

TRANSPORTATION RESEARCH
CIRCULAR

Number E-C248

September 2019

**12th International
Conference on
Low-Volume Roads**

September 15–18, 2019
Kalispell, Montana



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* Paper titles marked with an asterisk were withdrawn or not presented at the conference after the deadline for publication in this E-Circular.

Road Surfacing 1

Residue Performance Evaluation of Emulsified Asphalt

DONGDONG GE

ZHANPING YOU

SIYU CHEN

MINGXIAO YE

Michigan Technological University

INTRODUCTION

Emulsified asphalt has been widely used in various surface-treatment methods, such as chip seal, for low-volume roads preservation (1). It is widely used because of environmental and economic concerns (2). Normally the properties of emulsified asphalt are evaluated in terms of the emulsion perspective when water is not evaporated. The workability, settlement and storage stability, and demulsibility property of emulsified asphalt can be evaluated based on standard test methods (3–6). The Texas Department of Transportation developed a surface performance-graded (SPG) specification to evaluate the performance of surface-treatment binders (7). A separate group of researchers proposed emulsion performance-grade (EPG) specification for chip-seal treatments (8). Another group evaluated EPG and SPG systems for national implementation and tried to propose a unified specification (9). However, there is still no agreement about the standard method, especially the standard test conditions and the critical parameter values, to evaluate the performance of the emulsified asphalt residue. More attention should be paid to the performance of residue after water was removed from the emulsified asphalt because the performance of residue related to the characteristics during the service period, not just the property evaluated with the current standard method during the production and storage stage.

This paper is motivated by current emulsified asphalt test methods, which mainly focused on emulsified asphalt during the production and storage stage. Based on current test methods there are limited residue performance evaluations. There is no performance-grade (PG) evaluation system on emulsified asphalt residue. The objective of this paper is to evaluate the performance of emulsified asphalt residue, which focused on the PG of the residue. This paper will mainly focus on the performance of four emulsified asphalt residues commonly used in Minnesota and Michigan based on the PG system. The emulsified asphalt residue was acquired by conducting the emulsified asphalt distillation procedure. The effectiveness of distillation test conditions will be evaluated.

METHODOLOGY

Four different emulsified asphalt binders are adopted in the research. The CRS-2P is the asphalt emulsion used in Minnesota for seal coat. The CSS-1H is the asphalt emulsion used in Minnesota for fog sealing. The CRS-2M and CRS-2TR are two emulsified asphalts used in Michigan for seal coat.

In this study, four types of emulsified asphalt (CRS-2P, CRS-2M, CSS-1H, and CRS-2TR) were used. The four emulsified asphalt residues were acquired by conducting the distillation test based on the ASTM D6997. Different aging levels of asphalt residue were simulated in the laboratory using the Rolling Thin-Film Oven (RTFO) test and the Pressure Aging Vessel (PAV) test. After acquiring asphalt of different aging levels, the dynamic shear rheometer (DSR), the bending beam rheometer (BBR), the asphalt binder cracking device (ABCD), and the Fourier-transform infrared spectroscopy (FTIR) tests were conducted to analyze the performance of emulsified asphalt residue (10–15). For all the tests, at least three replicates were conducted on each test condition. The outline of the test plan is displayed in Figure 1.

