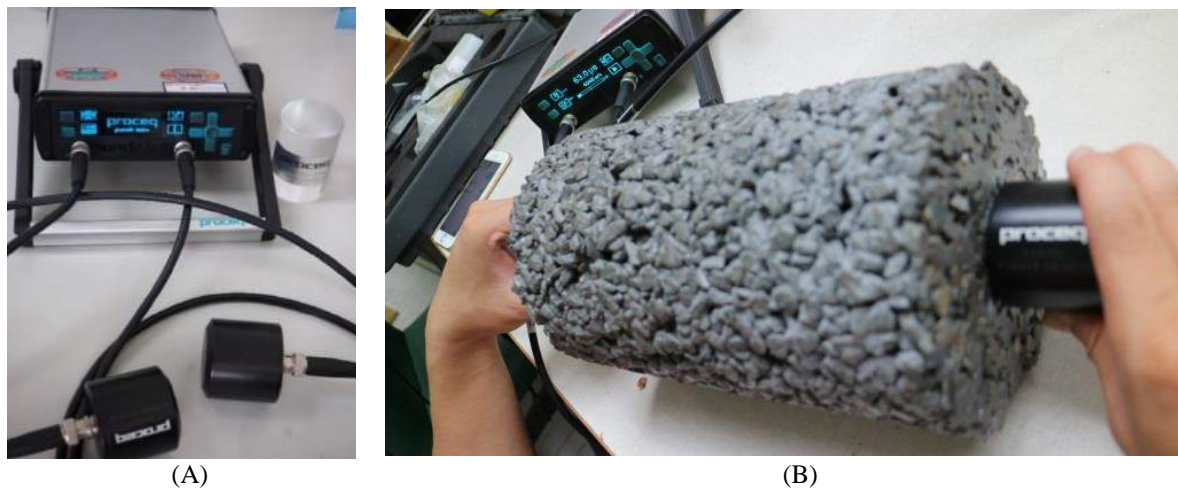


Figure 3. Scheme of Ultrasonic Pulse Velocity test (Photos taken by Ping-Ni Hou).

2.4 X-ray computed tomography

The nondestructive evaluation through the medical X-ray CT was conducted at the E-Da/I-Shou University Hospital. The Somatom® Emotion manufactured by the Siemens Healthcare is a 16-channel X-ray CT facility and provides maximum 0.75 mm in resolution in Z-axis. Figures 4 presents the scheme of performing the X-ray CT scan and the orientation of the segments. While performing the X-ray CT scan, the concrete cylinder was placed on the patient's table. The operator positioned the table properly and started the scan. The concrete cylinder was going back and forth through the ring of the facility along with the Z-axis and the embedded sixteen X-ray channels/sources acquired multiple X-ray digital images to construct the three-dimensional reconstruction momentarily. The setting of acquiring the concrete cylinder CT images was to use 110kV with automatic current (mA or μ A). Previously research work (Su et al 2014;2016) conducted have identified and adopted setting with the same voltage and automatic current during to scan PCC or asphalt concrete without any problem. After the segments acquired from the facility, series of segments in the medical formatting—DICOM were brought back to KUAS to proceed the image processing and analysis. It has to be noted that the image processing and analysis were conducted by Mathworks MATLAB R2015b and the ImageJ provided by National Institutes of Health. The goal was to quantitatively evaluate the voids area and ratio of each segments and then conduct a topdown and in-depth voids distribution along the Z-axis.

2.5 Constant-head permeability test

The indoor constant-head permeability testing device was assembled in-house to measure the permeability of pervious concrete with 150mm (6 inches) in diameter, shown in Figure 5. The constant head was set to 300mm and the outflow water was collected for fifteen seconds. The permeability K_T (cm/s) is computed as follows in Equation (3):

$$K_T = \frac{L}{h} \times \frac{Q}{A(t_2-t_1)} \quad (3)$$

Where: K_T : permeability (cm/s); L : sample height (cm); h : head difference; (t_2-t_1) : 15 seconds adopted; Q : volume of outflow water collected (cm³); A : area of concrete cylinder (cm²).

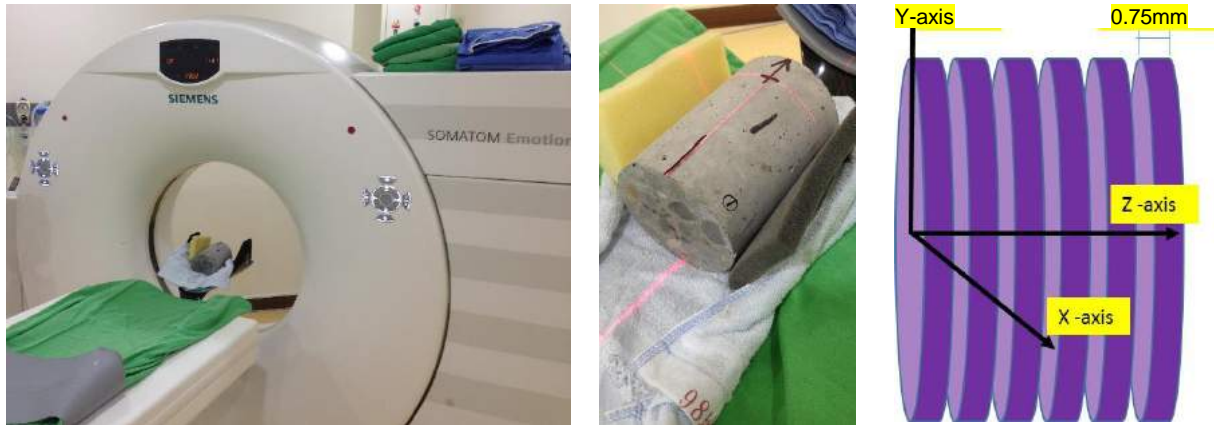


Figure 4. Scheme of performing the X-ray CT scan and the orientation of the segments (Photos taken by Dr. Yu-Min Su).



Figure 5. Scheme of constant-head permeability test (Photos taken by Ping-Ni Hou).

3 TESTING RESULTS IN COMPRESSIVE STRENGTH, UNIT WEIGHT, AND UPV

Table 2 shows experimental results in average compressive strength (MPa/psi), unit weight (kg/cm³), and 3, 12, 21, 28-day of pulse velocity (m/s). In terms of compressive strength, the larger the nominal aggregate size was, the stronger the strength exhibited. The tendency of unit weight followed the same trend as that of the compressive strength. As posed to the ultrasonic pulse velocity, Chandrappa and Biligiri,(2016) adopted 0.25 of the Poisson's ratio and so was this pilot study. The 3, 12, 21, 28-day of dynamic elastic modulus (GPa) were able to be converted by adopting equation 2 and the outcomes were shown also in Table 2. Both UPV and dynamic elastic modulus increased as the hydration time increased. More investigations are needed to comprehend the concrete consensus as well as UPV or dynamic modulus.

Table 2. Compressive strength, unit weight, and UPV of previous concrete

	P1	P2	P3
Compressive strength (MPa/psi), 28-day	8.52/1235.42	9.99/1449.15	11.26/1632.66)
Unit weight (kg/cm ³)	1731	1874	1907
UPV (m/s), 3-day	-	3928.57	4251.59
UPV (m/s), 12-day	3792.83	-	-
UPV (m/s), 21-day	-	4271.14	4320.37
UPV (m/s), 28-day	4018.00	4327.29	4387.56
E _d (GPa), 3-day	-	24.10	28.23
E _d (GPa), 12-day	22.47	-	-
E _d (GPa), 21-day	-	28.49	29.15
E _d (GPa), 28-day	25.21	29.24	30.06

4 ANALYSIS OF X-RAY COMPUTED TOMOGRAPHY

Table 3 indicated results in terms of air voids: the measured interconnected voids, the estimated by X-ray CT, and permeability. It is worthy to note again that the percentage of interconnected voids was measured and calculated by equation 1 and the percentage of air voids was an average of what estimated by X-ray CT. Figure 6 showed the top-down voids analysis along with the Z-axis of the pervious concrete cylinder. In-depth voids on each level of a cylinder can be extracted to be analyzed. Both voids presented that P2 appeared much porous than P1, followed by P3, which were consistent with results in permeability. Both measured and estimated voids are seemingly correlated and more specimens are needed to scan and relate in the future.

Table 3. Measured interconnected voids and air voids estimated by X-ray CT

	P1	P2	P3
Percentage of interconnected voids (%)	26.6	26.5	19.9
Percentage of air voids (%), by X-ray CT	22.2	24.0	19.0
Permeability (cm/s), constant-head	1.53	1.61	1.28

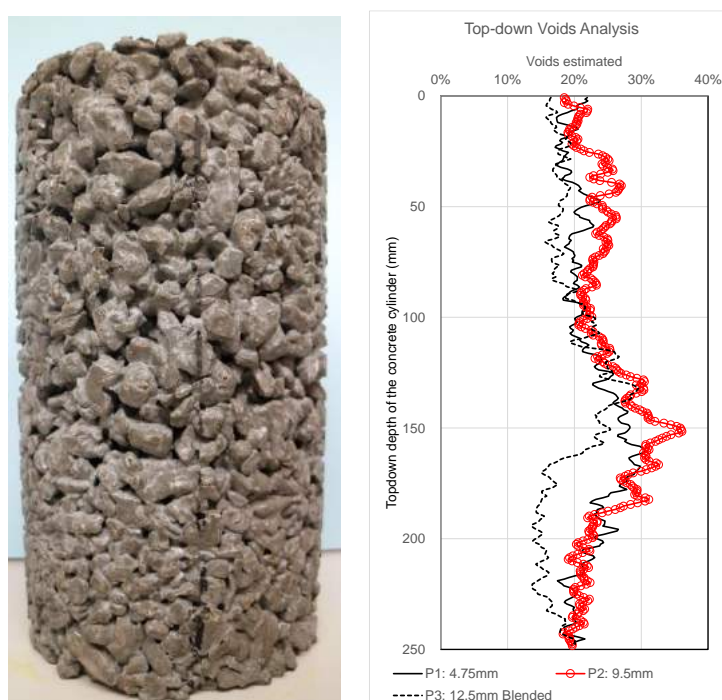


Figure 6. Top-down voids analysis (Photos taken by Ping-Ni Hou).

5 CONCLUSIONS

Three pervious concrete cylinder proportioned with different fractionated coarse aggregates were cast and assessed by pulse velocity and X-ray CT. Several preliminary results are concluded as follows:

- In terms of fresh and hardened concrete properties, the unit weight and compressive strength of P3 mixture containing blended 12.5mm coarse aggregates were higher than those of P1 and P2 composed of single fractionated aggregates. However, it was not a surprise to find that the percentage of interconnected voids of P3 was the lowest in comparison with these of P1 and P2.
- In terms of UPV or computed dynamic modulus, both increased with the hydration time of 3, 12, 21, 28-days as well as the fresh and hardened concrete properties, namely unit weight and compressive strength. More evaluations are required to perform.
- The top-down voids analysis on each pervious concrete cylinder performed by the X-ray CT can be extracted to evaluate the internal voids nondestructively. The average voids of P1, P2, and P3 estimated by X-ray CT were aligned with the measured interconnected voids, as well as permeability. However, more studies on the correlation are needed in the future.

6 ACKNOWLEDGEMENTS

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Enhancing Road Safety in Nepal: a systematic approach

Background

Nepal like many other developing countries, could not give due priority for Road safety in the last few decades. There was lack of Lead agency to plan, design, implement, coordinate and monitor activities related to road safety which caused Road safety as nobody's business. Although there were some individual efforts put by various stakeholders from some petty budgets available to them there was neither coordination nor continuity of such programs. There was absence of dedicated budgets and hence dedicated staff to carry the road safety agenda ahead. Road safety was given priority only when there were some donor funded projects. It was also talked and highlighted by everybody for a while when there were some major accidents causing huge fatality. Some good works done by a British funded project in the nineties on a major highway linking Kathmandu, the capital city has also lost continuity and a Unit created then known as Traffic Engineering and Safety Unit, TESU got function less in course of time. Road Safety Audit practice initiated at this stage was also not continued.

While, the Road traffic was growing annually at double digit, the road conditions were not up to the mark and road accidents were alarmingly increasing. Precious lives which could be saved were getting lost every day and its cost on society was huge. There were commitments from Nepal to curb down the rate of accidents significantly by 2020 in line with UN Decade of Action for Road safety and UNESCAP targets. The UN Global Action mandates member countries to develop their individual national plans for the decade (2011 to 2020) incorporating interventions under the following five pillars to road-safety.

- Road safety management
- Safer roads and mobility
- Safer vehicles
- Safer road users
- Post-crash response

It was urgently required to bring all the stakeholders under one umbrella, coordinate with them and put them into integrated action in a planned and systematic manner. The Government therefore decided Ministry of Physical Infrastructure and Transport, MoPIT as the Lead Agency and approved National Road Safety Strategy and Action Plan 2013- 2020. Road Safety Council is about to be formed. The plan is now put to implementation and positive improvements are now seen in the field of Road Safety.

The Status of Roads and Road safety in Nepal

The rate of accident in Nepal is quite high compared with neighbouring Asian countries. There are about six deaths every day on average. The total number of vehicle is 1.7 Millions. Based on this figure rate of fatal accident per 10000 vehicles per year are 12. If we exclude one million two wheelers from the total, the rate works out to be 33. This figure is still too high. Awareness campaigns are weak and there is even question about under reporting of the rate of accidents. The post accident care and presence of trauma hospitals are insignificant.

The following are the findings of RTAs in Nepal based on past research and monitoring.

- About half of all the RTAs nationwide occur in the Kathmandu Valley alone where nearly half of the country's fleet ply.
- Severity of the RTA injuries in the Kathmandu Valley is less pronounced compared to the rural areas and RTA fatalities amongst the vehicle fleet are higher in the regions outside the Kathmandu Valley.
- Pedestrians are the most vulnerable groups because pedestrian-safety has not been duly considered and People between 15 to 40 years of age are most affected in road-accident followed by those above 50 years.

- Urban areas suffer from significant number of motor-cycle accidents. In the rural areas, there are significant number of trucks and bus accidents.
- Bus accidents along the long-distance routes are of serious concern accounting for 13% and 31% of all the fatalities and serious injuries, respectively. Single bus accidents where the vehicle runs over the hill-roads represent the fatal RTAs of catastrophic proportions.
- 30 to 40% of the accidents happen after sunset when traffic is low. Accidents tend to cluster at Urban intersections, bridge approaches, and roadside built-up areas.
- Driver negligence, drunk driving, overtaking, over speed , random roadside parking, reckless pedestrian crossing, poor road conditions, etc., were identified as the major causes of accidents.
- The economic loss from RTAs in Nepal was at least NRs.22.7 billion(US\$ 41.2 million) annually or 0.4 % of the GNP at 2007 price. When the accident under-reporting are adjusted, the loss was 0.8% of the GNP annually. There are numerous safety issues on the Nepalese hill roads (which form a substantial portion of the road network) such as poor visibility at blind corners; poor shoulders; unforgiving side-drains, inadequate safety barriers at steep vertical drops; unscientific location of passing bays in single lane roads; lack of climbing lanes; very steep gradients at numerous sections, narrow sections at built-up areas, etc..

Table 1: Traffic Growth in Nepal

S.N.	Year	Vehicle	Growth %	Remarks
1	2004/05	482464	9.2	Motor
2	2005/06	537439	11.4	bikes
3	2006/07	625174	16.3	included
4	2007/08	710914	13.7	
5	2008/09	813484	14.4	
6	2009/10	1015271	24.8	
7	2010/11	1178911	16.1	
8	2011/12	1348995	14.4	
9	2012/13	1557478	15.5	
10	2013/14	1755821	12.7	
11	2014/15	1995404	13.6	
12	2015/16	2339169	17.2	

Average growth over last
12 years = 14.90%

Table 2 : Traffic Composition in Kathmandu Valley

Bus /mini	truck	Car/ van	motorcycle	others	Total
20214	18929	149264	722722	11971	923100
2.2	2.0	16.2	78.3	1.3	100

A Total of 43 % of Nepalese vehicles ply in Kathmandu valley out of which the share of Motorcycle is about 78%.

Figure 1 : Road Networks of Nepal

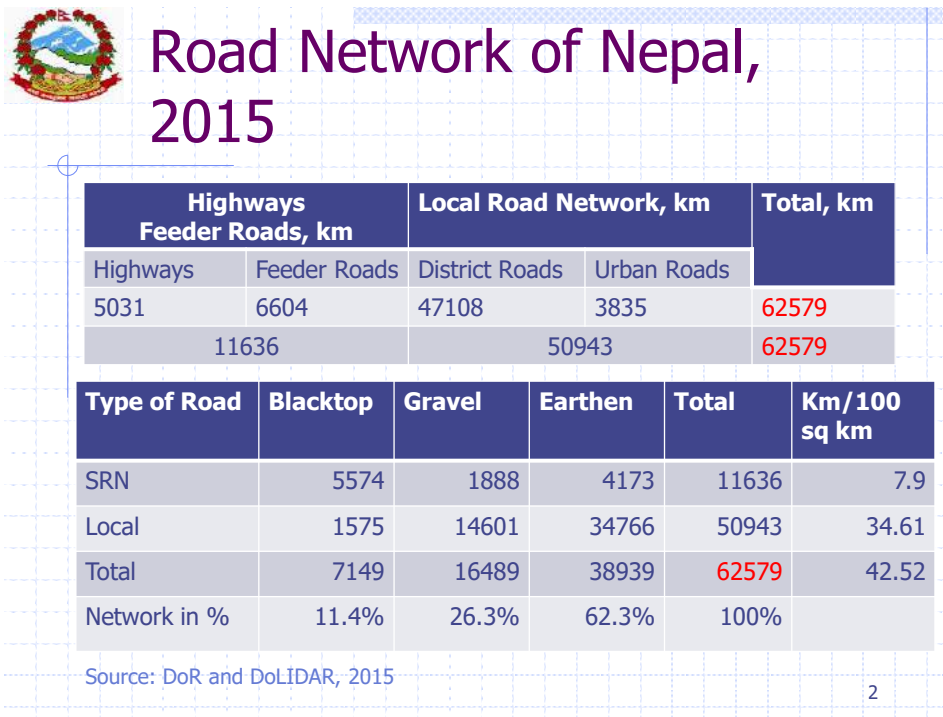


Table 3 : Trends in Road Accident

Year	Accidents	Fatalities	Serious injury	Slight injury	Injury/fatal ratio	Total vehicle	Fatality per 10000 veh
2001/02	3823	879	458	4138	5.23	362828	24.2
2002/03	3864	682	785	4442	7.66	400438	17.0
2003/04	5430	802	1659	3925	6.96	440137	18.2
2004/05	5532	808	1795	4039	7.22	480668	16.8
2005/06	3894	825	1866	3655	6.69	536443	15.4
2006/07	4546	953	2583	5331	8.30	625179	15.2
2007/08	6821	1131	2663	5245	6.99	710917	15.9
2008/09	8353	1356	3609	6457	7.42	813487	16.7
2009/10	11747	1734	4130	7383	6.64	1015271	17.1
2010/11	14013	1689	4018	8503	7.41	1178911	14.3
2011/12	14297	1837	4018	7811	6.44	1348995	13.6
2012/13	13582	1816	3986	8000	6.60	1557478	11.7
2013/14		1839				1660250	11.1
2014/15		2004				1995404	10.0
2015/16	10013	2006	4882	8166	6.50	2339169	8.6

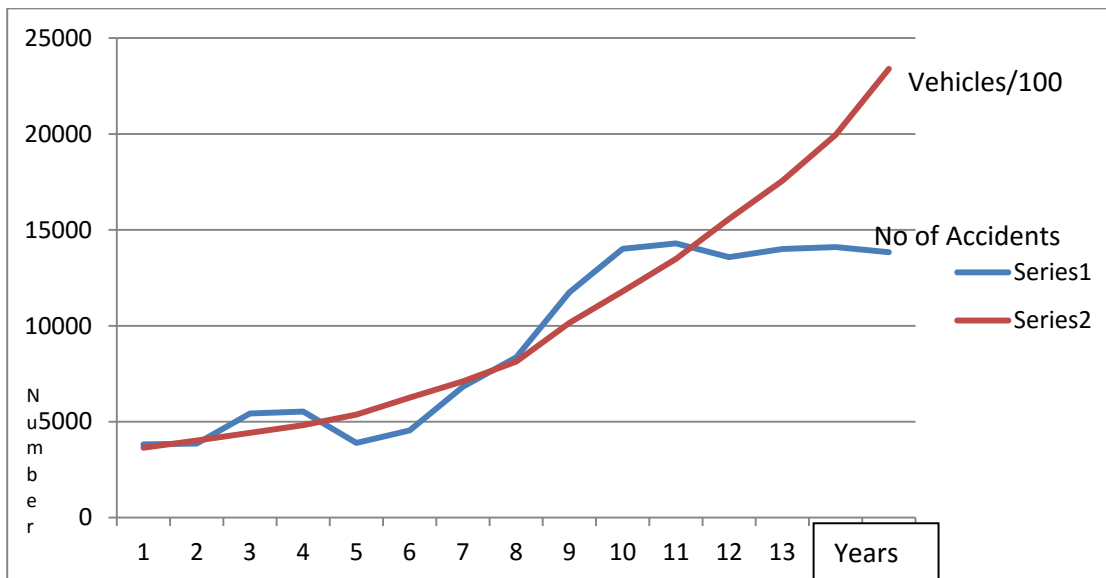
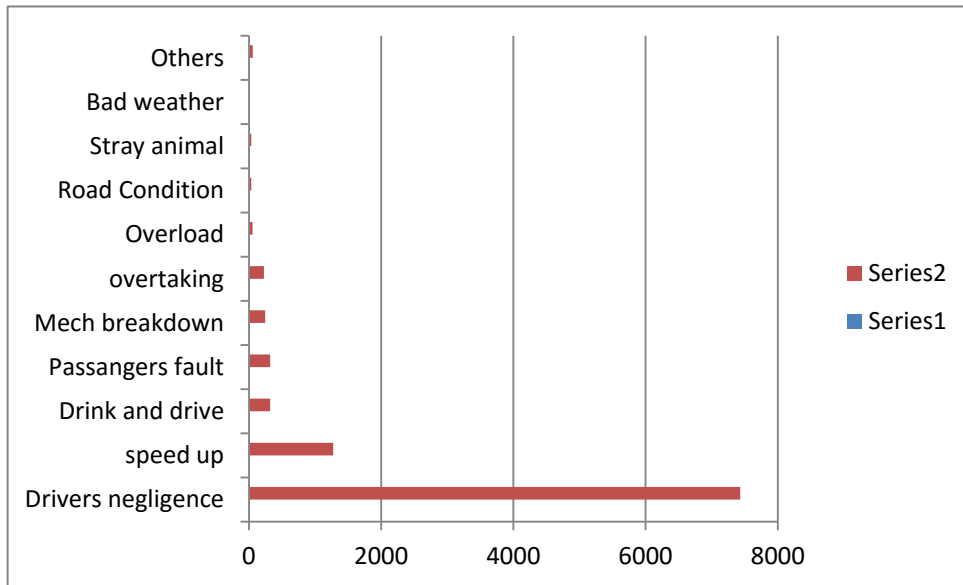


Table 3 Analysis of causes of accidents 2015/16

S.No	Causes	Number	Percentage
1	Drivers negligence	7433	74.2
2	speed up	1273	12.7
3	Drink and drive	321	3.2
4	Passangers fault	321	3.2
5	Mech breakdown	245	2.4
6	overtaking	226	2.3
7	Overload	53	0.5
8	Road Condition	34	0.3
9	Stray animal	34	0.3
10	Bad weather	16	0.2
11	Others	57	0.7
	Total	10013	100

Figure 3 :
Causes of
Accidents



The Strategy on Road safety

Government of Nepal has given top priority to road safety issue in its policy documents and annual budgets. It states about the strategy in **Nepal Transport Policy 2001** as

- **“to develop a reliable, cost effective, safe facility oriented and sustainable transport system that promotes and sustains the economic, social, cultural and tourism development of Nepal as a whole”**
- **Likewise the Vision and Mission are well defined in National Road safety Strategy 2013 as**

Vision: Safe road-infrastructures and services backed with effective post-crash response and conducive environment resulting in little or no casualties from the RTAs.

Mission: (i) To mitigate the loss of life, properties and economic loss from RTAs.

(ii) To complement the broader mission of the National Strategy on the Prevention and Control of Violence, Injuries and Disabilities

(iii) To meet the targets of the UN Decade of Action.

(iv) To provide a common framework for stakeholder agencies to implement the various interventions required to mitigate RTAs outcomes.

In order to implement the strategy Nepal Road Safety Council headed by the Transport Minister is envisaged. There is a steering committee led by Transport Secretary to implement the decisions of the Council. Ministry of Infrastructure and Transport is appointed as the Lead Agency for Road safety policy and actions. The steering committee would also fill the gap till the council is in function backed by appropriate legal mandates.

The Action Plan

The action plan is prepared so as to achieve the target of reducing the road accidents by 50% by the year 2020. The actions are time bound, detailed and supported by resources. A glimpse of the action plan is presented below:

Pillar 1 : Road safety management

The objective of this pillar is to set up mechanism to improve capacity to manage road safety through:

1. Adopt UN Legal instrument
2. Encourage adoption of road safety instruments
3. Increase horizontal coordination among stakeholders
4. Develop sustainable road strategy and accident reduction targets
5. Improve accident data collection and research

Pillar 2 : Safer Roads and Mobility

The objective of this pillar is to improve the inherent safety of road networks for all road users especially the vulnerable groups (eg pedestrians, bicyclists and motorcyclists). This will be achieved through:

Adoption of UN and international standards for the design of safe roads.

- Road safety audits and assessment
- Incorporating safe design practice during design, construction and operation of roads.

Pillar 3: Safer Vehicles

The objective of this pillar is to promote the universal adoption of both the active and passive technologies that are available for safe vehicles through the harmonization with the global standards, publicity and incentives for the consumers in their adoption.

Pillar 4 : Safer Road Users

The objective of this pillar is to develop comprehensive programmes to improve road-user behavior through the following activities:

- Sustained, stronger enforcement of traffic rules.
- Sustained road-safety awareness campaigns.
- Increased efforts to improve the use of seat-belts and helmets.
- Reduce drunk-driving and other risky behaviours
- Introduce better speed control
- Heavy penalty to undisciplined road-users including pedestrians.

Pillar 5 : Post crash Response:

The objectives of this pillar are to improve the post-crashes response, improve capacity of the health-care systems to provide emergency treatments and long-term rehabilitation for crash victims.

The Action plan is quite detailed with responsibility assignment, time frame and resource allocation.

Ongoing initiatives

Nepal is one of the signatories of Busan declaration 2006. The declaration aims to save 600,000 lives on the roads of Asia and the Pacific by a reduction of 20 per cent in the fatality rates and in the rates of serious injuries over the period 2007 to 2015 with following strategies:

Strategies:

- Making road safety a policy priority;
- Making roads safer for vulnerable road users, including children, pedestrians and motorcyclists;
- Making roads safer and reducing the severity of accidents (building “forgiving roads”);
- Making vehicles safer;
- Improving road safety systems, management and enforcement;
- Improving cooperation and fostering partnerships;
- Developing the Asian Highway as a model of road safety;

Steering Committee has been formed led by Secretary and including major stakeholders. This steering committee has started its work for budgeting and programming.

A separate budget head is assigned so as to support the Road safety program in a sustained manner. Donors have shown their interest on road safety and supports are being materialized.

Kathmandu Sustainable Urban Transport Project, KSUTP has started working towards improving the Footpaths, junctions and some key roads in the valley. It is constructing couple of bridges for pedestrians crossings and is improving the traffic lights in 3 junctions. It plans to establish a central traffic control room.

Government of Nepal has done substantial amount of Road widening/ Junction improvement works in the Kathmandu valley as well as major cities. The program is ongoing.

Recently Road rating Survey was carried out for 700 km of Highways with the help of iRAP . A Regional TA from ADB on Road safety enhancement in Nepal and Bhutan has completed in 2014.

Road safety audit for major highways have begun. Based on the survey results, Road safety barriers are being installed on 70 km of sample road such as Kathmandu Mugling road, Araniko highway and Karnali Highway.

Awareness programs are being organized by the stakeholders. Recently there was a traffic safety exhibition organized by Traffic Police in Kathmandu. This exhibition was visited by large number of people of different age group and was well appreciated.

Stakeholders are now collaborating to reduce Road Traffic Accident. Road safety society, a NGO is formed so as to enhance the Road safety activities

The traffic police have come up with a successful program of checking and penalizing drink and drive. The data shows this initiative has helped reducing the RTA in Kathmandu valley substantially.

Rewards and prizes are being distributed for best works on road safety.

Traffic Road safety week is organized regularly by Traffic Police. Media attention and coverage on Road safety is appreciable and increasing and the Civil Society gets active when there is a major accident on Nepal roads.

Nepal Automobiles sports Association, NASA has joined hand to promote Road safety activities.

The Government has decided to remove old public vehicles (older than 20 years) from Kathmandu.

Road upgrading projects have the mandatory provision of road safety elements and Road safety Audit.

Fine for Traffic rule violation has been doubled despite some resistance from drivers.

A regulation have been passed in 2014 which provisions for specific qualifications including age limits and separate license exams for obtaining driving license as Public vehicle driver.

Government has begun investigation of RTAs. There is a National Accident Investigation Committee formed in case of Major accidents on case basis and other accidents are investigated through a permanent committee chaired by the Regional Roads Director.

ADB, DFID and World Bank are supporting Road Safety Initiatives in Nepal and the new MCC project is planning Road safety as one of its project component.

Future Plans

The Ministry of Physical Infrastructure and Transport, MoPIT plans to amend prevailing Acts and regulations so as to specify clearly the role and responsibilities of major stake holders. This will also enable to establish a National Road safety Council headed by Transport minister. The Department of Transport Management, DoTM has delegated some of its power to Nepal Traffic Police. This has enabled the traffic police to take action against those who violate traffic rules. Driving License tests are being made more practical and scientific. There is a plan for separate and more rigorous license tests for public vehicle drivers. Necessary legislation is now available for this. The Driving Training Institutes which are envisioned by existing laws as an integral part of the licensing process will be better managed and assisted to perform their crucial roles. More works will be done under the upcoming phase of Kathmandu Sustainable Urban Transport Project. Likewise mass transport and Environmentally Sustainable Transport, EST will be promoted in the near future. The old vehicles will be replaced gradually. The nation will have a federal structure completed in two years and the Traffic and Road Safety activities will be performed from provincial governments as well in line with the program of the National Road Safety Council.

Conclusion and Recommendations

There were sincere and serious efforts in Nepal for last couple of years towards enhancing Road safety. The National Road Safety Strategy is now in place. The Road safety action plan 2013- 2020 is prepared, approved and put to implementation. This plan is well backed by necessary resources: internal as well as external. The stakeholders are gradually getting integrated and collaborating for the common cause of enhancing road safety. Donor communities are also keen to support the positive initiatives taken by the government. New roads construction including upgrading works is now subject to proper safety audit. Works are in the final phase to draft a separate Road Safety Act. The works performed so far has started producing results. The rate of accidents of last three years has stabilized. Considering the large growth rate of vehicles it can be seen that number of accidents and the fatalities per 10000 vehicles are reducing during last five years. There is now expectation and confidence among stakeholders that we can deliver if we are united. This initiatives needs to be continued with additional backings of resource and legal mandates. Since things are now positively moving good results are expected to come by 2020. The political commitment, the budgetary provisions and stakeholder supports exhibited in last couple of years has laid a very good foundation for road safety enhancement in a systematic manner.

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Investigating the Contributing Factors Affecting Traffic Crash Caused by Young Drivers

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ABSTRACT

Young drivers are considered as most risky in driving compared to other age groups of drivers. Understanding the contributing factors affecting traffic crashes caused by young drivers leads to more effective actions for improving road safety. This paper aims to examine the contributing factor affecting the occurrence of young drivers' crashes. Crash data from 2010 to 2015 has been collected from Abu Dhabi Traffic Police database. At-fault drivers' aged between 18 to 24 years is defined as the young drivers in the analysis process. Descriptive statistics show that 23% of severe crashes are caused by young drivers. These crashes resulted 26% of the total fatalities of traffic crashes. Logistic regression model was developed to identify and quantify the variables that affect the occurrence of crashes related to young drivers. The model findings indicated that speeding, sudden lane changing and tailgating are the most caused of young drivers crashes. Young drivers are probably to be involved in multi-vehicle crashes and pedestrian-related crashes. Regarding the environmental factors, young drivers have higher risk at night and at intersections compared to older drivers.

INTRODUCTION

Each year, about 1.2 million people are killed, and up to 50 million are injured in road because of traffic crashes ⁽¹⁾. In 2008, HAAD (Health Authority – Abu Dhabi) report ⁽¹⁾ documented that for each 3 fatal in Emirate of Abu Dhabi (the capital of United Arab Emirates), 2 of them related to traffic crashes. Known the human factors are consider as the major cause of traffic crashes, driver’s age is one of these main factors that have significant contribution in crash occurrence.

Recently, many researchers giving attention to the young drivers because of the combination of youth and low inexperience puts them at high risk level. Their inexperience means they have less ability to spot hazards, and their youth means they are particularly likely to take risks but they can’t act correctly during critical situations. In Abu Dhabi (AD), statistics show that 23% of sever crashes and 29% of overturned crashes were related to young driver. In addition, 26% of the fatality and 23% of the serious injuries were resulted in crashes caused by young drivers during the last 6 years (from 2010 till 2015) as shown in table 1. Despite the increasing on the percentage of young driver’s overturned crashes from 26% in 2010 up to 35% in 2015, the number of these type of crashes occurred by young driver has been decreased in the same period.

Table 1: Statistics related to young drivers in Abu Dhabi

Abu Dhabi Emirate	2010	2011	2012	2013	2014	2015	Average
Total number of crashes related to at-fault driver	2648	2382	2115	2115	1905	1857	2170
Severe crashes (*)	598	552	489	462	418	428	491
% of crashes (*)	23%	23%	23%	22%	22%	23%	23%
Total number of overturned crashes	468	431	354	305	254	270	347
Number of overturned crashes (*)	120	120	99	90	80	95	101
% of overturned crashes (*)	26%	28%	28%	30%	31%	35%	29%
Total number fatalities	376	334	271	289	267	245	297
Number of fatalities (*)	96	98	74	53	61	74	76
% of fatalities (*)	26%	29%	27%	18%	23%	30%	26%
Total number of serious injuries	400	390	364	366	240	293	342
Number serious injuries (*)	78	109	84	74	51	68	77
% of serious injuries (*)	20%	28%	23%	20%	21%	23%	23%

(*) related to young driver crashes

In general, there is no much information in regard of young driver behavior in the Gulf Corporation Council (GCC) countries. Hence, the interaction between young drivers’ behavior in terms of their involvements in traffic crashes has not been much investigated or published in prior studies. In (AD), a tangible effort has recently been done to improve the road safety through the three main strategic approaches: Engineering, Education and

Enforcement. Accordingly, road crash fatalities have reduced from 376 in year 2010 to 245 in year 2015 (about 35% reductions).

Understanding the young driver behavior is the key factor to maintain a proper reduction rate of fatalities and improving the road safety strategies. Especially in the case of UAE where more than two hundred different nationalities are living along with different education levels, culture, language, and driving skills background which creates a considerable challenge for different AD agencies that are responsible for road safety. Accordingly, the main objective of this study is to investigate the contributing factors affecting young driver risk potential to be involved in a traffic crash. Relationships between at-fault young drivers involved in traffic crashes and their demographic characteristics were investigated.

LIRETUTRE REVIEW

Who are the young drivers? In the United States drivers classified as young when they fall within the age group between 15 and 20 years old ⁽²⁾. UK drivers know the young drivers as those drivers aged between 17 and 24 years ⁽³⁾. In Australia young drivers are all drivers who hold driving licenses and under the age of 25 years ⁽⁴⁾. OECD countries, which include 41 countries, including Germany, Sweden, the Netherlands, Switzerland, Japan and South Korea know young drivers as drivers aged between 18 and 24 years ⁽⁵⁾.

Based on World Health Organization (WHO) ⁽⁶⁾, more than 1,000 people under the age of 25 years are killed in road traffic crashes around the world every day. Road traffic injuries are the leading cause of death globally among 15–19-year-olds, while for those in the 10–14-years and 20–24-years age brackets they are the second leading cause of death.

Even in countries that have high level of road safety indicators such as OECD countries more than 8,500 young drivers die in every year and death rates for young drivers are about double those of older drivers. In these countries, young drivers represent about 27% of all drivers' fatalities, despite the fact that they represented only about 10% of the population. In particular, the young drivers' fatalities included nearly 4,000 fatalities in the United States, more than 750 in Germany, 645 in France, and more than 300 in Japan and Spain ⁽⁵⁾. As a share of all driver fatalities within the EU countries, the proportion of fatalities for young drivers ranges from 18 % in Denmark to 32% in Germany. In contrast, the share of this age group in the total population ranges from 8% in Denmark to 13% in Ireland. In New Zealand (2013) young drivers aged 15–24 were involved in 71 fatal traffic crashes, 485 serious injury crashes and 2,581 minor injury crashes. Of these crashes, the 15–24 year-old drivers had the primary responsibility in 55 of the fatal crashes, 394 of the serious injury crashes and 1,932 of the minor injury crashes. These crashes resulted in 64 deaths, 512 serious injuries and 2,659 minor injuries. The total social cost of the crashes in which 15–24 year-old drivers had the primary responsibility was \$737 million. This is 24 percent of the social cost associated with all injury crashes ⁽⁷⁾.

On the other hand, many studies have been conducted to examine the severity of young driver related crashes and their contributing factors. Curry, et al. (2015) ⁽⁸⁾ investigated the factors affecting the young driver related crashes. The data has been selected from all drivers who have an intermediate license and aged between 17-20 years

old. The outcomes indicated that the crash rates were highest in the first month of obtaining the license. These rates started to decrease and once the driver transition his intermediate license to full license the rates increased again. Amarasingha, et al. (2014)⁽⁹⁾ tried to find the deference in crash involvement between young female driver and young male driver. Drivers from 15 to 24 years were considered as young drivers. A logistic regression model was developed to identify the Factors which increase young female drivers' injury severity and young male drivers' injury severity. The outcomes showed that the rear-end collision & collision with vehicle are the factors affected the severity of male driver. For the female driver the overturn and run over crashes are the two factors that affect the crash's severity.

Alver, et al. (2014)⁽¹⁰⁾ evaluated traffic violation getting by young driver and the effect of this behavior on the traffic crashes. The data has been collected from face-to-face questionnaire and the target driver is the young driver aged 18-29 years old. Ordered probit model & binary logit model has been used. The results revealed that the traffic crash rate was 38.3% for those who stated that they received at least one violation during the past 3 years. Dissanayake, S. (2004)⁽¹¹⁾ investigated the effect of the single vehicle crashes by young driver on crash severity. The young driver considered as 16-25 years old. A logistic regression model was used to develop a model to predict the crash severity. The results suggested that the speeding and not using a seat belt were the two most factors affect the crash severity. Dissanayake, S. (2004)⁽¹²⁾ developed a model to identify the factor affecting the severity of young driver related crash. The single vehicle crashes data has been used in the model. The outcome showed that the speeding and not using a restraint device (for example the seat belt) were the most two factors increase the crash severity. Also, the young driver crash severity determined high when the crashes occurring in the curve or on the bridge.

Chliaoutakis, et al. (2002)⁽¹³⁾ examined the influence of aggressive behavior for the young driver on the traffic crashes. The data has been collected from 356 participants who aged between 18-24 years using self-reported questionnaire. The result showed that the young drivers who drive without any predefined their trip destination are more likely to involve in car crashes. In other words, the young drivers who use their car just for fun and not as a transportation mean are causing many crashes. Also, it found that the age is a significant factor in traffic crashes. McGwin, G. & Brown, D, (1999)⁽¹⁴⁾ conducted a study to identify the characteristics of young driver related crashes. The data has been collected from police –reported crashes which occurring during year 1996. The results explained that the young driver cause a traffic crash because of skill shortage and run the risk.

According to the previous statistics and the literature review the objective of this paper is to investigate which factors are affecting to the crashes caused by young driver in Abu Dhabi Emirate.