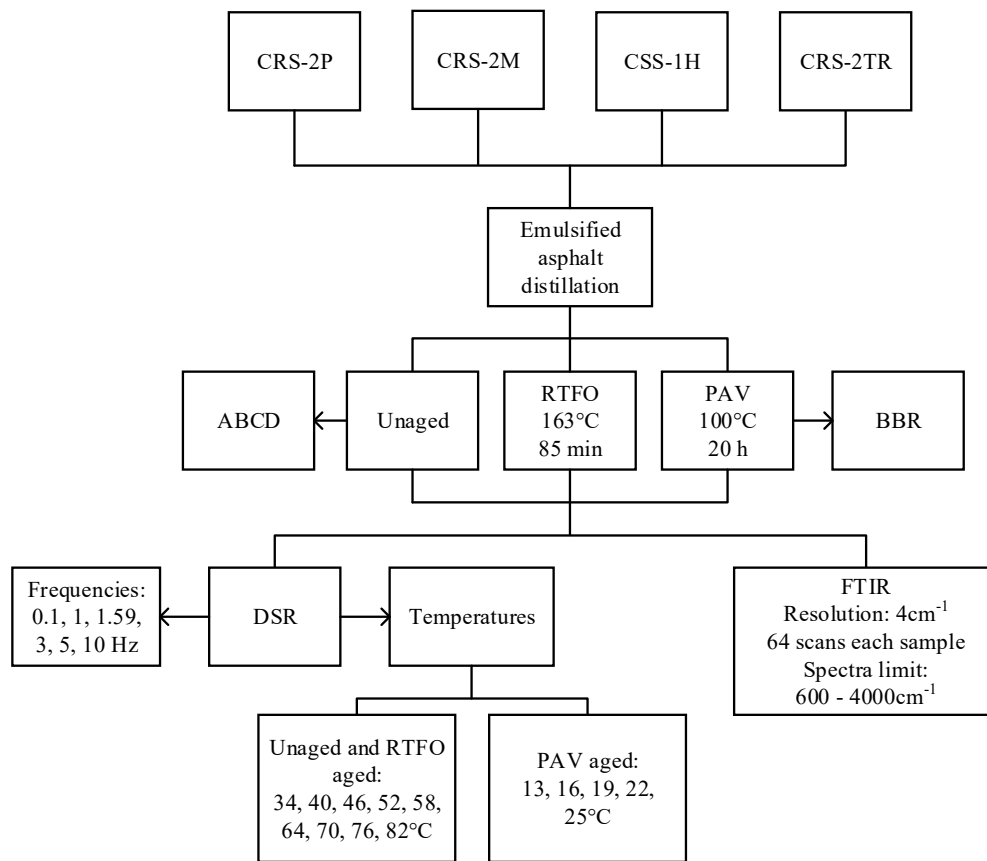


FIGURE 1 The test plan outline for the evaluation of the emulsified asphalt residue.

FINDINGS

During the RTFO aging procedure, the evaporation of volatile component in emulsified asphalt residue decreased the mass, and the oxidation of asphalt residue increased the mass. The mass change is the combined effect of the volatilization and oxidation process. The mass of CRS-2P, CSS-1H, CRS-2M, and CRS-2TR residues after the RTFO test decreased 0.82%, 1.19%, 0.64%, and 1.25%, respectively. During the distillation of emulsified asphalt, some surfactants were retained in the residue. The surfactants volatilized during the short-term aging; this may be the reason why the mass change was higher than 1%. This also proved that the parameter value for the asphalt during the short-term aging evaluation should be adjusted to fit for the emulsified asphalt binder; 1.5% or 2.0% may be a more appropriate value.

The G^* of CSS-1H emulsified asphalt residue was higher than other types of emulsified asphalt residues under different aging conditions. This may be because the CSS-1H was produced with harder asphalt binder than other emulsified asphalt. The influence of the source of asphalt to produce emulsified asphalt should be studied in the future research. Based on the ASTM D6373, for unaged asphalt binder, the minimum $G^*/\sin\delta$ is 1.0 kPa under 10 rad/s angular frequency at the test temperature. The temperatures that met the criteria for CRS-2P, CSS-1H, CRS-2M, and CRS-2TR were 64°C, 70°C, 58°C, and 58°C, respectively. Based on the ASTM D6373, for RTFO-aged asphalt binder, the minimum $G^*/\sin\delta$ is 2.2 kPa under 10 rad/s angular frequency at the test temperature. The temperatures that met the criteria for CRS-2P, CSS-1H, CRS-2M, and CRS-2TR were 64°C, 70°C, 64°C, and 58°C, respectively. Based on the DSR result on the unaged and RTFO-aged emulsified asphalt, the high-temperature PG of CRS-2P, CSS-1H, CRS-2M, and CRS-2TR were 64°C, 70°C, 58°C, and 58°C, respectively.

Based on the BBR test results, the low-temperature PG of CRS-2P was -28°C, the low-temperature performance grade of CRS-2M was -22°C. Both the CSS-1H and CRS-2TR failed to meet the standard criteria at -22°C.

The ABCD test was conducted to evaluate the low-temperature performance of different emulsified asphalt binders. The low-temperature performance grade of CRS-2P, CSS-1H, CRS-2M, and CRS-2TR based on the ABCD test are -27.8°C, -23.0°C, -32.2°C, and -34.7°C, respectively. The low-temperature grade for CRS-2P was similar based on the BBR and ABCD test results. The BBR test underestimated the low-temperature grade for CSS-1H, CRS-2M, and CRS-2TR. The low-temperature grade difference for CRS-2P between BBR and ABCD test was the largest.