DATA SOURCE & DESCRIPTION

The employed data in this study was extracted from the traffic crashes database of Abu Dhabi Traffic Police for six years from 2010 to 2015. Severe crash (i.e. any crash involving at least one injury) data are used in this analysis due to the limitation of the available property damage only crash data. The database includes different sets of data

groups; basic crash information, at-fault drivers and vehicle's characteristics, casualties' information, environmental and surrounds condition, traffic violations, etc. Full data of about 12,611 crashes has been extracted and utilized in this study. It is worth mentioning that the driver's community in AD consists of more than one hundred different nationalities. Male drivers represent 85% of total number of licensed drivers and 92% has age less than 45 years. Approximately, 2947 crash report related to young driver has been used in the analysis. Young driver is defined as the driver aged between 18 to 24 years old.

To achieve the objectives of this study, detailed descriptive statistical analysis will be firstly provided. After that an investigation of the contributing factor that affects young drivers' related crashes will be conduct by applying logistic regression modeling approach.

STATISTICAL DESCRIPTIVE ANALYSIS

Comparisons between at fault young drivers and others was conducted in terms of the frequency of traffic crashes, crash severity, crash types, crash causes and traffic violations during the period 2010 till 2015 which occurred in Abu Dhabi Emirate. Figure 1 illustrates the distribution numbers of crashes and fatalities resulted from the young driver during 2010 till 2015 in Abu Dhabi Emirate. It's observed that in spite of crashes number decreased by 28% during study period, the percentage of these crashes out of total crashes has almost the same trend. On the other hand, although their fatality has been declining significantly during the study period by 23%, the percentage of these fatalities out of total crashes fatalities have significantly increased.

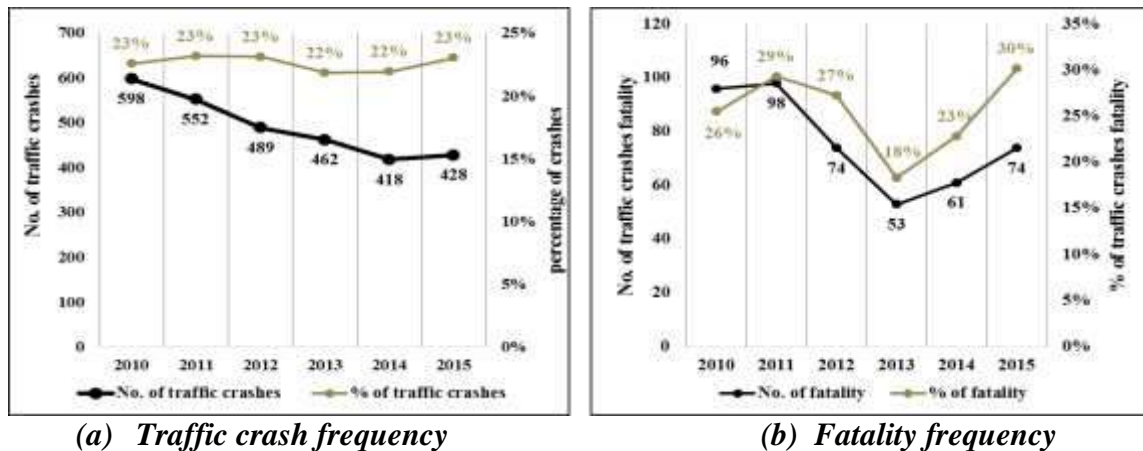


Figure 1 Young drivers traffic crashes and resulted fatalities

Figure (2) presents the severe crash rate per 10,000 drivers for both young drivers and other drivers. In general, a significant increase (about 54.5%) in the young drivers fatality crashes rate was observed in the last six years. Which improve the theory of classify young drivers as a critical driver with regards to traffic safety.

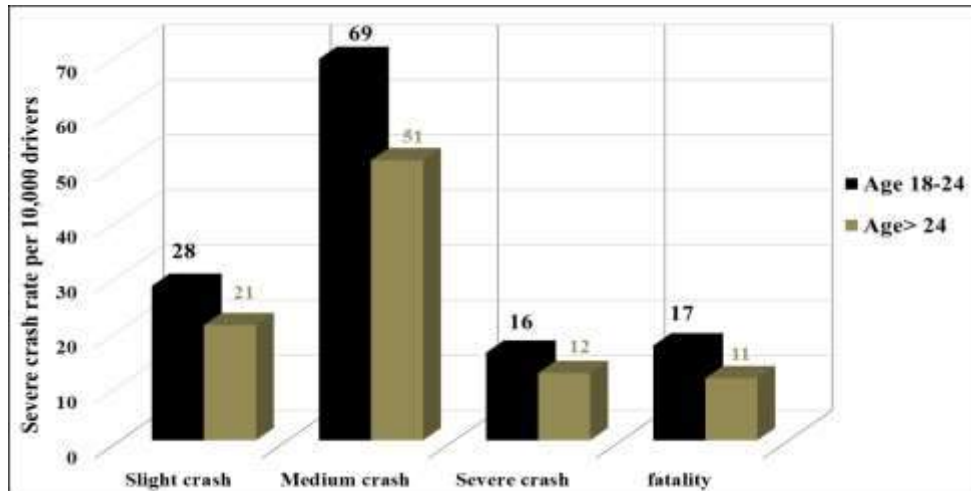


Figure 2 Crashes severity during 2010-2015

Table 2 shows the characteristic of at-fault driver involved in severe crashes. The table shows that the majority (about 60%) of at-fault young drivers were local. In addition, it was observed that the most age of those at-fault young drivers was 23 years old, which seems that young drivers gain more confidence at this age and they tend to commit many mistakes. These findings prove the necessity of conducting educational campaigns at Universities, Colleges and high schools where young at this age found.

In addition, table 2 indicates that 74% of at-fault young driver were obtained their driving license from Abu Dhabi Emirate and 13% from the other Emirates. Regarding the gender, the male young drivers were more likely to cause a severe crash than the female for both age groups (young driver & old driver).

Table 2: The characteristic of at-fault drivers who involved in severe crashes in Abu Dhabi during 2010-2015

Variable	categories	18 -24 year		> 24 year	
		Frequency (average)	Percentage	Frequency (average)	percentage
Gender	male	454	92.4%	1417	89.1%
	female	38	7.6%	173	10.9%
Nationality	UAE	293	59.6%	362	22.7%
	GCC	22	4.4%	52	3.3%
	Arab countries	69	14.1%	358	22.5%
	Asian countries	105	21.4%	767	48.2%
	Other countries	3	0.6%	52	3.2%
Source of driving license	Abu Dhabi	366	74.5%	1106	69.5%
	Dubai	27	5.4%	228	14.3%
	Other emirates	35	7.2%	126	7.9%
	GCC	21	4.2%	41	2.6%
	others	35	7.2%	91	5.6%

Figure 3 shows that the crash percentage decreases with increasing the experience years of driving up to 20 years. After 20 years of experience the crash percentage increases that can be explained due to the elder age of the drivers.

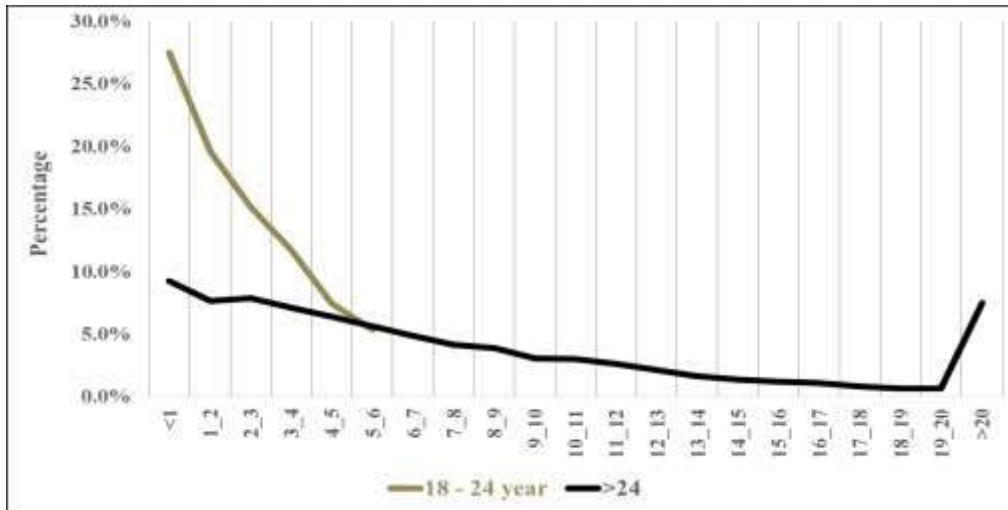


Figure 3 Traffic crashes depend on driver experience

Table 3 illustrates the characteristic of young driver related crashes. For both young drivers and others, it can be clearly observed that 82% of the injuries and fatality resulted from young driver crashes were classified as slight to middle injuries and 18% were classified as severe & fatal injuries. In addition, table 3 shows the percentage of the highest seven causes of traffic crashes related to young drivers and others. Sudden lane change (19%), driving too fast (15%) were the main causes of young driver crashes. In general, non-significant differences exist between young drivers and others in terms of the causes of traffic crashes except a slight increment for those due to speeding. Table 3 also presents the percentage of the highest five types of traffic crashes related to young drivers and others. It was found that the highest percentage of the crash type was overturned (21%), followed by pedestrian crashes (17%), rear-end (16%), sideswipe crashes (14%) and angle crashes (13%). It shows non-significant differences in the percentage share values of the crash types over the six years for most crash types except a significant increment in overturned crashes related to young drivers.

With respect to the land use where the crash occurred the analysis shows that (26%) of young drivers crashes occurred on the rural area, (25%) occurred on residential area and (16%) occurred on commercial area. Regarding to the crash location, the majority of young driver related crashes occurred at non- intersection (74%), while about (15%) occurred at intersection.

Table 3: The characteristic of young driver related crashes

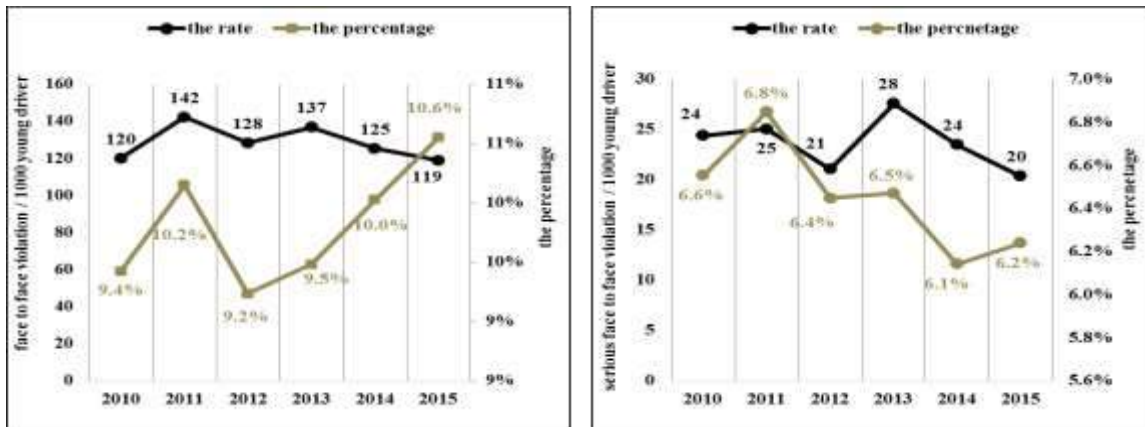
Variable	categories	18 - 24 years		> 24 years	
		frequency	percentage	frequency	percentage
Injury severity	Slight Injury	1893	37.5%	6312	39.6%
	Medium Injury	2232	44.2%	6824	42.8%
	Severe Injury	464	9.2%	1509	9.5%
	Fatalities	456	9.0%	1295	8.1%
Crash causes	Sudden lane change	562	19.1%	1575	17.0%
	Careless driving	258	8.8%	804	8.7%
	Tailgating	313	10.6%	1044	11.3%
	driving too fast for conditions	442	15.0%	985	10.6%
	Not following road directions	225	7.6%	679	7.3%
	un respecting of road user	194	6.6%	742	8.0%
	running the red light	201	6.8%	997	10.8%
	Others	730	24.8%	2445	26.4%
Crash Type	Overtuned	604	20.5%	1317	14.2%
	Rear-end	471	16.0%	1515	16.3%
	Sideswipe	403	13.7%	1327	14.3%
	Pedestrian	506	17.2%	1777	19.2%
	Angle	379	12.9%	1524	16.4%
	others	562	19.1%	1809	19.5%
Area type	Rural area	764	25.9%	2496	26.9%
	Commercial area	473	16.1%	1932	20.8%
	Residential area	747	25.3%	1939	20.9%
	Others	941	31.9%	2904	31.3%
Crash Location	At Non-intersections locations	2186	74.2%	6238	67.3%
	At intersections	432	14.7%	1918	20.7%
	roundabout	220	7.5%	641	6.9%
	Others	87	3.0%	474	5.1%

Number of traffic violations can be used as an indicator of drivers' behavior changes. The database includes two types of violations; in presence (by Face-to-Face tickets) and in absent (by automated enforcement devices). On this study we depended on (Face-to-Face tickets) data to be sure that the violator driver was belongs young drivers' category. Table 5 shows that the total number of Face-to-Face traffic violations increased during the years 2010 to 2015 by 37%, while the total number of young drivers' Face-to-Face traffic violations increased by about 45%.

TABLE 4: Face-to-Face Traffic Rule Violations Records

Face-to-Face Violation	Years					
	2010	2011	2012	2013	2014	2015
Total	504,599	615,907	641,314	745,159	719,156	691,220
Young Drivers	26,959	35,329	34,616	39,411	38,429	38,987
Percentage	5%	6%	5%	5%	5%	6%

Face-to-Face (over-Speeding traffic violations) represented about 11.31% of the total number of Face-to-Face traffic violations over study period, while (young drivers' Face-to-Face over-Speeding traffic violations) represented about 11.01% of the total number of young drivers' Face-to-Face traffic violations. Moreover, data shows that in spite of slight reduction (17%) of Face-to-Face (over-Speeding traffic violations) over study period, (young drivers' Face-to-Face over-Speeding traffic violations) significantly reduced by (35%).



a) face to face violations

b) serious violations

Figure 4 young driver face to face violations

With respect to Face-to-Face violations figure (4-a) presents the number of Face-to-Face violation for every 1000 young drivers and the percentage of these violations during the period from 2010 till 2015. In general there is no significant deferent in number of Face-to-Face violation during the study period. The highest rate of these violations was in 2011 about 142 violations per 1000 young drivers and the lowest rate was in 2015 around 119 violations per 100 young drivers. Although the rate was decreased during the study period the percentage of those violations was increased. As for serious violations figure (4-b) illustrate the rate of serious Face-to-Face violations and the percentage of these violations out of total Face-to-Face violations. It can be clearly observed that both the rate and the percentage of this type of violations were decreased during the study period.

LOGISTIC REGRESSION MODEL

In order to define the factors affecting the occurrence of young drivers' crashes combined to other drivers' crashes, logistic regression analysis applied. Binary logistic model is a good approach to deal with binary outcomes (1= young drivers' related crashes, 0= other drivers' crashes). Also, as one of the aims of the study was to develop models to predict

young drivers' related crashes, logistic regression was identified as the most suitable approach to identify the important factors.

In case of binary logistic regression model, the response variable, y takes the form of either of the two binary values (0 or 1). For k explanatory variables and $i=1, 2, 3, \dots, n$ individuals, the model takes the form as follows

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

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

α = Intercept parameter,

β = Vector of slope parameters,

X_i = Vector of explanatory variables.

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

$$\text{OR} = \frac{\pi(1)/[1-\pi(1)]}{\pi(0)/[1-\pi(0)]} \quad \dots \dots \dots (2)$$

Where,

OR = Odd Ratio

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

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

The model was estimated by using SPSS software package. The total number of traffic crashes involved in the model estimation is about 12,611 crashes.

Explanatory Variables

Many variables were tested includes vehicle, at-fault driver, roadway, and environment characteristics. At that stage it considered the fact that the quality of the modeling could be expected to increase to a certain level once the number of variables increases. Secondly, selection of the variables was carried out depending on the assumption that a particular variable would affect the young drivers' crashes. The descriptions of 13 explanatory variables that are considered for the modeling are provided along with their statistics in Table 5. All the explanatory variables are binary. Binary variables take the form of either 0 or 1; for example, if a crash occurs on WEEKEND, the variable WEEKEND has been assigned "1" as its value; otherwise "0" is assigned to this variable. One binary logistic regression model was developed by considering crash been related to young driver as the response variable and the description of the model which is binary in nature (YOUNG DRIVER_RELATED = 1 if at fault driver is young driver (18-24 years old), =0 otherwise). The model has been development using SPSS software.

Table 5: Explanatory variable considered in the modelling process

Variable	Description
<i>SUDDEN_LANE_CHANGE</i>	= 1 if crashes occurred because of improper lane change, = 0 otherwise
<i>SPEEDING</i>	= 1 if crashes occurred because of speeding, = 0 otherwise
<i>TAILGATING</i>	= 1 if crashes occurred because of not keeping enough space, = 0 otherwise
<i>LAND_USE</i>	= 1 if crashes occurred on commercial & residential area, = 0 otherwise
<i>LOCATION</i>	= 1 if crashes occurred at signalized intersection or roundabout, = 0 otherwise
<i>NIGHT</i>	= 1 if the crash occurred after sun set, = 0 during daylight
<i>OVERTURNED_CRASH</i>	= 1 if Vehicle upside down, = 0 otherwise
<i>PEDESTRIAN_CRASH</i>	= 1 if vehicle hit the pedestrian, = 0 otherwise
<i>REAR_END_CRASH</i>	= 1 if crash type is rear -end, = 0 otherwise
<i>OBJECT_COLLISION</i>	= 1 if vehicle hit an object (tree, sidewalk, street light), = 0 otherwise
<i>WEEKDAY</i>	= 1 if the crash occur during weekdays, = 0 crash occur during weekend (Friday & Saturday)
<i>MULTI_VEHICLE_CRASH</i>	= 1 if vehicle hit another vehicle, = 0 otherwise
<i>UAE_NATIONALITY</i>	= 1 if driver is UAE Nationals, = 0 otherwise

MODEL RESULTS AND DISCUSSIONS

Table (6) shows the results of the estimated parameters of the logistic regression model. The results indicate that eight variables are significantly changed for young drivers compared to others at significant of 95%. These variables are sudden lane change, driving too fast, location, when the crash occurred at night, crash type overturned & pedestrian, multi vehicle crashes and UAE nationality.

The significance of sudden lane change and overturned crashes variable can be justified due to driving too fast (which is significant variable) and the desire for speed and competition cars to satisfy the desire of superiority felt by young drivers. Among these variables, young drivers' crashes have higher probability to be occurred during night time on commercial and residential areas due to the passion of young to spend the night with friends in shopping centers and cafes. In addition, pedestrian crashes of young drivers increased by 1.73 times compared to other drivers. This finding reflects the inability of young drivers to take the right decision and right proper appreciation of the distances. Also, young drivers have more probability to be involved in multi-vehicle crashes than other drivers by about 1.57 times over. Moreover, it is remarkably that local young drivers' crashes increased by 4.44 times compared to other nationalities young drivers (Although few in number compared to other nationalities young drivers). This variable can be justified due to local's high income and their ability to own luxury high speed vehicles and UAE society is characterized as a small community.

The results also show that three variables are significant affect young drivers' crashes at a significant level of 90%. These variables are tailgating (not keep a safe distance), rear-end crashes, and object collision. These findings improve the reality of Inability of young drivers to take the right decision and right proper appreciation of the distances because of limited experience.

Table 6: The results of the regression model

Variables	B	S.E.	Wald	Sig.	Odds ratio
SUDDEN_LANE_CHANGE	.176	.065	7.396	.007*	1.192
SPEEDING	.374	.066	32.483	.000*	1.453
TAILGATING	.179	.096	3.501	.061**	1.196
LAND_USE	-.015	.047	.107	.744	.985
LOCATION	.117	.047	6.255	.012*	1.124
NIGHT	.227	.045	25.195	.000*	1.255
OVERTURNED_CRASH	.682	.221	9.498	.002*	1.977
PEDESTRIAN_CRASH	.547	.222	6.083	.014*	1.728
REAR_END_CRASH	.390	.226	2.969	.085**	1.477
OBJECT_COLLISION	.409	.225	3.306	.069**	1.505
WEEKDAY	-.040	.050	.640	.424	.961
MULTI_VEHICLE_CRASH	.451	.218	4.272	.039*	1.570
UAE_NATIONALITY	1.491	.045	1105.827	.000*	4.439
CONSTANT	-2.540	.221	131.955	.000	.079

* Significant at $\alpha = 0.95$ level

** Significant at $\alpha = 0.90$ level

CONCLUSION

The main objective of this paper is to investigate the contributing factors affecting young driver risk potential to be involved in a traffic crash. Relationships between the at-fault young drivers involved in traffic crashes and their demographic characteristics were investigated. Traffic crashes and violations data from AD Traffic Police database during six years (2010-2015) were employed in this study. The main findings of the data analysis and modelling results can be summarized as follows:

- About quarter of sever crashes were related to young driver. In addition, 26% of the fatality and 23% of the serious injuries were resulted because of young driver related crashes during the last 6 years (from 2010 till 2015).
- A significant increase (about 54.5%) in the young drivers fatality crashes rate was observed in the last six years. Which improve the theory of classify young drivers as a critical driver with regards to traffic safety.

- The total number of Face-to-Face traffic violations increased during the years 2010 to 2015 by 37%, while the total number of young drivers' Face-to-Face traffic violations increased by about 45%..
- Sudden lane change (19%), driving too fast (15%) were the main causes of young driver crashes. In general, non-significant differences exist between young drivers and others in terms of the causes of traffic crashes except a slight increment for those due to speeding.
- No significant differences in the percentage share values of the crash types over study period for most crash types except a significant increment in overturned crashes related to young drivers
- Pedestrian crashes of young drivers increased by 1.73 times compared to other drivers. This finding reflects the Inability of young drivers to take the right decision and right proper appreciation of the distances.
- Young drivers have 1.57 times more than other drivers in case of multi-vehicle crashes.
- It is remarkably that local young drivers' crashes increased by 4.44 times compared to other nationalities.

ACKNOWLEDGEMENTS

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Organosilane ‘Warm Compaction’ Technology for Green Roads

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Abstract:

New Organo-Silane ‘Warm Compaction’ technology, apart from other green benefits, enables dropping of mixing and compaction temperatures of Hot Mixes. Although, the concept of Warm Mix Asphalt (WMA) has been around for a while, the available WMA technologies only talk of drop in mixing temperature and not the compaction temperature, which the new Organo-Silane technology does. It is therefore prudent to call this technology, a ‘Warm Compaction’ technology.

In addition, this ‘Warm Compaction’ technology improves strength, fatigue resistance and water resistance, leading to potential extension of pavement life, contributing to sustainability.

This paper presents the summary of laboratory studies made at the IMT (Instituto Mexicano del Transporte), Mexico, and Braunschweig University, Germany to assess stiffness modulus, fatigue-resistance and water-resistance of this ‘Warm Compaction’ technology at different mixing and compaction temperatures for optimization.

1 GREEN ROADS

We have taken this earth on lease from our future generations. It is our duty to return it to them in liveable condition if not better. Striving for sustainability through prudent use of limiting natural resources and restraint in emissions is the call of the day.

As it is said, ‘Nations don’t build roads. Roads build a nation’. It is very important to develop road infrastructure at a brisk pace, if we want to attain / sustain high economic growth. At the same time, it is equally important to ensure that such infrastructure development is sustainable. Prudent use of limiting natural resources therefore, becomes our responsibility now, more than ever before.

There may be many ways of bringing in sustainability in building roads. Here are some of the ways in which Zydex Nanotechnology contributes to sustainability in road construction :

- **Reducing the usage** of limiting resources like bitumen, aggregates etc.
- Extending the life cycles of the roads, so as to **defer the demand** for such resources
- Reducing fuel consumption and low temperature operation leading to **reduction in emission**

This paper discusses the ‘Low Temperature Operation’ aspect of the Zydex Nanotechnology in particular.

2 CONVENTIONAL WARM MIX ASPHALT AND ITS LIMITATIONS

The concept of Warm Mix Asphalt (WMA) has been around since the 90’s. It was developed in Europe in response to EEC countries signing the 1997 Kyoto Treaty to reduce greenhouse gases.

A warm asphalt mix process (WAM) has been developed in Europe and was reported by Harrison and Christodoulaki at the First International Conference of Asphalt Pavements in Sydney, 2000. A more complete report was given by Koenders et al. at the Eurobitume congress in 2000. [1] Their paper describes an innovative warm mixture 3 process that was tested in the laboratory and evaluated in large-scale field trials (in Norway, the UK and the Netherlands) with particular reference to the production and laying of dense graded wearing courses. [2] Their work resulted in the development of WAM Foam, Warm Asphalt Mix with foamed bitumen. [3] At the Eurobitume congress in 2004, Barthel et al. introduced the use of a synthetic zeolite additive to produce warm mix asphalt. The zeolite creates a foaming effect that results in a higher workability of the mix. [4] Warm mixes have received some attention in Europe and Australia since around 2000. The pavement industry in North America started to give warm mixes some interest a few years later and in June 2005 the National Center for

Asphalt Technology (NCAT) published two reports about the use of Sasobit, a synthetic wax, and Aspha-min, a synthetic zeolite, in warm mix asphalt. [5,6].

Lower mixing temperature in case of WMA, is certainly a huge advantage, but the technology does have certain challenges as follows.

- Lower temperatures used for WMA can result in incomplete drying of the aggregates and the resulting trapped water in the coated aggregates may cause moisture damage.
- Finding the right balance between lowering the production temperatures, applying anti-stripping agents and achieving a sufficiently moisture resistant asphalt mixture might be a challenge when using WMA

This is probably the reason behind WMA technology not catching up all over the world at the pace initially anticipated.

3 ORGANOSILANE ‘WARM COMPACTION’ TECHNOLOGY

The available WMA technologies only talk of drop in mixing temperature and not the compaction temperature. The new Organosilane technology now allows reduction in both, mixing as well as compaction temperature.

In addition the new Organosilane technology also improves coating efficiency, leading to faster and complete coating of even the fines. This is because the Organosilane chemistry reduces the surface tension of bitumen for faster and better wetting.

The workability of the Organosilane mixes is also observed to be better and the compaction easier.

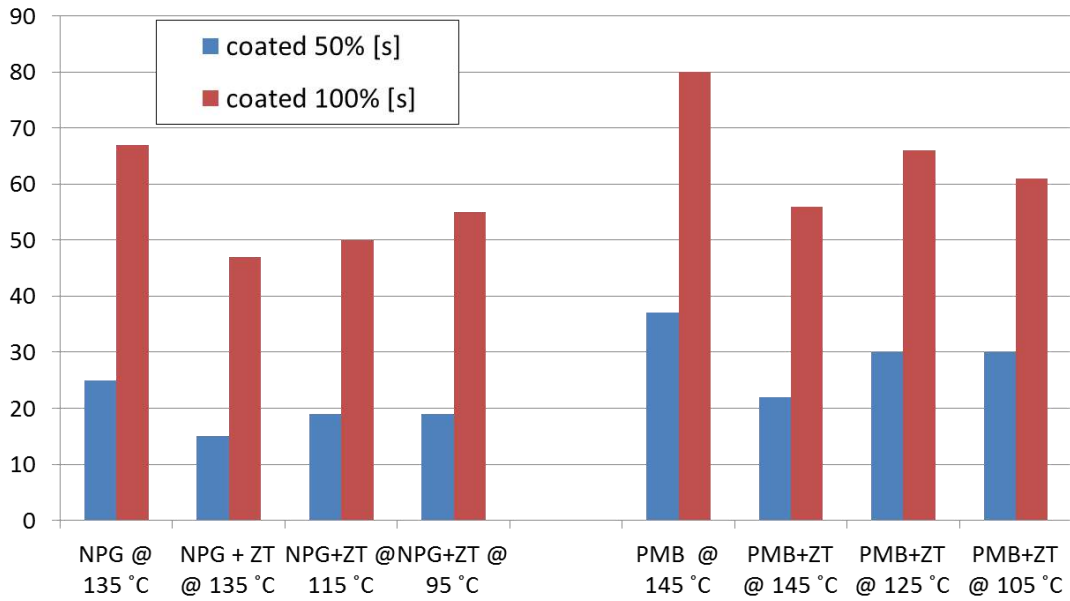
The low temperature mixing, low temperature compaction and the other benefits mentioned above are verified by laboratory studies.

4 CASE STUDY - TRANSFER CENTRE FOR THE ROAD SECTOR (TSW), BRAUNSCHWEIG UNIVERSITY, GERMANY

Table 1 : Compositions of the type of asphalt applied

Asphalt		AC 16 B S	AC 11 D S
Bitumen		50/70	25/55-55 A
Bindemittelgehalt	M.-%	4,3	6,0
Anteil ZycoTherm [®] , bezogen auf den Bindemittelgehalt	M.-%	0,10	0,15
Gesteine		Gabbro	Gabbro
> 16,0 mm	M.-%	2,8	-
11,2 - 16,0 mm	M.-%	28,5	0,5
8,0 - 11,2 mm	M.-%	12,1	20,2
5,6 - 8,0 mm	M.-%	12,8	10,8
2,0 - 5,6 mm	M.-%	15,2	23,3
0,063 - 2,0 mm	M.-%	22,0	37,6
< 0,063 mm	M.-%	6,6	7,6

Graph 1
Coating Efficiency of ZycoTherm at different mixing temperatures



Graph 2
Effect of ZycoTherm on Power / Energy Consumption while mixing

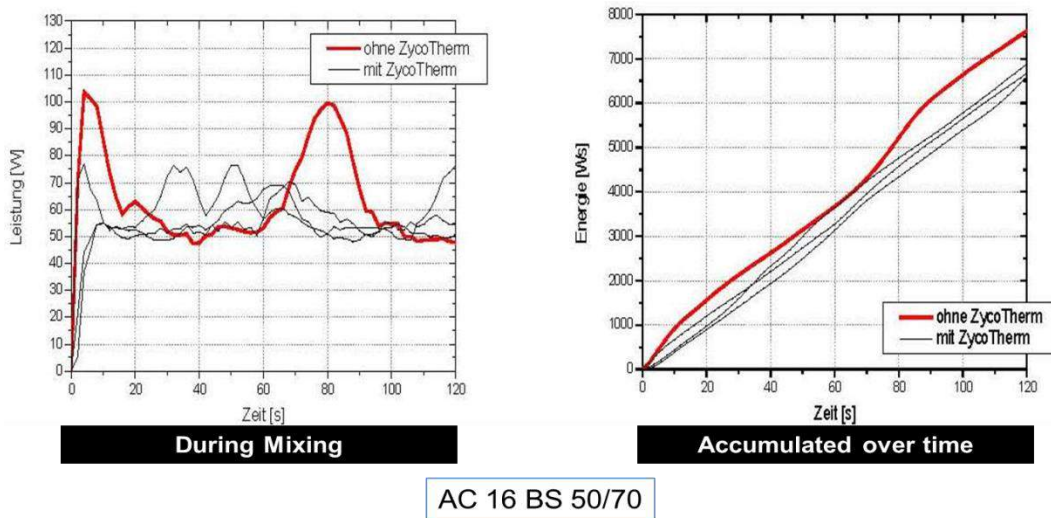


Table 2
Effect of ZycoTherm on Compaction Resistance and Voids

Mix Type	Binder Type	C / ZT	Compaction Temp °C	50 % Coating Sec	Compaction Resistance 21Nm	Void Content Vol %
AC 16 BS	50/70	C	135	25	41.6	7.0
		ZT	135	15	43.5	7.1
			115	19	42.8	7.2
			95	19	41.3	6.9
AC 11 DS	25/55-55 A	C	145	37	37.3	4.3
		ZT	145	22	34.2	4.6
			125	30	36.5	5.0
			105	30	36.5	5.3

Innovationsgesellschaft Technische Universität Braunschweig mbH



Summary

- Goal of the examinations carried out here was to document the influence of the binding agent ZycoTherm® on the mixing process and to document the compression properties of asphalt.
- For this purpose, two types of asphalt that are common in Germany, an asphalt binding agent AC 16 B S with 50/70 and an asphalt concrete for asphalt surface layers AC 11 D S with 25/55-55 A, were mixed with and without ZycoTherm® and compressed at different temperatures. An overview of the variants produced is specified in table 5-1.
- During the mixing processes, the power consumption of the laboratory mixer was recorded and the time that the degrees of coating of 50 %, 75 %, 90 % and 100 % were reached was noted. The times for reaching the degree of coating of 50 % are specified in table 5-1. Therefore, the mixed material is coated quicker when ZycoTherm® is added. On average, 20 % less time is required. With regard to the mixing performance, for AC 16 B S a tendency was recognised that less performance was required when adding ZycoTherm®.
- The compression resistance does not show any difference for both asphalt types when adding ZycoTherm®, as documented in table 5-1.
- The raw density of the asphalt mixture produced can be considered as equal for both types of asphalt.
- The densities by volume of the roller compressed asphalt test plates only indicate small differences. The void content is specified as outcoming result in table 5-1.

Table 3
Overview for producing the asphalt variants

<i>Variant</i>	<i>Mix type</i>	<i>Binder type</i>	<i>Addition of ZycoTherm®</i>	<i>Compaction Temp.</i>	<i>50% Coating achieved</i>	<i>Compaction resistance</i>	<i>Void content</i>
-	-	-	-	^o C	s	21 Nm	Vol.-%
1a	AC 16 B S	50/70	No	135	25 s	41.6	7.0
1b			Yes	135	15 s	43.5	7.1
1c			Yes	115	19 s	42.8	7.2
1d			Yes	95	19 s	41.3	6.9
2a	AC 11 D S	25/55-55 A	No	145	37 s	37.3	4.3
2b			Yes	145	22 s	34.2	4.6
2c			Yes	125	30 s	36.5	5.0
2d			Yes	105	30 s	36.5	5.3

- For AC 16 B S, no tendency could be specified with regard to the void content. The void contents are within a narrow range so that this can be considered as equal.
- The void contents for AC 11 D S increase with reducing compression temperature. This increase however, is not within the testing accuracy so that here, only one tendency can be specified, there is no statistical security. Deviations within these ranges can still be considered as close to practice.
- As a summary, it can be stated that when adding ZycoTherm® you can reduce the time for coating the stones and with a significantly reduced compression temperature, you can achieve an almost consistent void content.

5 CASE STUDY – INSTITUTO MEXICANO DEL TRANSPORTE (IMT)

Parameters for testing

Asphalt binder : AC 20 (PEMEX, Mexico) with ZycoTherm: 0.075%; reference OBC 5.5% and with ZycoTherm OBC 5.8%; Aggregates: La Canada Quarry

Sample size: 50mm H X 63 mm W X 380 mm L

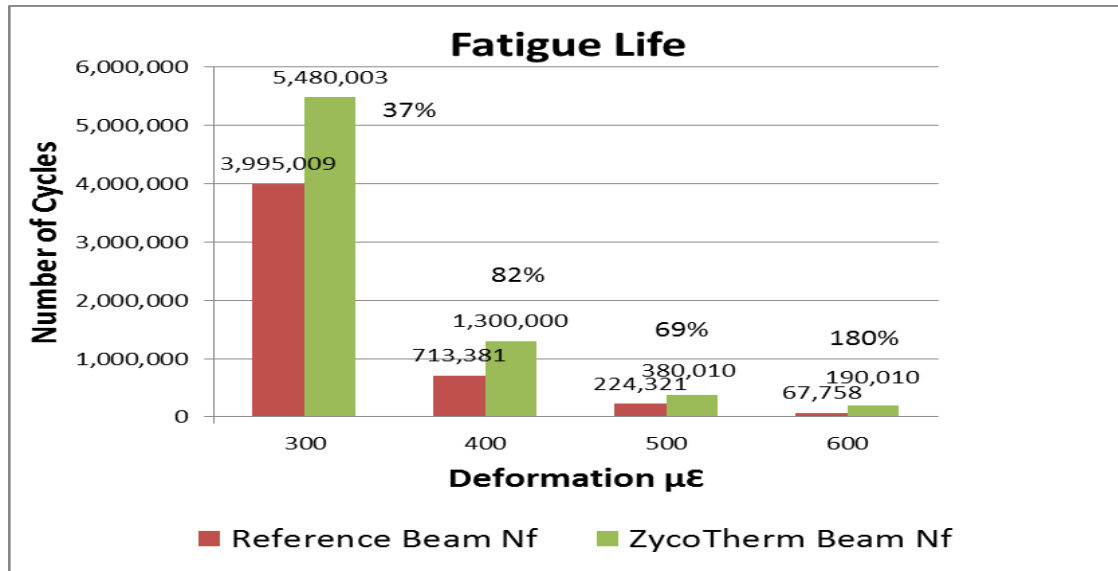
Air voids : 6%

Test temperature : 20 °C

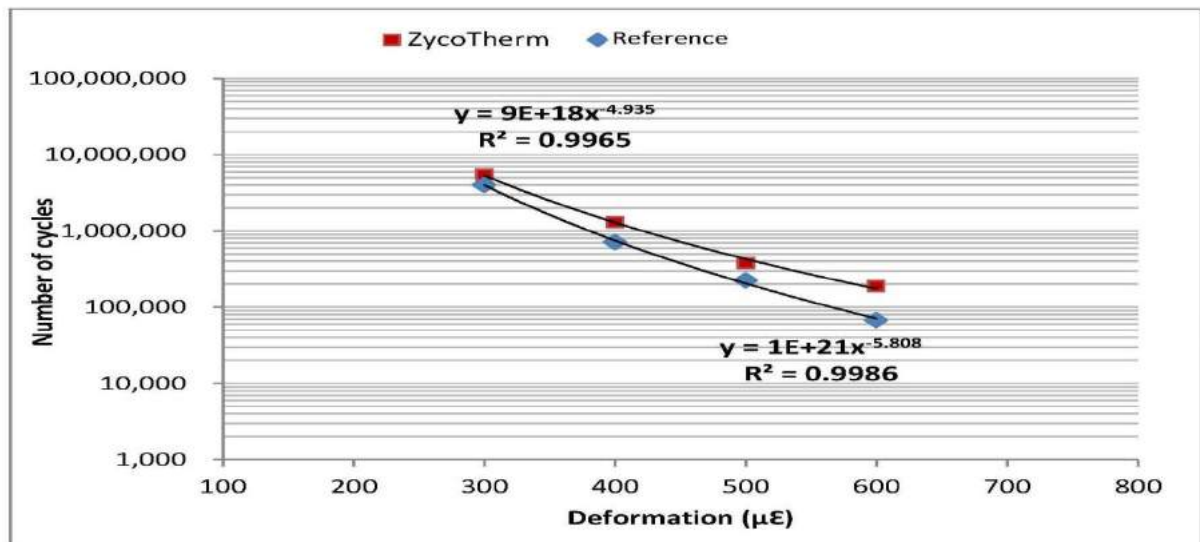
Deformation Frequency : 10 Hz for four levels - 300, 400, 500 & 600 µε (represents pavement deformation under normal transit condition)

Asphalt mix fatigue life failure criteria: Number of cycles in which the beam stiffness modulus decreases 50% compared to its initial value

Graph – 3
Higher Number of Cycles means Higher Fatigue Life

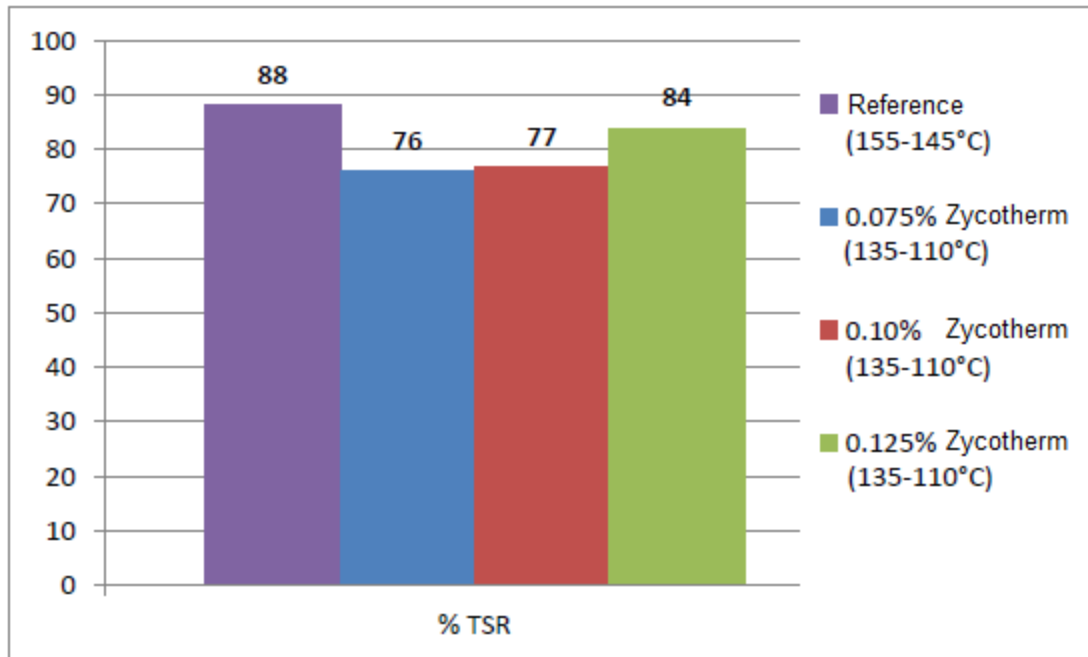


Graph - 4
Wohler curve (fatigue law)