In order to examine the water existence in emulsified asphalt residue, the -OH (3,000–3,600 cm^{-1}) functional group in water was evaluated in the unaged emulsified asphalt residue. That there are no peaks in the test result proved that there is no water in the distilled residue. The mass loss during the RTFO aging procedure had no relationship to the water; the mass loss was mainly due to the volatilization and oxidation process.

The FTIR test was adopted to evaluate the chemical bonds of the emulsified asphalt residue during the aging procedure. The spectrum area of C = O and S = O area for different emulsified asphalt binders under different aging conditions are calculated and the results are

displayed in Table 1. Based on the FTIR test results, The S = O and C = O bond were not significantly influenced by the RTFO aging. The RTFO aging was not significantly influenced by the S = O and C = O in emulsified asphalt. The S = O of some emulsified asphalt residues did not correlate to the aging conditions of emulsified asphalt residue. The C = O can better characterize the aging of emulsified asphalt than S = O. Based on the FTIR test result, there is no water in the unaged asphalt residue, which proved the effectiveness of the distillation procedure to remove water in emulsified asphalt. However, the distillation procedure used 260°C to evaporate water from emulsified asphalt, and the high temperature may have already aged the binder. Lower distillation temperatures or shorter distillation duration can reduce the influence of high temperature and long duration on the aging performance of emulsified asphalt.

CONCLUSION

In this paper, two emulsified asphalts used in Minnesota for seal coat and fog sealing, and two emulsified asphalts used in Michigan for seal coat, were adopted. The DSR, BBR, ABCD, and FTIR tests were used to analyze the performance of emulsified asphalt residue. The standard mass change criteria should be adjusted to fit the emulsified asphalt binder. The high-temperature performance grade was different. The BBR test underestimated the low-temperature grade for CSS-1H, CRS-2M, and CRS-2TR. The mass loss during the RTFO aging procedure had no relationship to the water, and the mass loss was mainly due to the volatilization and oxidation process. The C = O can better characterize the aging of emulsified asphalt than the S = O. A lower distillation temperature or a shorter distillation duration can reduce the influence of high temperature and long duration on the aging performance of emulsified asphalt. Even though such an observation is based on limited samples and tests, it will be necessary to further investigate in order to verify such a relationship.

TABLE 1 Spectrum Area of C = O and S = O for Different Emulsified Asphalt Binders

Asphalt Type	Chemical Bond	Unaged	RTFO Aged	PAV Aged
CSS-1H	C = O	0.489	0.598	0.790
	S = O	0.346	0.351	0.442
CRS-2P	C = O	0.832	1.059	1.306
	S = O	0.214	0.340	0.280
CRS-2M	C = O	0.428	0.578	0.912
	S = O	0.197	0.194	0.358
CRS-2TR	C = O	0.362	0.493	0.796
	S = O	0.282	0.226	0.380

ACKNOWLEDGMENTS

This study is sponsored by the Michigan Department of Environmental Quality (MDEQ). The U.S. National Science Foundation (NSF) also provide partial research grant through the SusChem/Collaborative Research Program to enable this study to be completed. Any opinions, findings, and conclusions expressed in this material are those of the authors and do not necessarily reflect the views of NSF and MDEQ.

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Design, Construction, and Preliminary Investigations of Otta Seal in Iowa

SHARIF Y. GUSHGARI

YANG ZHANG

ALI NAHVI

HALIL CEYLAN

SUNGHWAN KIM

ALI ARABZADEH

CHARLES T. JAHREN

Iowa State University

CHARLES OVERBY

Norwegian Public Roads Administration

Faced with limited financial resources, pavement engineers constantly seek more durable and more economical technologies for road preservations and rehabilitations. Consequently, there have been many efforts to study resurfacing strategies, including various types of sealing for local roads. Among different sealing methodologies, Otta seal is a technique that has not yet been studied sufficiently in the United States. For this investigation, the first Otta seal site in the state of Iowa was constructed using a double-layer Otta seal design over 6.4 km of cracked asphalt pavement. Otta seal design and construction details are documented and discussed, and test sections using various aggregates are compared for performance. The key lesson learned was that proper aggregate selection within gradation limits and aggregate spread rates were critical factors for Otta seal performance. Otta seal capability for holding loose aggregate particles and for dust control were examined, and there were indications that excessive proportion of fine aggregate particles could lead to diminished performance associated with fugitive dust emissions and unbound aggregate particles. Although the Otta seal provided a smooth surface satisfying road user and agency requirements, it did not significantly add structural capacity to the existing asphalt pavement. The findings from this study will benefit road officials and other decision makers who need to consider alternatives for resurfacing distressed low-volume asphalt roads.