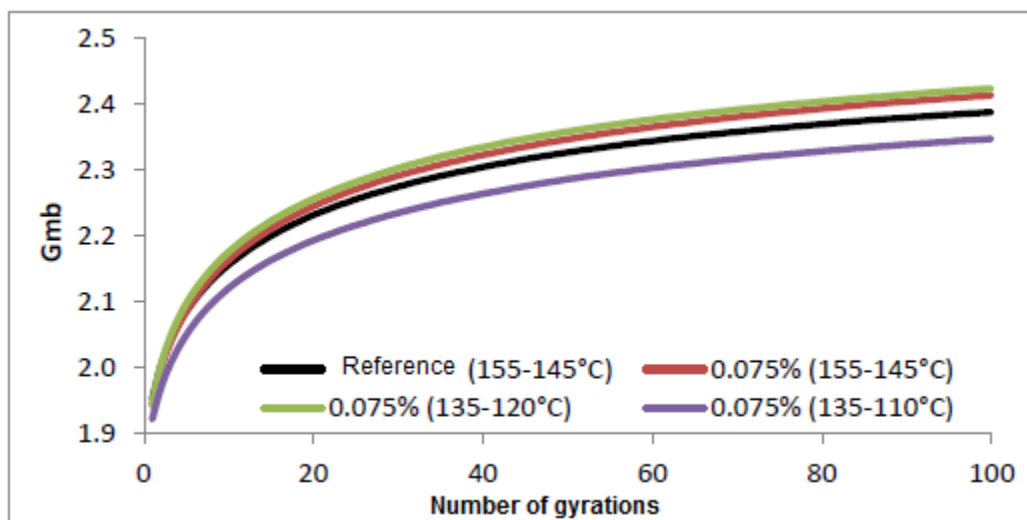
- For all levels of deformation (300, 400, 500 and 600 µε), samples with optimum content of Zycotherm (.075%) presented extended life for fatigue, so the incorporation of Zycotherm significantly favors the asphalt mix.
- The amount of load repetitions recorded for the Zycotherm samples were increased by 37 to 180% depending on the level of deformation.
- It was also observed that Zycotherm addition to the asphalt mix modifies the slope of the fatigue law, which means that Zycotherm samples are less susceptible to deformation levels, compared with the reference samples.

Graph – 5
Effect of ZycoTherm on TSR

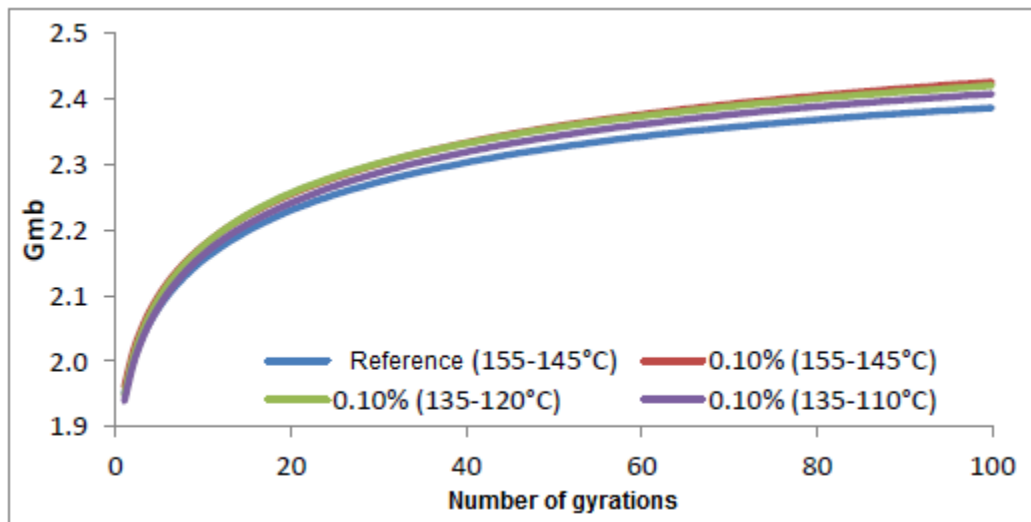


- It was observed that with the addition of Zycotherm in the highest dosage (0,125%) the TSR specification was achieved ($\geq 80\%$), even when mixing and compaction temperatures were reduced to 135° and 110°.
- Asphalt mixes with 0,075% and 0,1% presented slightly inferior to 80% TSR values. The conditioned specimens with additive presented 2 to 3 saturation cycles to achieve saturation levels between (70-80%).
- For all missing dosages, it can be estimated that TSR results will be acceptable for temperature reduction.
- It was observed that for mixing 135°C and compaction 110°C temperatures, TSR increased when the Zycotherm dosage was increased as well.

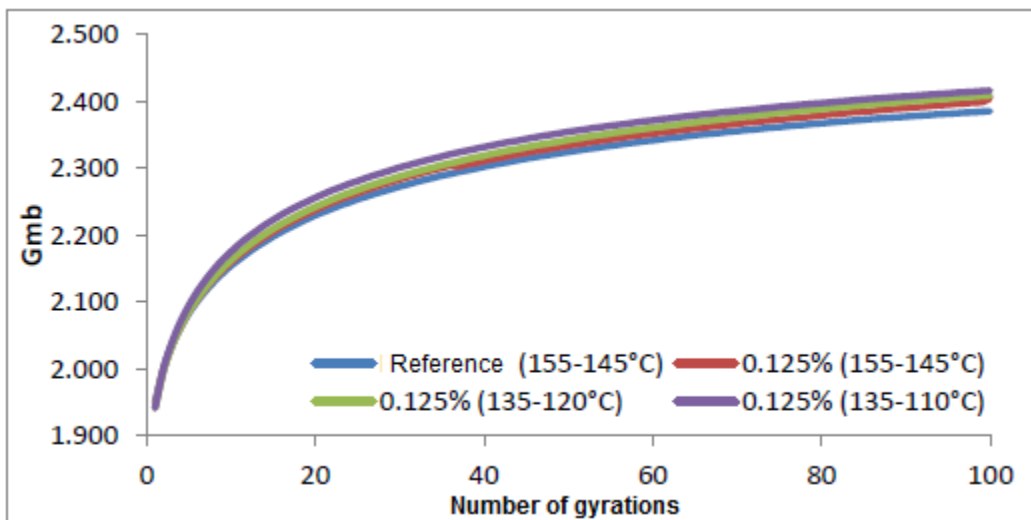
Graph – 6
Effect of ZycoTherm (@0.075%) on compaction density



Graph – 7
Effect of ZycoTherm (@0.1%) on compaction density

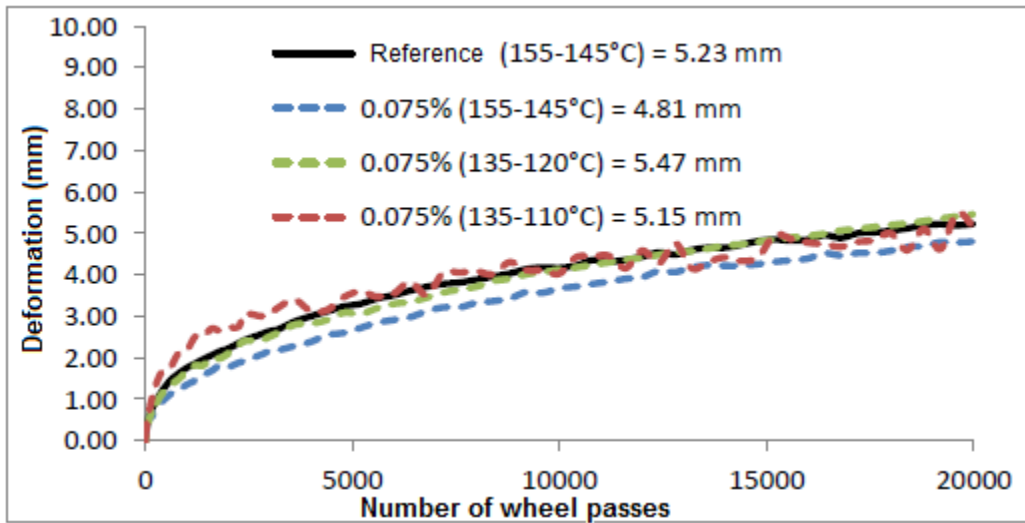


Graph – 8
Effect of ZycoTherm (@0.125%) on compaction density



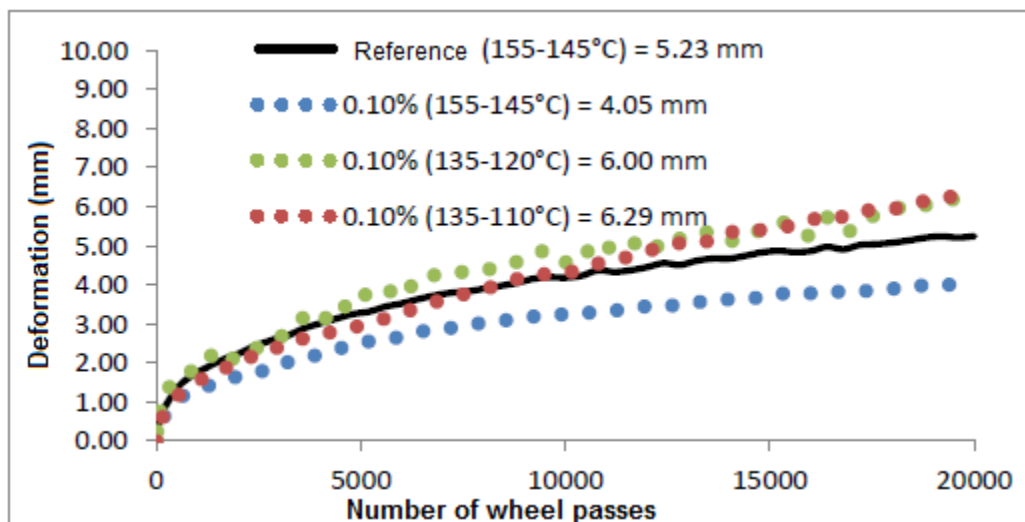
- When Zycotherm is used a greater compaction can be achieved for the same and lower temperatures than the reference mix.
- If mixing and compaction temperatures are reduced benefits in time can be obtained; more time for hauling and waiting before compaction is possible with the Zycotherm addition.
- Fuel savings during the mixing process and less carbon footprint benefits can be obtained as well.

Graph – 9
Wheel Tracking performance for ZT @ 0.075%



- When the 0,075% additive dosage is incorporated into the asphalt mix, it is observed that deformation is maintained with acceptable tolerance, for similar temperatures as the reference mix.
- When mixing and compaction temperatures are reduced, deformation is still within high transit levels, according to AMAAC specifications.
- There is not affectation in rutting behavior when lowering mixing and compaction temperatures.

Graph – 10
Wheel Tracking performance at ZT @ 0.1 %



- The asphalt mix with Zycotherm present a superior behavior when produced and compacted at the same temperatures as the reference mix.
- When adding 0,1% of Zycotherm to the mix the mixing temperature can be lowered in 20°C and compaction temperature to 35°C, deformation is slightly affected, but it is still whitin the high transit levels and this results may be between the test dispersion.
- If mixing and compaction temperatures are reduced benefits in time can be obtained; more time for hauling and waiting before compaction is possible with the Zycotherm addition.

6 CONCLUSION

In summary, addition of Zycotherm, the Organosilane additive to bitumen

1. Allows the mixing temperature to be lower
2. Gives equivalent or better compaction at lower compaction temperatures.
3. Improves coating efficiency, leading to faster and complete coating.
4. Reduces mixing effort and saves energy.
5. Results in acceptable levels of TSR at lower mixing temperatures.
6. Gives excellent rut resistance at lower mixing and compaction temperatures.

Overall, the Organosilane technology, in addition to lower mixing temperatures, allows lower compaction temperature and gives all the benefits stated above.

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October 16-20, 2016 – Kuala Lumpur, Malaysia

PAPER TITLE	Pragmatic Road Hazard Assessment with the aid of new technology		
TRACK			
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KEYWORDS:

Road Hazard Assessments, Road Safety, Journey Management

ABSTRACT:

Many Oil and Gas producers and service companies have already achieved substantial and sustained improvements in road safety performance over the last decade, evident in the typical lagging indicator: Motor vehicle crash frequency rate, often published in Corporate Sustainability reports. The single highest risk factor in commercial land transport remains: the Environment in which operations exist where 1.25 million people continue to die on the world's roads each year (World Health Organisation Global Status Report on Road Safety, 2013).

The traditional method of assessing the environmental risk is by performing a Road Hazard Assessment (RHA) to determine the hazards and risk rating by individual route, this approach is cumbersome and dated. Every time a new route is established in the operation, the current best practice methodology would require a specialist to travel the new route and populate a hazard register with all of the recognizable hazards and the associated risk rating. This method is impractical and therefore rarely achieved in a timely manner.

In the search for a pragmatic solution, an alternative methodology to fixed route Road Hazard Assessment is explored in light of new Journey Management technologies available to industry. The technology enables crowdsourcing of hazard information, desktop validation, and real-time analysis of route risk. The benefits and challenges in adopting this technology in the Oil & Gas community will be assessed in both isolation and in conjunction with Journey Management.

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Introduction

Many Oil & Gas producers and service companies have already achieved substantial and sustained improvements in road safety performance over the last decade, evident in the typical lagging indicator: Motor vehicle crash frequency rate, published in public Corporate Sustainability reports. Different measures are used by different organisations in terms of classification (*e.g.* kms/miles, serious/all incidents, work/private/all), however, despite the inconsistency in measurement, a trend of improvement appears across the board as show in Figure 1 below.

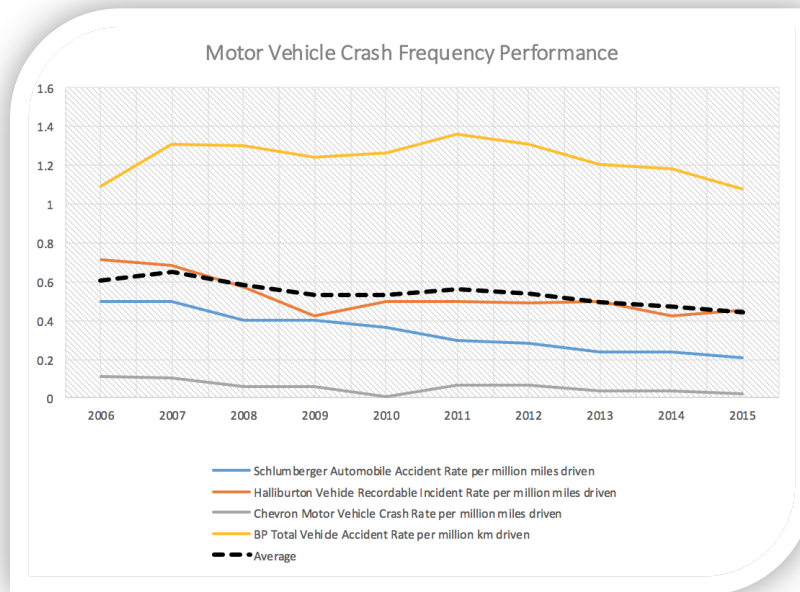


Figure 1: Motor Vehicle Crash Frequency Rate trends 2006-2015.

The single highest risk factor in commercial land transport remains: the environment in which operations exist where 1.25 million people continue to die on the world's roads each year (World Health Organisation Global Status Report on Road Safety, 2015).

Oil & gas operators and service companies attempt to quantify the risk from the road environment by performing Road Hazard Assessments (RHA) to determine the hazards and produce a risk rating by individual route, this approach is cumbersome and dated. Every time a new route is established in the operation, the current best practice methodology would require a specialist to travel the new route and populate a hazard register with all of the recognizable hazards and the associated risk rating. This method is impractical and therefore rarely achieved in a timely manner.

Journey Management (JM) is the holistic management of vehicle movements through a process of compliance audit, risk rating of hazards, and route planning. This process provides more holistic planning and risk assessment of vehicle movements and greater assurance of journeys completed successfully otherwise offering early intervention to reduce loss. With advances in technology, JM has evolved and opened up opportunities to evolve the traditional RHA methodology.

The technology enables crowdsourcing of hazard information, desktop validation by an independent party, and real-time analysis of route risk. The benefits and challenges in adopting this technology in the Oil & Gas community will be assessed both in isolation and in conjunction with Journey Management.

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Traditional Road Hazard Assessment

RHA seeks to quantify the risk exposure from road hazards on the routes used by the organisation. There are different views on the process and method of risk quantification however a commonly used reference is the OGP365-1 Land Transport Safety Recommended Practice - Road Hazard Assessment Guidance Note 1 (OGP, 2003). The guidance includes hazard classification, risk ranking classification, and escalating factors (Figure 2); Essentially the framework for risk assessment, similar to the Job Safety Analysis (JSA), a tool for qualitative analysis of a job procedure or practice (Rausand, 2005). The process requires subjective analysis of hazards to determine the risk presented to the operation based on the existing road safety controls adopted.

Summary of Road Hazard Identification and Assessment (OGP365-1)		
Hazard Type Table		
Driving surface, width, and integrity	Potential collision objects	Road design/intersections
Narrow bridge/tunnel...	Trees/rocks	Corners – Blind/Sharp/Banked
High Crowning	Encroaching road banks	Intersection – Blind/Sharp
Excessive loose gravel	Narrow bridge/bridge rails...	Railway crossings...
Paved or oiled surface breaking up	Height restrictions...	Crest of hill...
Washboard	Avalanche risk...	Steep hill...
Potholes/ruts	Passing other vehicles – Poor visibility...	Risk of grounding [out]...
Shoulder – Sharp drop off/Washout/Soft/Narrow	Meeting other vehicles...	Inadequate or absence of appropriate signage
Ditch conditions – Deep/Obstacle strewn/Fluid filled	Animals	Line of sight obstruction...
Ice roads/ice bridges	Pedestrians	Dust
Risk Ranking		
Low Risk	Medium Risk	High Risk
Exposure managed by application of existing policy, procedure or practice	Exposure managed by enhancement of existing policy, procedure, or practice and/or requires use of utility devices	Exposure mitigated through intervention by party or parties other than driver
Exposure management within control of driver	Exposure management remains within control of the driver	Control must be in place for journey to proceed
	Control must be in place for journey to proceed	
Escalating Factors		
Fog or Smoke	Road surface liable to deteriorate rapidly when wet	
Poor drainage/frequent mud	Low illumination	
Snow and ice	Driver experience	
Shadowed areas...	Fatigue/low alertness	
Crosswinds/blowing snow	Conflicting priorities (e.g. crew change)	
Weight restrictions (road/bridges)	3 rd parties conflict/animals (traffic conditions)	
Sun(e.g. low angle in winter)	Rain – Visibility, pooling on roads	
Winds		

Figure 2 Summary of OGP365-1 Land transportation safety recommended practice guidance note 1.

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Road Hazard Assessment in Isolation

During the RHA process, a Hazard Register is populated for a given route containing all of the identified road hazards, location, risk rating, cell phone signal strength, and comments. In isolation, this document forms the basis for driver briefing before they travel the route to inform the driver of the expected hazards. This document is normally contained within the company intranet and/or in hard copy within the transport department.

In practice, maintaining a current and comprehensive depository of RHA Hazard Registers for all common routes travelled by staff and/or contractors is resource intensive requiring specialists to complete and consequently does not occur consistently. Additionally, variances in the number of identified hazards and risk rating vary greatly from organisation to organisation. This results in inconsistency and can even occur in multiple organisations even when they operate within the same geographical area.

Road Hazard Assessment Integration with Journey Management

The OGP365-2 Land transportation safety recommended practice – Guidance note 2 – Journey Management provides a recommended example for journey assessment and approval. This method categorises the journey in to low, medium, or high risk according to a points system assessing the risk factors, as seen in Figure 3 below.

Summary of OGP365-2 Journey Management Risk Assessment		
Low risk = <=14 points		Med risk = 15-24 points
		High risk = >=25 points
A	Security escort requirements	0 points for no escort requirement scaling to 20 points for dual manned patrol
B	Security situation	0 points for no known problems scaling to 20 points for known possible problems
C	Number of vehicles & passengers	1 point for 2+ vehicles with 1+ passengers per vehicle scaling to 6 points for single heavy vehicle with no passenger
D	Distance from base	Low points for <50km scaling to 11 points for more than 200km
E	Road conditions	<ul style="list-style-type: none"> - Paved Road = 1 point - Mixed (less than 50% paved) = 2 points - Unpaved = 4 points - Mountain = 8 points
F	Day/night driving	0 points for driving between 6am & 7pm & <35km scaling to 20 points for driving between 7pm & 6am > 20km
G	Weather	1 point for Dry weather scaling to 8 points for fog/dust
H	Communication	0 points for mobile/satellite phone/radio scaling to 4 points for no comms, single vehicle
I	Driver hours on duty	0 points for driver slept > 8 hours in last 24hrs scaling to 8 points for under 16hrs on-duty by end of planned journey
J	Driving contractors usage	0 points for using permanent contract vehicle and driver with inspection and training approved by company, scaling to 20 points for using contract vehicle without inspection by company and driver without training by company

Figure 3: Summary of OGP365-1 Land transportation safety recommended practice guidance note 1.

The road conditions contribute to the risk score for the total journey risk but only account for the road type in 4 categories; Paved, Mixed, Unpaved, and Mountain. The work carried out in the Road Hazard Assessment is largely ignored and the known risk does not convey across to the total journey risk score.

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The evolution of commercially available journey management systems are now catering for the known risk presented from road hazards and factoring in to the total journey risk score. This is achieved by categorising journey risk in to 3 distinct categories; Environment, Asset and People (Figure 4).

Category	Risk score
+ Environment	High
+ Asset	Low
+ People	High

Figure 4: Risk categories

This enables journey stakeholders to rapidly identify where the risk is highest. A further example is taking the high risk environment rating and expanding the details to reveal the causes (Figure 5).

Category	Risk score
- Environment	High

Non-Compliance	Assessment outcome	Risk score
6		
Road Hazards	There are road hazards on this journey	High
Security issues	There are no security issues/risks on this journey	Low
Night driving	The assessment result is within the required limits	Low
Weather	The weather conditions are GOOD for this journey.	Low
GSM or satellite coverage	YES	Low
Road conditions	Good & blacktop	Low

Figure 5: Environment rating details

The method of risk categorisation enables simple adjustments to the journey plan to be made during the planning process (e.g. changing the route slightly to avoid high risk road hazards), reducing the risk profile of the overall journey.

Risk Assessment per Route

Adopting the current recommended practice for RHA provides a hazard register per pre-determined route. The process requires for all common routes to be routinely surveyed to update the hazard register. Inefficiencies can and do exist where routes crossover each other or new routes are assessed completely. The concept of Road Hazard Assessment and the evaluation of environmental risk within the JM technology significantly enhances this process. Road Hazards no longer need to be tied to a fixed route, instead they are recorded as a geographical location with the required information. Commercially available JM solutions enable the journey route to be created in real-time utilising a routing engine to increase efficiency of route selection. When the route is created, the JM plan is able to 'collect' all the road hazards on the roads which the route passes through and form the environmental risk assessment based on the collected hazards. This enables a multitude of routes to be used and the known hazards to immediately impact the risk rating.

The next challenge with adopting the OGP365-1 framework or similar for the RHA process is the risk ranking classification is provided per road hazard and there is no guidance on overall route road hazard risk classification.

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Oil & Gas operators/service companies that adopt the OGP365 Land Transport Safety Recommended Practice or a derivative of the same, should have sufficient procedures or practices that enable the driver to control the vehicle and navigate road hazards without resulting in a motor vehicle crash. This assumption is based on:

- Driver capability
 - Driving license issued from relative government authority
 - Defensive Driver Training certificate
 - Fit-for-work
- Vehicle reliability
 - Fit-for purpose
 - Inspected prior to journey

Therefore, the assumption is that as long as there are no physical constraints (*e.g.* Bridges/tunnels) that would prevent the vehicle from reaching the destination without interference, and no escalating factors present (*e.g.* Fog/Snow), then the resulting risk ranking for all the identified hazards will be Low.

In absence of a documented industry best practice, there are different perspectives used across the Oil & Gas industry for calculating overall route hazard risk, examples are:

1. The highest risk rating for all road hazards in the route will determine the overall risk rating for the 'environmental' risk for the route
2. Same as above, plus the combination of many low or medium risk road hazards compound to result in a higher risk rating for the route

The first example appears rational on the basis that where a high risk road hazard is identified then the risk for the journey will be high due to the high risk road hazard encounter being imminent. What the perspective ignores is the potential for existing policies, procedures or practices to break down and expose higher risk.

The second example also appears rational, allowing for multiple road hazards with low or medium risk to compound in to a higher risk rating given that a series of road hazards introduces more exposure than just a single road hazard. This perspective relies on a repeatable formula for risk assessment typically using a calculation of the number and risk rating of each road hazard. The reliance on the number of identified road hazards alone introduces potential for error where many low risk road hazards may be the result of a much more comprehensive RHA exercise as opposed to a less comprehensive effort. The ability to create repeatable RHA's with the same volume of hazards identified is influenced by the hazard perception abilities of the individual or team conducting the assessment. For this reason, there is an increased reliance on the assessor/s that conduct the RHA.

In both the methods the robustness of the controls adopted according to policies, procedures or practices are not duly tested. This could lead to individual process failures compounding the risk of the road hazard. An example may be if Defensive Driver Training (DDT) is a mandatory requirement during which the driver is trained specifically on navigating crests, the risk presented by a driver not being trained on approach to a crest will make the risk of a crest (An identified hazard) potentially higher than Low. The risk rating of Low is therefore reliant on the DDT control being robust in terms of content, delivery, and controls around ensuring drivers actually successfully completed DDT.

When a new route is going to be used by the organisation, a suitably qualified individual or team is required to travel the route and conduct the RHA. The resource/s required to be able to adequately identify hazards is not always readily available and it is not uncommon for organisations to deem the unknown as highest risk, therefore a route without a completed RHA is deemed as high risk. When Journey Management is required, the risk for the journey will in this case increase to High requiring a senior stakeholder to accept the risk for each journey to start using the new route.

Road Hazard Assessor Profile

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There appears to be no documented industry best practice for determining the skills and experience required for an individual to conduct a RHA. This leads organisations to use different criteria for selecting candidates and typically the resource performs the RHA's as a part time role, if not the task is outsourced. The hazard identification process requires observation and assessment of the road area to identify hazards although the current OGP365-1 risk classification process ignores the complexity of roadside furniture. An example may be that frangible light poles may line the street and considered low risk due to the design of the poles, however due to the installation method used locally, the frangible poles may not deform correctly in the event of a collision and act more like a tree. The risk may be considered low initially, however a trained eye may identify the higher risk due to the incorrect installation.

So how do we define the skills and experience required to sufficiently perform the RHA process? A pragmatic approach is required to provide feasible solution that will ensure that sufficient knowledge and experience is available to identify potentially high risk hazards without burdening operations with requirements that are not practical to implement. Borowsky *et al.* (2010) find that older experienced drivers were able to better detect cues and detect more potentially hazardous situations than novice drivers. Additionally, there are several road safety engineering qualifications that focus on the safety aspect of road infrastructure and furniture design and placement. The challenge is the feasibility of having older experienced drivers with road safety engineering qualifications available to assess the routes used by operations on a frequent basis.

The 'crowdsourcing' concept offers an opportunity to innovate. Surrogate Road Hazard Assessors may be sourced from driver trainers, HSE professionals, and other company resources that frequently travel the routes. The concept of enabling other internal resources to capture potential road hazards supported by photographic evidence provides an opportunity to gain more frequent updates. The potential issue is that due to the subjectivity of hazard perception, each hazard reported would require validation from a suitably qualified Road Hazard Assessor. The technology available today allows surrogate Road Hazard Assessors to capture the detail of perceived hazards using just a smart phone and uploading to a central database, ready for review by a Road Hazard Professional. Once the hazard has been vetted, it can be added to a live Road Hazard Register.

When there are multiple organisations operating in the same geographical area, there is now an opportunity to share Road Hazard Registers online between organisations to ensure that each other organisation benefits from each other's contributions. This enables contractor A to record a new road hazard and the operator and contractor's B, C, and D to all benefit from the information.

In order to share the Road Hazard Registers between organisations, there must be a common language and common risk classification system. While a unified global standard may not be possible, certainly a common standard for the organisations operating within the same geographical location would be of benefit.

In the context of Journey Management, with the available technology commercially available today, the use of crowdsourced road hazard information updates will enable planned and even departed journeys to be reassessed in terms of risk based upon new hazard information received regarding road hazards on the route selected. This can transform the risk assessment from an environmental risk perspective.

Summary

The safety performance of Oil & Gas land transport operations has improved on average and this is supported by the trends from the abovementioned operators and service companies. The improvements are diminishing and performance stagnating. Now is the time to identify improvements through advances in technology since the original recommended practices were produced.

The recommended practice for Road Hazard Assessment is based on assessing the fixed routes and classifying each road hazard with a risk rating to produce a hazard register per route. The recommended practice ignores the risk profile of the hazards per route, instead utilising different criterion in journey management planning to risk profile the environmental risk.

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There are no specific recommendations regarding the skills and experience of the person who should conduct a road hazard assessment and in practice, the frequency at which road hazard assessments are performed and/or the consistency of the information captured varies. Care needs to be taken to ensure sufficient quality without the impracticalities of deploying road safety engineers on a full time basis in to sites to constantly monitor for road hazards. With parallel works on hazard perception being driven by computer-based hazard perception technology, we can learn that with experience, drivers become more attuned to cues that suggest a hazard.

Advances in technology can now enable any person with a smart phone to capture new road hazards and report to a central remote database. Suitably qualified resources may then validate the road hazard and report in to a live common road hazard database that may be shared across organisations within a geographical location. This creates a crowdsourcing model of road hazard data collection with quality control from a centralised resource validating road hazard data for many different geographical locations.

More up-to-date road hazard register databases enable a step change in the journey management process, no longer utilising just distance and terrain inputs to quantify environmental risk. There is still a requirement to utilise escalating factors in the JM plan as the presence will escalate the risk of the route.

Recommendations

Organisations should assess the current processes used for evaluating environmental risk from a journey management perspective. Commercially available technology can enable crowdsourcing of road hazard data by less qualified but more readily available resources. The quality control mechanism can be centralised and when implemented will provide more up to date and accurate hazard register database.

Organisations overlapping in geographical area should work together to define a localised standard for assessing risk profiling logic per road hazard, per route, and per journey. This logic should account for the robustness of the controls adopted to mitigate risk by using go/no-go compliances, all of which can be applied in commercially available systems to enable crowdsourcing of road hazards and use of the same information in the journey management plans, resulting in a uniform language of journey risk.

If adopted, there are efficiencies to be created.

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Application of appropriate repair materials for asphalt pavement in Japan

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1. INTRODUCTION

The transportation infrastructure in Japan was developed in a large part during the high economic growth period from 1950s to 1970s, and has contributed greatly to the growth of Japan's economy. The transportation infrastructure constructed at that time has passed more than about fifty years, and decrepit roads need to be renovated. Therefore, there is growing interest in appropriate, efficient and budget-conscious maintenance and repair methods. For road pavement, demand for new construction has been on the decline since the 1980s, opening an era of maintenance of existing pavements.

The maintenance repair methods of pavement involve a technique to construct new pavement or to achieve life prolongation of existing pavements. The latter approach can lead to substantial cost savings compared to rebuilding, if implemented at the correct time using appropriate repair materials.

We have developed a number of pavement repair materials for different types of distress, which are easy to use and highly durable, with the aim of efficient maintenance of paved roads. These materials are already in widespread use and have accumulated a track record. This paper describes the features of these materials and provides examples of their application.

2. CLASSIFICATION OF THE DISTRESS IN ASPHALT PAVEMENT

Distress to asphalt pavements mainly consists of cracking, rutting, and loss of flatness. Other forms of distress are level differences, potholes, and road surface roughness caused by raveling. The causes of such distress include repeated loading by traffic, the durability of the materials involved, design issues, construction work, and the effect of environmental conditions such as rain, UV, and so on. It is thought that these factors influence each other (Japan Road Association 2006a, b and 2013).

Distress to pavements can be broadly divided into structural distress and road surface distress. Structural distress is addressed by all replaceable or other large-scale projects undertaken from a long-term perspective, with consideration given to future traffic volume. Road surface distress, on the other hand, is typically addressed by emergency repairs to ensure the safety of traffic. Figure 1 describes the evolution of distress to pavements.

Road surfaces in service are subjected to forms of distress such as the raveling, aggregate scattering, and cracking. Under the influence of continual traffic loads, these distress proceed deeper into the pavement and escalate to structural distress. Therefore, repairing a pavement at an early stage before road surface distress becomes structural distress can be an effective means of extending the life of existing pavement. It should be noted, however, that because there are so many different types of distress, this approach may not be successful unless an appropriate material is chosen and put in place at the correct time.

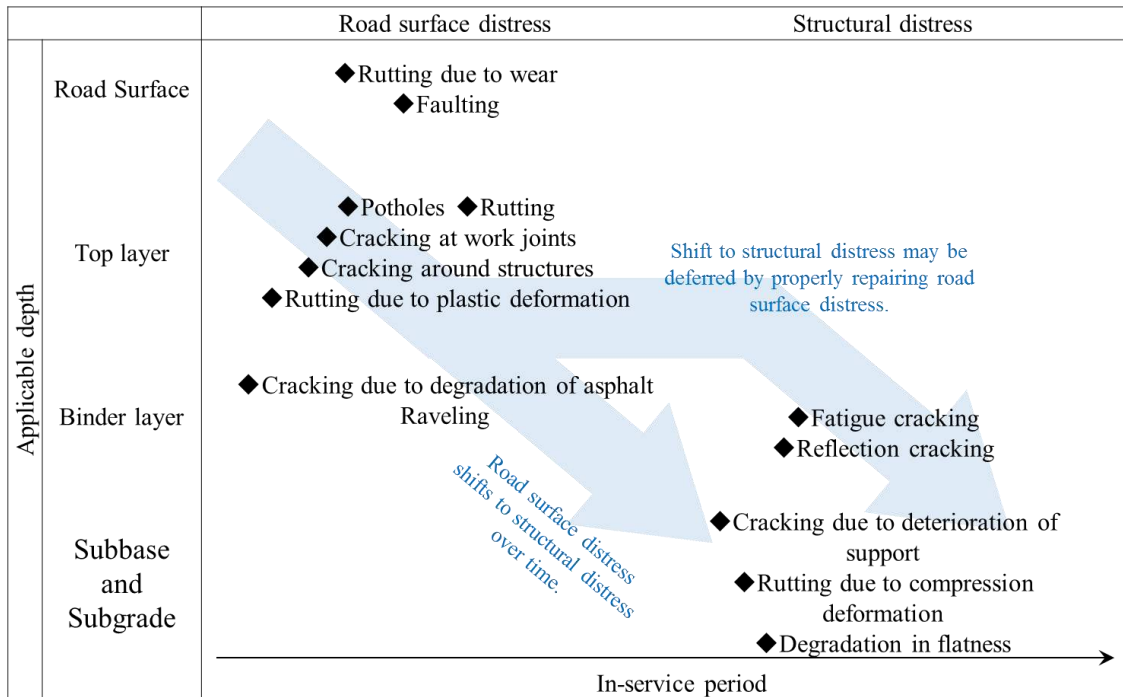


Figure 1. Shift from road surface distress to structural distress (Japan Road Association 2006a, b and 2013)

3. REPAIR MATERIALS

The repair material and method need to be appropriately selected based on the type and degree of distress. Table 1 lists several repair materials which has developed in recent years to deal with the most common forms of distress. All of these materials can be used at ambient temperature, so there is no need for heating or temperature control. They also have highly portability and workability for making quick repairs. The following subsections provide an outline of these materials, and describe their composition, properties and application method.

Table 1. Major repair materials

Distress	Repair material
Cracking	Cold-mix crack injection material
	Slurry-type material
Potholes	Repair material for pothole
Faulting	Resin-type road surface repair material

3.1 Repair materials for cracks

If the pavement suffers cracking or has furrows due to poor connections at joints, rainwater will infiltrate the pavement, regardless of the size of the occurring distress. If it is not repaired, thus accelerating the change to structural distress. Figure 2 shows different pavement cracking patterns. The conventional way of repairing cracks is to pour a hot injection material such as blown asphalt into them, but this method requires a dissolving pot and other equipment since the hot injection materials have to make a high temperature to melt. Furthermore, it is difficult to completely fill very thin cracks with widths of 10 mm or less, or deep cracks. Rainwater can enter the pavement through such half-filled cracks, and in the end, no substantial life extension benefit may be gained. Additionally, cracking can occur not only in linear patterns but also in complicated shapes, as in the case of alligator cracks. In such a situation, the repair work needs to be performed over a

planar area. We therefore developed both an injection-type and a paste-type material suitable for different types of cracks. (Japan Road Association 2013)

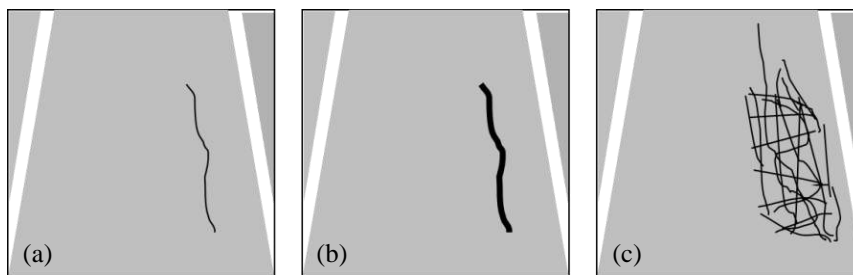


Figure 2. Schematic diagrams of cracking patterns: (a) thin linear crack, (b) thick linear crack, and (c) alligator cracks

3.1.1 Cold-mix crack injection material

(1) Appearance and application

The cold-mix crack injection material which we developed is a low-viscosity liquid at ambient temperature and hence has excellent penetrability characteristics. Figure 3 shows the appearance of the material and application method. The product is composed of the main material contained in a plastic bottle, and a curing agent in a separate small glass bottle. The curing agent is added to the main material bottle and shaken for approximately ten seconds. The contents are then poured into the crack to be filled, as shown in Fig. 3(b). Traffic can be opened within an hour after the injection. Cracks of any width can be filled because the width of the injection nozzle can be adjusted by cutting it at the appropriate point. This newly-developed crack injection material is applicable not only to cracks in asphalt pavements, but also to cracks and furrows in concrete pavements.

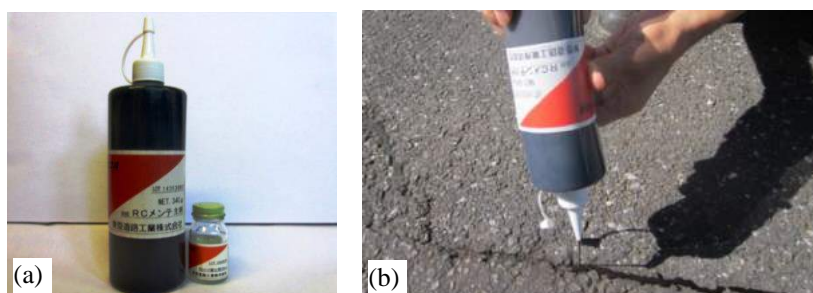


Figure 3. Crack injection material: (a) appearance (b) application method

(2) Composition and properties

Table 2 shows the composition of the cold-mix crack injection material.