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ROAD SURFACINGS 1

Full-Scale Field Implementation of Otta Seal in Iowa

BRIAN P. MOORE

Iowa County Engineers Association Service Bureau

BRIAN KEIERLEBER

Buchanan County, Iowa

BRANDON BILLINGS

Cerro Gordo, Iowa

ADAM SHUTT

Louisa County, Iowa

TRAVIS MALONE

Adams County, Iowa

INTRODUCTION

Iowa has over 71,000 mi of unpaved secondary roads with many carrying very low daily traffic volumes. Although the volume may be low, the system must carry the increasing number of heavy agricultural vehicles prominent in Iowa. Iowa's county road departments spend over \$110 million annually for aggregate replacement on gravel roads alone. The excellent performance of Otta seal in other countries and in neighboring states has increased the interest of Iowa County Engineers, who are always in search of a more economical approach for low-volume road service and maintenance.

Otta seal, first developed in Norway, was developed as a low-cost maintenance surface alternative for unpaved gravel roads with low bearing capacity under spring-thaw periods. Consequently, Otta seal has been used in northern Europe and Africa, among others, as an economical and practical alternative to traditional bituminous surface treatments. Until recently, only two states in the United States (Minnesota and South Dakota) had reported Otta seal construction experiences. Both states reported some promising and long-lasting results from their Otta seal projects.

Iowa County Engineers desired to implement this technology to verify its short- and long-term performance. In 2017, Cherokee County, Iowa, built Iowa's first Otta seal project through the Iowa Highway Research Board (IHRB) Project TR-674, "Evaluation of Otta Seal Surfacing for Low-Volume Roads in Iowa" (Figure 1). Based on the success of this first project, 14 mi in four counties were constructed in 2018 (Figure 2). These were built as part of IHRB Project TR-753, "Evaluation of Otta Seal Surfacing for Low-Volume Roads in Iowa—Phase II."



FIGURE 1 2017 construction.

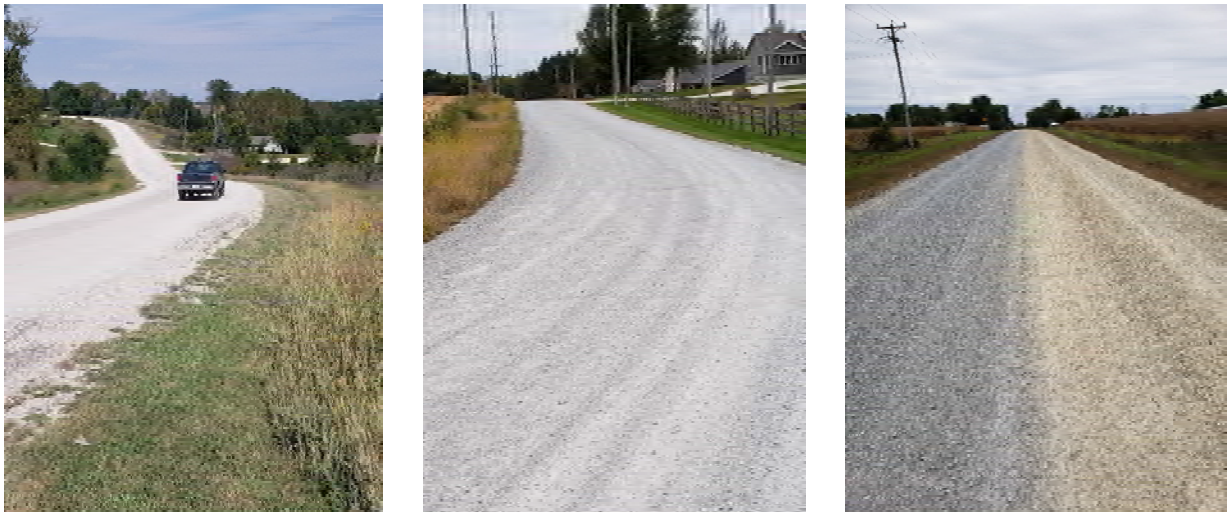


FIGURE 2 2018 construction.

METHODOLOGY

Results of TR-647 have shown that Otta seal has many positive advantages:

- It allows the use of uncrushed aggregate (up to 1-in. diameter), leading to cost reduction in aggregate production and transportation;
- It acts as an impermeable surfacing by filling up aggregate voids, thus preventing water from penetrating moisture-susceptible gravel roads;
- It does not require the use of a prime coat in construction;
- It can be opened to traffic immediately after construction; and
- Fewer periodic maintenance activities are required between reseals.