Table 2. Composition of cold-mix crack injection material

Component	Content
Rubber modified asphalt emulsion	95%
Curing agent	5%

(3) Improved workability

In addition to the bottled crack injection material shown in Fig. 3, we also provide a road crack maintenance injection (RCMI) method that uses the equipment. This method involves filling cracks with the injection material using an injector that simultaneously discharges the main material and the curing agent at a fixed ratio. Figure 4 shows an example of such an injector and the RCMI method in action. The use of this

equipment allows the material to be poured continuously, thereby minimizing material loss and the time required for repair work; this is expected to contribute to work efficiency.

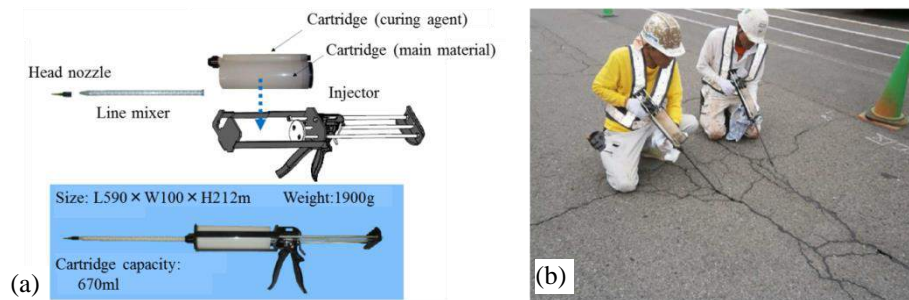


Figure 4. (a) Example of injector and (b) product application method

3.1.2 Slurry-type material

The cold-mix crack injection material introduced in the preceding subsection is poured into one crack after another, thus ensuring that each crack is completely filled and repaired. For alligator cracking, however, this process need a long time. In this case, a planar approach is more time-efficient than filling individual cracks. Accordingly, we developed a slurry-type material that can be applied to alligator cracks by injecting it over a planar area.

(1) Application method

Figure 5 shows the application method. It does not require a special, dedicated tool, and can be easily applied using only a hand mixer and a rubber rake. With this method, a wide area can be repaired within a short period of time. Figure 6 shows the appearance of a road surface after applying this method. This slurry-type material can also be applied to linear cracks using a rubber rake.



Figure 5. Application method

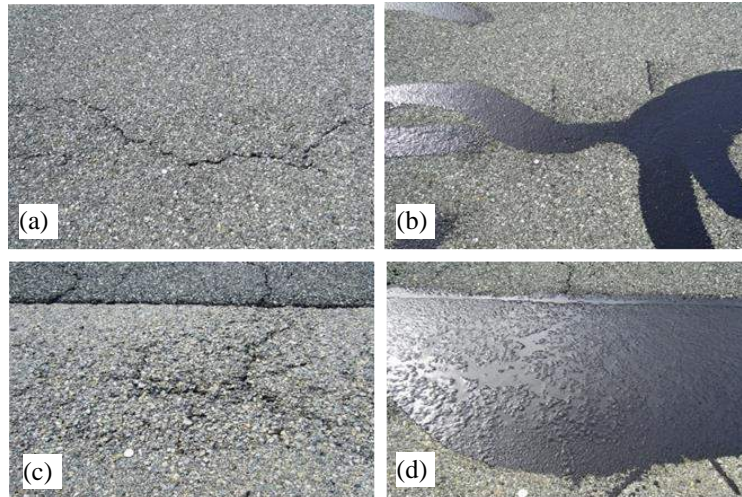


Figure 6. Application of the slurry-type material to linear and alligator cracks: (a) Linear cracks–before application, (b) Linear cracks–after application, (c) Alligator cracks–before application, and (d) Alligator cracks–after application

(2) Composition and properties

Table 3 shows the composition of the slurry-type repair material. The aggregate and the SBR modified asphalt emulsion are mixed immediately before using it. The material has high anti-abrasion performance after hardening.

Table 3. Composition of slurry-type material

Component	Content
Graded aggregate and curing agent	75%
SBR modified asphalt emulsion	25%

To evaluate the degree of penetrability of the material into cracks, we conducted a penetrability test in compliance with Handbook of Pavement Investigation and Examination Method (Japan Road Association 2007). A slit was formed using two glass sheets and was filled with the repair material. The slit was 100-mm long, 200-mm deep, and 3-mm wide. Figure 7 shows the penetrability test results. The slurry-type repair material is seen to have significantly better penetration than a typical hot injection material, allowing the entire length and width of a crack to be filled.

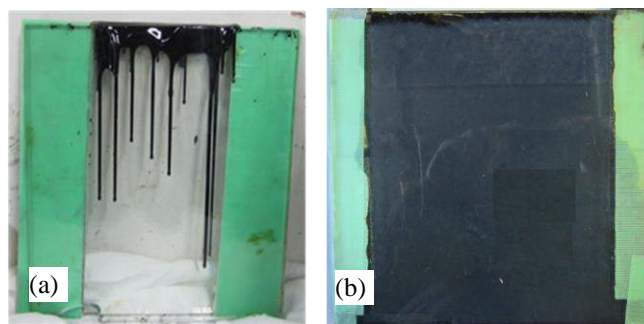


Figure 7. Penetration test results: (a) Typical hot injection material and (b) Slurry-type material

(3) Application to small-scale repair projects

We also developed a packaged version of the slurry-type material for small-scale repair projects. The uniqueness of this product lies in the vinyl package that contains an aggregate combined with a curing agent and SBS modified asphalt emulsion that acts as the binder. The contents weigh two kilograms. The

appearance of the product is shown in Fig. 8. The aggregate and the modified asphalt emulsion are packaged together in one bag, but separated by an internal partition.



Figure 8. The packaged slurry-type material

Figure 9 shows the work procedure for the packaged slurry-type material. The bag is held with the aggregate section up, and downward pressure is applied so that the partition breaks and the aggregate and curing agent enter the emulsion side. The components are mixed by kneading the bag well for approximately thirty seconds. The mixture is then ready for use; it should be spread immediately using a trowel, and allowed to cure for one to two hours before traffic opening. By having the two different materials in one package, this product saves the time and labor involved in asphalt mixture preparation, and allows uniform production and application without soiling the surroundings.

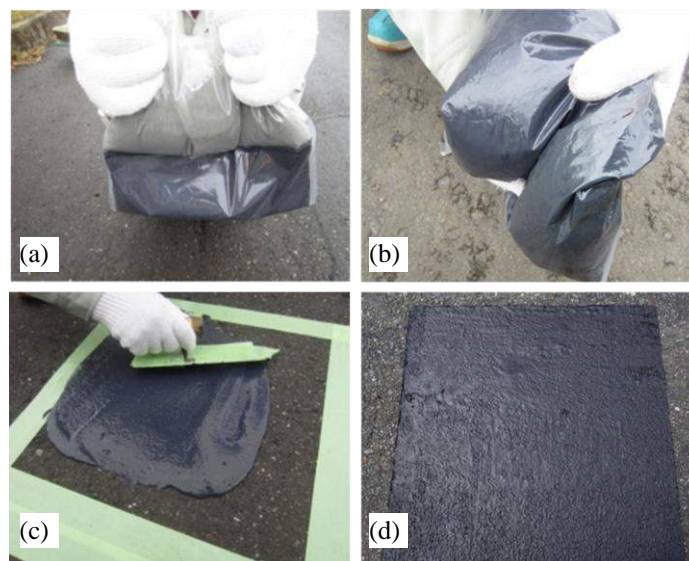


Figure 9. Work procedure for packaged slurry-type material: (a) Breaking the partition, (b) Mixing, (c) Spreading, and (d) Final appearance

3.2 Repair material for potholes

Potholes cause not only discomfort for drivers but also lead to traffic accidents. Thus, they need to be repaired immediately once created. This subsection discusses a pothole repair material that we developed, which has high portability and durability. (Japan Road Association 2013)

(1) Appearance and application

Figure 10 shows the appearance of the pothole repair material. This product is a cold-mix asphalt mixture. The solvent uses oil derived from a plant. One bag contains twenty kilograms of the mixture. Since the maximum size is five millimeters, it can be applied to potholes with various shapes. For this reason, the

product is also suitable for repairing narrow or complex areas, such as the vicinity of structures, in addition to level differences and potholes.

Figure 11 shows the work procedure for the pothole repair material. Before starting the application procedure, dust and gravel should be removed from the surface to ensure that the material adheres well to the base. When the package is opened, the material immediately begins to harden; therefore, it should be used as soon as possible. After opening the package, the pothole repair material for patching is spread over the affected area to a thickness of one to five centimeters, and the surface is leveled using a shovel or the like. A tamper or a plate compactor is then used to complete the project. Although traffic can be immediately released, the durability of the product gradually increases as the solvent vaporizes, and reaches a sufficiently high level in about 24 hours. This product requires only simple compaction, so the road can be used immediately following the completion of the repair work. The mixture is compacted by a traffic vehicle, and the strength of the mixture increases.



Figure 10. Appearance of repair material for pothole



Figure 11. Work procedure for pothole repair material: (a) Cleaning, (b) Unpacking and spreading/leveling, (c) Compacting, (d) Completion, and (e) Traffic opening

(2) Composition and properties

Table 4 shows the composition of the pothole repair material. This is a cutback asphalt mixture using plant-derived oil. It is significantly more durable than conventional repair materials because the specially modified material is contained.

Table 4. Composition of pothole repair material

Component	Content
Aggregate	95%
Modified asphalt, and cutback material	5%

We conducted a Marshall stability test at ambient temperature in order to evaluate the change in the strength of the material during the curing period. The material was cured at 20°C, and the Marshall stability was measured immediately after the production and one day, three days and seven days later. The results are compared in Fig. 12 to those for conventional mixture. The stability of our pothole repair material increases over time, particularly during the first day. In addition, this highly durable, pothole repair material develops its strength more quickly and exhibits a higher strength than conventional mixture.

We also conducted a Cantabro test in order to evaluate the resistance to aggregate scattering (Japan Road Association 2007). The test was carried out after curing the product at 5°C for 24 hours, and the results are shown in Fig. 13, together with those for conventional mixture. The Cantabro loss for our pothole repair material is seen to be much smaller than that for conventional mixture, indicating that it exhibits excellent resistance to aggregate scattering soon after curing.

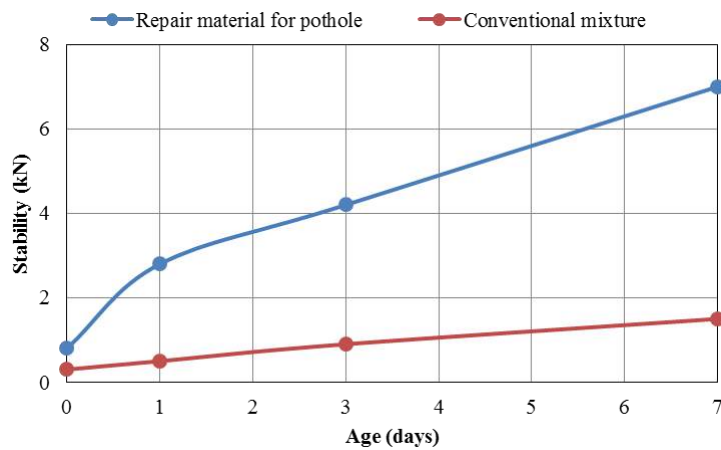


Figure 12. Change in Marshall Stability at ambient temperature with time

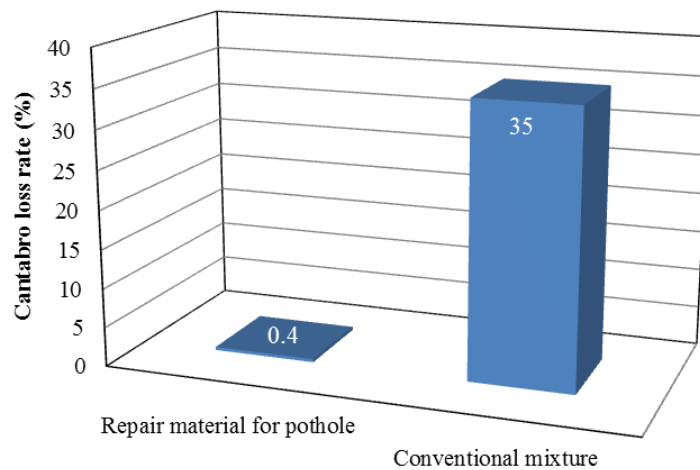


Figure 13. Cantabro test results

3.3 Resin-type road surface repair material

Figure 14 shows examples of level differences; these can be caused by differential settlement with different pavement structure or construction defect at a joint between an existing pavement and a newly-built pavement. Level differences also cause discomfort for drivers, as well as vibration, noise, and even damage to cargo carried by vehicles. Thus, any level difference occurring on the road surface requires immediately repairing.

Materials for repairing level differences must have high adhesiveness to existing pavement so as not to be easily raveled due to the passing of vehicles. This subsection discusses a resin-type road surface repair material that we developed for repairing of level differences. (Japan Road Association 2013)

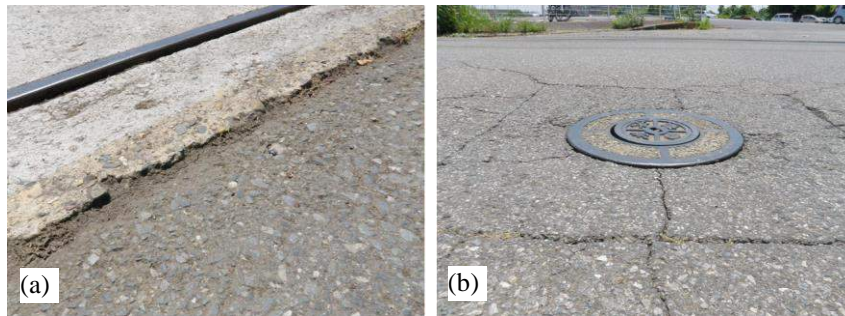


Figure 14. Occurrence of level differences: (a) concrete structure and (b) Utility hole

(1) Appearance and application

The resin-type road surface repair material shown in Fig. 15 is a mixture of an aggregate and a special acrylic resin, and can be applied using a trowel. It consists of a resin, a curing agent, an aggregate and anti-slip sand, all of which come in a single cardboard box. Figure 16 shows the application method for this product, and Fig. 17 shows working appearance. First, the curing agent is added to the aggregate bag and the two are well mixed. The resin is then added, and the resulting mixture is well blended. The material is poured over the affected area and the surface is leveled using a trowel. The process is finished by spreading the anti-slip sand.

Because this resin-type road surface repair material uses sand as an aggregate, it is applicable not only to rehabilitation for level differences at bridge joints or around utility holes but other kinds of small-scale repair projects, including patching very small potholes.



Figure 15. Appearance of resin-type road surface repair material



Figure 16. Application method for resin-type road surface repair material: (a) Adding the curing agent to the aggregate, (b) Mixing, (c) Adding the resin, (d) Mixing, (e) Spreading and leveling, and (f) Spreading anti-slip sand

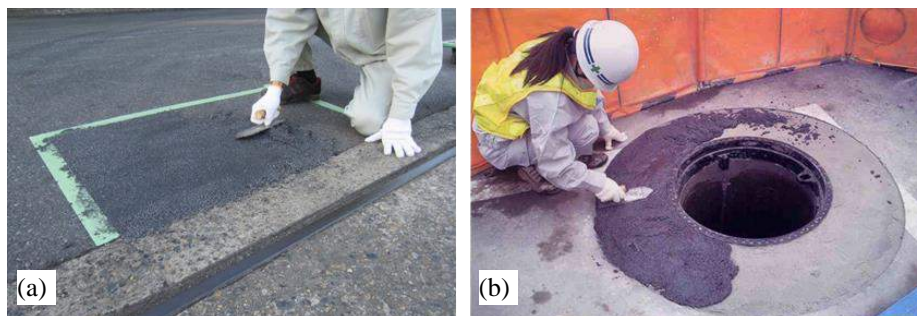


Figure 17. Application appearance: (a) level difference at a constructed joint and (b) around a utility hole

(2) Composition and properties

Tables 5 and 6 show the composition and properties of the resin-type road surface repair material, respectively. The figures in the properties table represent the results of tests conducted in compliance with Handbook of Pavement Investigation and Examination Method (Japan Road Association 2007). The resin-type road surface repair material has significantly higher marshall stability and dynamic stability than asphalt mixtures, because it uses a special acrylic resin as the binder, instead of asphalt. Moreover, it exhibits excellent adhesiveness to the base and follows the expansion and contraction of the pavement. It also quickly hardens so that traffic can be released about twenty minutes after the repair work.

Table 5. Composition of resin-type road surface repair material

Component	Content
Aggregate	60%
Resin and curing agent	30%
Anti-slip sand	10%

Table 6. Properties of resin-type road surface repair material

Items	Properties
Curing time (25°C)	5-10 min
Density	1.866 g/cm ³
Stability (60°C)	17.9 kN
Residual stability (60°C)	88.8%
Dynamic stability (60°C)	31,500 times/mm
Bending strain (-10°C)	6.14×10 ⁻³
Raveling test (-10°C)	1.09 cm ²

4. APPLICATION

Table 7 summarizes all the products described in this paper. For each of the repair materials, ○ means “recommended”, × means “difficult”, and — means “not applicable”.

Table 7. Recommended applications

Repair material	Percentage of cracking			Height of level difference		Size of pothole		Curing time
	Less than 10%	10-20%	More than 20%	Less than 5 mm	5-50 mm	25-50 mm	More than 50 mm	
Cold-mix crack injection material	○	○	×	-	-	-	-	10 min
Slurry-type material	○	○	○	○	-	-	-	30 min
Repair material for pothole	-	-	-	×	○	○	○	Immediately
Resin-type road surface repair material	○	○	○	○	○	○	×	30 min

○ means “recommended”, × means “difficult”, and — means “not applicable”.

5 CONCLUSIONS

This section summarizes the characteristics of each repair materials.

- The cold-mix crack injection material is a repair material to be injected into cracks or furrows. Due to its excellent penetrability characteristics, it can fill the entire length and width of any crack regardless of outside air temperature. It can also be applied using the equipment-based RCMI method, which allows continuous crack repair.
- The slurry-type material has high penetrability characteristics and completely covers pavement cracks, providing an excellent water cut-off effect. It is a cold repair mixture that is applicable not only to linear cracks but also to alligator cracks. It is particularly effective for road surface refreshing and for repairing areas with high cracking rates. As the binder and aggregate are packaged in one bag, it allows easy and efficient small-scale repairs.
- The repair material for pothole is a cutback cold-mix asphalt. It is applied in simple steps: opening the bag, spreading and leveling, and simple compaction. Traffic can be released immediately after its application. Compared with other typical materials, this product exhibits high durability, plastic deformation resistance, and waterproofness. It can be applied regardless of weather conditions or the season of the year. It is used mainly for repairing potholes and smoothing level differences but can also be applied to narrow areas around structures.

- The resin-type road surface repair material is a cold-mix repair material with a special acrylic resin acting as the binder. This composition exhibits excellent resistance to weather, raveling, and chemicals, and strong adhesiveness to the existing base. It hardens extremely quickly so that traffic can be released about twenty minutes after curing.

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PAPER TITLE	ISO 39001:2012 – Road traffic safety (RTS) management systems – Requirements with guidance for use		
TRACK	Road Safety		
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Include up to 5 keywords

- Road safety
- Management system standard
- International Organization for Standardization (ISO)
- Integration (with other management system standards)
- Safety Performance Factors (SPF)

ABSTRACT:

ISO committee TC 241 – Road traffic safety (RTS) management systems has developed the new management system standard for road traffic safety – ISO 39001:2012. ISO 39001 is an important and integral part of the work of the United Nations Road Safety Collaboration/UNRSC and Decade of Action for Road Safety 2011-2020. ISO/TC 241 currently has over 45 member countries and 20 international liaison organizations, including the World Bank and the World Health Organization. Reference will be made to parts of ISO 39001 – for information, clarification, purpose and application. Examples of good practice of ISO 39001 will be presented. ISO 39001 is designed to work on its own, parallel or integrated with other ISO MSS, management system standards, in any size or type of private or public organization, to build a structured and effective Road Traffic Safety/RTS system to save lives.

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ISO 39001:2012 specifies requirements for a road traffic safety (RTS) management system to enable an organization that interacts with the road traffic system to reduce death and serious injuries related to road traffic crashes which it can influence. The requirements in ISO 39001:2012 include development and implementation of an appropriate RTS policy, development of RTS objectives and action plans, which take into account legal and other requirements to which the organization subscribes, and information about elements and criteria related to RTS that the organization identifies as those which it can control and those which it can influence.

Road traffic safety (RTS) is a global concern. It is estimated that around 1,3 million people are killed and 20 million to 50 million are injured on roads around the world each year, and that this level is rising[10]. The socio-economic and health impacts are substantial.

The international standard ISO 39001 provides a tool to help organizations reduce, and ultimately eliminate, the incidence and risk of death and serious injury related to road traffic crashes. This focus can result in a more cost-effective use of the road traffic system.

ISO 39001 identifies elements of good RTS management practice that will enable the organization to achieve its desired RTS results.

ISO 39001 is applicable to public and private organizations that interact with the road traffic system. It can be used by internal and external parties, including certification bodies, to assess the organization's ability to meet the requirements.

Experience from around the world has shown that large reductions in death and serious injury can be achieved through the adoption of a holistic Safe System approach to RTS. This involves a clear and unequivocal focus on RTS results and evidence-based actions, supported by appropriate organizational management capacity.

Government cannot achieve these reductions alone. Organizations of all types and sizes, as well as individual road users, have a role to play. By adopting this International Standard, organizations should be able to achieve RTS results at levels that exceed what can be achieved through compliance with laws and standards, and their own objectives, and, at the same time, contribute to the achievement of societal goals.

The management system specified in this International Standard focuses the organization on its RTS objectives and RTS targets and guides the planning of activities that will realize these goals by using a Safe System approach to RTS. Annex B describes categories of RTS results, the Safe System approach and a framework for good practice RTS management, and shows how they can be aligned with this International Standard.

Annex A provides some guidance on the implementation of this International Standard.

The RTS management system can be integrated into, or made compatible with, other management systems (see also Annex C) and processes within the organization.

ISO 39001 promotes the use of an iterative (plan, do, check, act) process approach that will guide the organization towards delivery of the RTS results.

The international standard ISO 39001 specifies requirements for a road traffic safety (RTS) management system to enable an organization that interacts with the road traffic system to reduce death and serious injuries related to road traffic crashes which it can influence. The requirements in ISO 39001 include development and implementation of an appropriate RTS policy, development of RTS objectives and action plans, which take into account legal and other requirements to which the organization subscribes, and information about elements and criteria related to RTS that the organization identifies as those which it can control and those which it can influence.

The international standard ISO 39001 is applicable to any organization, regardless of type, size and product or service provided, that wishes to

- improve RTS performance,
- establish, implement, maintain and improve an RTS management system,
- assure itself of conformity with its stated RTS policy, and
- demonstrate conformity with this International Standard.

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ISO 39001 is intended to address RTS management. It is not intended to specify the technical and quality requirements of transportation products and services (e.g. roads, traffic signs/lights, automobiles, trams, cargo and passenger transportation services, rescue and emergency services).

It is not the intent of ISO 39001 to imply uniformity in the structure of RTS management systems or uniformity of documentation.

RTS is a shared responsibility. This International Standard is not intended to exclude road users from their obligations to comply with the law and behave responsibly. It can support the organization in its efforts to encourage road users to comply with the law.

All requirements of this International Standard are generic.

Where any requirement of this International Standard cannot be applied due to the nature of an organization and its products or services, that requirement can be considered for exclusion, provided the exclusion and the reason for exclusion are documented.

Where exclusions are made, claims of conformity to this International Standard are only acceptable where these exclusions do not affect the organization's ability to establish, implement, maintain and improve an RTS management system successfully.

Introduction

Nearly 1.3 million people in the world die each year of road crashes – more than 3000 deaths each day. Another twenty to fifty million people suffer non-fatal injuries, which are also an important cause of disability.

Ninety per cent of road traffic deaths occur in low- and middle-income countries, which claim less than half the world's registered vehicle fleet. Road traffic injuries are also the leading cause of death for people from 10 to 24 years of age. Significant numbers of road traffic fatalities and injuries can be prevented by addressing the leading causes, which include excess speed, lack of seat-belt and child restraint use, drinking and driving, lack of helmet use by riders on two-wheel and three-wheel motorized vehicles, poorly designed and inadequately maintained roads, unsafe infrastructure and vehicles, and inadequate trauma care.

Unless immediate and effective action is taken, road deaths are predicted to become the fifth leading cause of death in the world by 2030, resulting in an estimated 2.4 million fatalities per year. This is, in part, a result of rapid increases in motorization without sufficient improvement in road safety strategies and land use planning. The economic consequences of motor vehicle crashes have been estimated between 1% and 3% of the respective GNP of the world countries, reaching a total of over USD 500 billion. Reducing road casualties and serious injuries will reduce suffering, unlock growth and free resources for more productive use.

These are facts provided by the United Nations and the World Health Organization.

Decade of Action for Road Safety and United Nations Road Safety Collaboration

The UN General Assembly resolution 64/255

(http://www.who.int/violence_injury_prevention/publications/road_traffic/UN_GA_resolution-54-255-en.pdf) of March 2010 proclaimed 2011-2020 the Decade of Action for Road Safety, with a global goal of stabilizing and then reducing the forecasted level of global road fatalities by increasing activities conducted at national, regional and global levels.

Resolution 64/255, requested the World Health Organization and the United Nations regional commissions, in cooperation with the United Nations Road Safety Collaboration (UNRSC) and other stakeholders, to prepare a Plan of Action for the Decade as a guiding document to support the implementation of its objectives.

UN Secretary-General, Mr Ban Ki-moon, has said: "I call on Member States, international agencies, civil society organizations, businesses and community leaders to ensure that the Decade leads to real improvements. As a step in this direction, governments should release their national plans for the Decade when it is launched globally on 11 May 2011."

The UNRSC was established in 2004 to support efforts to address the global road safety crises. The Collaboration is chaired by the World Health Organization and has brought together more than 50 international organizations, governments, nongovernmental organizations, foundations and private sector entities to coordinate effective responses to road safety issues.

The Decade of Action for Road Safety offers a common road map for reaching the ambitious goal of reducing road-traffic fatalities globally by 50 per cent. The success will only be measured by its achievements. The target is to save 5 million lives by 2020.

The implementation plan is guided by the following five pillars:

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- 1) Road-safety management;
- 2) Safer roads and mobility;
- 3) Safer vehicles;
- 4) Safer road users;
- 5) Post-crash response.

Visit the official WHO, UNRSC and Decade websites:

All WHO data is online, see <http://apps.who.int/ghodata/> and go to Injuries and violence on the left hand column. These data are also published in the first Global Status report; see <http://apps.who.int/ghodata/>.

All resolutions can be found on the UNRSC website <http://www.who.int/roadsafety/en/index.html> check the subcategory Resolutions on this homepage.

www.who.int/roadsafety/decade_of_action and www.decadeofaction.org

Sign up for email updates on the Decade:

www.who.int/violence_injury_prevention/email_signup/en/index.html

Join the Decade on Facebook: www.facebook.com/roadsafetydecade

Follow the United Nations Road Safety Collaboration (UNRSC) on Twitter @UNRSC

Share Decade and other road safety photos on Flickr: www.flickr.com/groups/roadsafetydecade

ISO technical committee TC 241, Road Traffic Safety (RTS) and ISO 39001

One of the important stakeholders in the United Nations Road Safety Collaboration/UNRSC is ISO, International Organization for Standardization, represented by Mr. Peter Hartzell, Secretary of the technical committee TC 241 – Road traffic safety management systems. This is the technical committee responsible for developing the new management system standard for road traffic safety – *ISO 39001 – Road traffic safety (RTS) management systems – Requirements with guidance for use*. The standardization work, with the secretariat at SIS, Swedish Standards Institute (the Swedish ISO member NSB/National Standardization Body), was initiated in 2008 and ISO 39001 was published in October 2012. ISO 39001 is an important and integral part of the work of UNRSC.

ISO/TC 241 has 45 member countries (both developed and developing countries) and 20 international liaison organizations, including the World Bank and the World Health Organization.

Read more about ISO work, technical committees and international standards on www.iso.org

The initial idea of an international standard for RTS came from Sweden's work with the Vision Zero programme, where the goal is zero deaths and zero serious injuries due to road traffic crashes. This new ISO management system standard can provide a tool for systematic work and commitment among organizations that affect the safety of the road system. Dr. Claes Tingvall, Director of Traffic Safety at the Swedish Transport Administration, who is the founding father of Sweden's Vision Zero programme, and also the previous (and first) Chair of ISO/TC 241, has said: "I believe that the new standard (ISO 39001) could become as important to road traffic safety as for example the seatbelt or ESP (Electronic Stability Program)."

ISO 39001 is a management system standard, including both requirements and guidance for use. The status as a requirement management system standard puts ISO 39001 in the same distinguished category and family as world-wide established and recognized ISO 9001 (Quality management) and ISO 14001 (Environment management).

ISO technical management board, TMB, has decided on a new common high level structure for all ISO management system standards and ISO/TC 241 has been part of this process.

The new high level structure is now a part of *ISO/IEC Directives, part 1, annex SL, chapter 8 – Guidance on the development process and structure of an MSS* (management system standard). It includes the common structure, terms and definitions and common text for management system standards.

ISO/TC 241 has been an early adopter in this process and the new structure is included in ISO 39001. This is a clear advantage as it enables aligning, working parallel or integrating ISO 39001 with other management systems in an organization where the organization is already working according to e.g. ISO 9001 or ISO 14001. There are already more than one million organizations worldwide that are certified to ISO 9001 and about a quarter of a million to ISO 14001.

ISO management system standards High Level Structure

The new structure for all management system standards, according to which ISO 39001 is developed, has a high level structure of ten clauses which address the following:

- 1) Scope;
- 2) Normative references;
- 3) Terms and references;

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- 4) Context of the organization;
 - 5) Leadership (including commitment and policy);
 - 6) Planning;
 - 7) Support (including coordination, resources, competence, communication and information);
 - 8) Operation;
 - 9) Performance evaluation (including monitoring, measurement, analysis and evaluation, internal audit and management review);
 - 10) Improvement (including nonconformity and corrective action and continual improvement).
- These ten clauses will be common to all management system standards.

ISO 39001 embraces all these parts and also includes its unique content, especially in the planning clause, where RTS performance factors and RTS objectives are addressed. There is also a section within the clause of performance evaluation that addresses road traffic crash and other road traffic incident investigation.

Annexes in ISO 39001

- **Annex A** (informative) provides guidance on the use of the international standard.
- **Annex B** (informative) addresses international work relating to road traffic safety management frameworks.
- **Annex C** (informative) shows the correspondence between ISO 39001:2012 (RTS), ISO 9001:2008 (Quality) and ISO 14001:2004 (Environment).

The Bibliography consists of relevant references including e.g. abovementioned ISO standards, as well as a World Report on Road Traffic Injury Prevention (www.who.int/violence_injury_prevention/publications/road_traffic/world_report/en) and a Global status report on road safety by WHO (www.who/global_status_report_on_road_safety_2009).

The following sections address some of the unique core elements of ISO 39001.

RTS performance factors

The definition of this term is “a measurable factor, element and criterion contributing to RTS that the organization can influence and that allows the organization to determine impacts on RTS.”

Clause 6.3 RTS performance factors states the following:

The organization shall identify for use RTS performance factors from the following list of risk exposure factors, final safety outcome factors and intermediate safety outcome factors, depending on the context (see Clause 4) of the organization and on the risks and opportunities it has identified.

- a) Risk exposure factors:
 - distance travelled and road traffic volume, including vehicle and road user type, whether influenced or not influenced by the organization;
 - volume of product and/or service provided by the organization.
- b) Final safety outcome factors, e.g. the number of deaths and serious injuries
- c) Intermediate safety outcome factors:

these safety outcome factors are related to the safe planning, design and use of the road network and of the products and services within it, the conditions for entry and exit of those products, services and users, as well as the recovery and rehabilitation of road traffic crash victims.

Additional RTS performance factors shall be developed by investigating relevant road traffic incidents and identifying RTS deficiencies.

Based on the RTS performance factors, the organization shall specify elements and criteria in appropriate detail to determine, monitor and measure RTS objectives and RTS targets. The organization shall document this information and keep it up to date.

The definition for the term RTS targets is “detailed performance to be achieved, consistent with the policy and RTS objectives that an organization applies to itself or together with interested parties.”

For example, seat belt use represents both the element and the criterion in relation to the RTS performance factor “use of personal safety equipment”. For the RTS performance factor “vehicle safety”, a consumer safety rating represents the element and the rating level the criterion. Further guidance on the use of the RTS performance factors by different types of organizations is given in annex A.11.

RTS objectives and planning

Furthermore, Clause 6.4 RTS objectives and planning to achieve them address the following:

The organization shall establish RTS objectives at relevant functions and levels.

The RTS objectives shall:

- be consistent with the RTS policy;
- be measurable (if practicable);

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- take into account applicable requirements;
- be monitored;
- be communicated;
- be updated as appropriate.

The organization shall retain documented information on the RTS objectives and the RTS targets.

When establishing and reviewing its RTS objectives and RTS targets, an organization shall take into account its risks and opportunities (in Clause 6.2), its RTS performance factors (in Clause 6.3) and element and criteria (in Clause 6.3) as well as give consideration to its management capacity. It shall also consider its technological options, its financial, operational and business requirements, and the views of interested parties.

When planning how to achieve its RTS objectives and RTS targets, the organization shall determine

- what will be done;
- what resources will be required;
- who will be responsible;
- when it will be complete; and
- how the results will be evaluated.

The action plans shall be documented and reviewed as necessary.

Competence

Clause 7.3 addresses Competence where it states that the organization shall:

- determine the necessary competence of person(s) doing work under its control that affects its RTS performance;
- ensure these persons are competent on the basis of appropriate education, training, or experience;
- where applicable, take actions to acquire the necessary competence, and evaluate the effectiveness of the actions taken;
- retain appropriate documented information as evidence of competence.

Applicable actions can include, for example the provision of training to, the mentoring of, or the re-assignment of current employed persons, or the hiring or contracting of competent persons.

Incident investigation

Clause 9.2 address Road traffic crash and other road traffic incident investigation.

The organization shall establish, implement and maintain a procedure(s) to record, investigate and analyse those road traffic crashes and other incidents in which they are involved that lead, or have the potential to lead, to death and serious injuries of road users, in order to:

- a) determine the underlying factors that it can control and/or influence and that can be causing or contributing to the occurrence of those incidents;
- b) identify the need for RTS corrective action;
- c) identify opportunities for RTS preventive action.

Continual improvement

Similar to other ISO management system standards, ISO 39001 also addresses continual improvement (in Clause 10.2).

The organization shall continually improve the suitability, adequacy or effectiveness of the RTS management system.

This can be achieved through the use of the RTS policy, RTS objectives and RTS targets, audit results, analysis of monitored events, corrective and preventive actions and management review.

The above-mentioned unique parts of ISO 39001 make this international standard an effective and efficient tool to work for improving RTS in a structured manner. Being a management system standard, similar to that of ISO 9001 and ISO 14001, makes ISO 39001 even stronger in application in any type or size of organization, both in developed and in developing countries.

Challenge – leadership, competence and resources

The relevance of any project, including those involved with RTS, is often a function of its leadership, competence and available resources. These factors are major challenges and necessary to address, particularly for developing countries, to progress and succeed in the goal of reducing road crash fatalities and suffering, to unlock growth and to free resources for more productive use.

In the work of ISO technical committees, funding is as important as in any other organization. The challenge is often finding, coordinating and supporting the right competence, i.e. relevant subject matter experts participating in the work, both at and in between meetings. It is important to involve standardization experts (i.e. from ISO and from NSB/National Standardization Bodies) to support the administration and the process of developing the standard. However, it is crucial to identify and gather the subject matter experts from respective country member. This is especially important for developing countries that need to make the very best of the networking and sharing of good practices and bring experience learnt back with them to their public and private organizations – to make a difference in

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the RTS work. Standards are created by those that participate and it is important that those that do participate are the relevant subject matter experts.

For those that want to find out more about this important work, and to ensure that your country is represented in ISO/TC 241, contact your country's NSB/National Standardization Body, through www.iso.org/isomembers

Update on the standardization work

Below are a couple of examples of pilot cases of application of ISO 39001, which can be similarly applied in both developed and developing countries.

Pilot cases of ISO 39001

Japan

The type of organizations involved in the pilots included transport, insurance, consultancy and car leasing companies.

The purpose was to:

- construct a management system and control system in regards to road traffic safety;
- improve awareness of road traffic safety for drivers in the company; to decrease accidents;
- announce the policy of road traffic safety to its clients.

The challenge consisted mainly in identifying relevant RTS performance factors for each company/industry.

Sweden

The type of organizations involved included the Swedish road transport industry:

- 8 000 companies;
- 80% of road transport companies in Sweden;
- 50 000 employees;
- 30 000 motor vehicles.

The purpose of the pilot was to:

- include ISO 39001 in the organization's management system (QMS/ISO 9001, EMS/ISO 14001, ISO 45001/ Occupational Health and Safety management system);
- establish a formula for risk assessment ($R=SC$).

Where

RTS Risk (R) = the probability (S) in percent for a consequence (factor $C = 8$ to 10) to occur within 10 to 20 years multiplied by the factor for the consequence.

Where

$R = 0$ to 1000 ; $S = 0$ to 100 ; and $C = 0$ to 10

Where

$C =$

- 1 incident with injury risk
- 2 minor injury
- 3
- 4 injury
- 5
- 6 major injury
- 7
- 8 serious injury
- 9 very serious injury
- 10 fatality

The challenges of the pilot included the following factors affecting risk:

- Speed;
- Use of safety belt;
- Alcohol and drugs;
- Fitness of driver (incl. considering fatigue);
- Hands free (mobile phone);
- Separation; oncoming traffic;
- Surface texture and friction;
- Load securing.

The Swedish pilot concluded that "All stakeholders should take measures and take responsibility for their own part of RTS and collaborate with other stakeholders. ISO 39001 will be a good standard and guidance in this process."

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For discussion

- **Core arguments**
Humane and economic benefits to save lives (especially for developing countries that are over-represented in road related deaths).
- **Objections vs. Arguments**
 1. Expensive cost vs. Effective investment for the future, especially for developing countries (aspects: humane, economic, environmental, occupational health and safety, etc.).
 2. Close to impossible to achieve (Vision Zero) vs. There is no alternative, we all want to and need to continue the good work that is already being done and spread the activities and positive outcomes.
Paradigm shift of attitude.
 3. Who will be in charge vs. Everyone needs to take responsibility and action to make a difference, at various levels (personal, local, regional, national and international). Various approaches, e.g. collaborations (UNRSC), ISO (standards), public sector (governments and agencies) and private sector (corporations and NGOs, etc.).
- **Current and future challenges and action**
The following are examples of what we all can do to contribute to improve RTS, both on a personal level as well as on an organizational level:
 - Challenge your surroundings to get involved and to be part of the solution, e.g. local public/private sectors, employer, customers, staff, local community, etc;
 - Follow and get involved in the activities of UNRSC and Decade of Action for Road Safety 2011-2020;
 - Embrace ISO 39001: be prepared for publication and certification (e.g. through a pilot application), plug-in and integrate with other ISO MSS (e.g. ISO 9001/Quality, ISO 14001/Environment, ISO 45001/Occupational Health and Safety management system);
 - Upgrade your organization's internal/external audit process including certification;
 - Be aware of local and national information drives, legislation, include RTS in recruitment and staff training process, RTS training in schools (life-long learning);
 - Link RTS to the corporate (or public agencies) culture, structure and strategic process;
 - Link RTS with (corporate) branding and include RTS elements in recruitment and procurement;
 - Stay open minded for good/best practices (local, national and international), share (give and take);
 - Continue sharing between developed and developing countries;
 - Stay in a constructive and an innovative mode.

Conclusion

Road traffic safety, RTS, is a world-wide challenge, concern and focus. There are a lot of activities and tools supporting RTS. E.g. the United Nations Road Safety Collaboration/UNRSC, Decade of Action for Road Safety 2011-2020 and the publication of the new international management system standard ISO 39001. Additionally to these international efforts, there are many local, national and regional activities and projects linked to RTS.

It is important to keep focus on the subject of RTS being high up on the agenda of decision makers.

We need to keep the sense of urgency related to RTS.

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