Based on the successful research findings of TR-674, the technical advisory committee recommended another research project, TR-753, to provide Iowa County Engineers with some of the following tools and answers:

- Establish standard specifications, including quality assurance/quality control (QA/QC) procedures for Iowa Otta seal construction projects;
- Evaluate various local aggregate sources in Iowa for suitability for Otta seal construction;
- Develop a rational or engineered approach for determining the optimum application rates for asphalt binder and aggregate in Otta seal construction; and
- Identify road surface–base preparation requirements before Otta seal application.

FINDINGS

As part of TR-753, almost 14 mi of county-owned roadways were treated with a double Otta seal in the summer of 2018.

To be as consistent as possible across all test sections, a set of preliminary QA/QC procedures were provided to each county based on the type of binder and aggregate source being used. Table 1 shows the procedures used by Cherokee County for their construction.

TABLE 1 2018 Otta Seal Construction Procedures

County	Cherokee	Buchanan	Louisa	Ringgold
AADT	130–210	30–500	70	60
Mileage	5.7	1.2	1	6
Existing surface	Gravel	Gravel, PCC, ACC	Gravel	Gravel

AADT = annual average daily traffic, ACC = asphalt cement concrete, and PCC = portland cement concrete.

Materials

Emulsion

HFMS designates high float and medium set. HFMS-2s is a mixture of asphalt, water, and emulsified agent, typically containing up to 35% water. HFMS-2s requires 8 to 10 days for setting (allowing the water to evaporate), meaning that the second layer of Otta seal construction (for double Otta seal projects) is delayed at least 8 to 10 days after the first layer is constructed. HFMS-2s must be from an approved source by Cherokee Road Department and Iowa Department of Transportation.

Aggregate

Based on the traffic volume and truck traffic on Old 21 Road north to 500th Street and O Avenue to 520 Street, it was recommended to apply a dense aggregate gradation for Otta seal construction in Cherokee County.

Construction Criteria

- Construct Otta seal only between May 1 and October 1;
- Construct Otta seal only during daylight hours;
- Construct Otta seal only when the pavement temperature and air temperature is 35°F and rising;
 - Construct Otta seal only if wind does not cause uneven spraying of the bituminous material for mixture; and
 - No rain before and during the construction, but if it rains after construction, it will not affect the final surface.

Equipment

Emulsion Distributor

- Must check all the nozzles if any are clogged,
- Must be calibrated before the spraying,
- Must check the application pressure, and
- Must spray in triple pattern application.

Aggregate Spreader

- The truck aggregate haul box must be clean and free of other materials.
- The aggregate spreader and the truck hook-up hitches must be in good condition.
- Use a self-propelled mechanical-type aggregate spreader that is capable of distributing the aggregate uniformly to the required width and rate.

Rollers

- Four pneumatic rollers at a minimum weight of 12 tons are required.
- One pass with a 10- to 12-ton static tandem steel roller after the initial rolling is required.
 - The roller-tire sizes, ratings, and pressures must comply with the manufacturer's recommendations.
 - The tire pressure must be the same on all tires.
 - The tire surface must be smooth.

Before Construction

- Record ambient and surface temperatures;
- Ensure all quantity of required materials are available on site;
- Make sure enough personnel are present on site;
- Verify road closure and detour signs are present; and
- Determine wind speed and direction.

Application Process

Spraying HFMS-2s

- A test section of 100 ft long is recommended to ensure the application rate of 0.5 gal/yd² is maintained. This must be confirmed by the contractor and field engineer.
 - The required storage temperature is 122°F–140°F.
 - The required application temperature is 122°F–185°F.
 - Continue with the emulsion spraying if the above requirements are met.

Spreading Aggregate (Class A and Crushed Limestone)

- Calibrate the aggregate spreader in accordance with ASTM D5624.
- Apply aggregate at the rate of 50 lbs/yd² in steel container and weigh it to confirm the actual aggregate application rate, which is 50 lbs/yd².
 - Verify that the aggregate spreading truck operates at the same speed as the emulsion spraying truck.
 - Continue with the aggregate spreading if the application rate is correct.

Rolling and Compaction Operation

- A minimum of 15 passes with a pneumatic-tired roller on each direction (total of 30 passes in both directions). The field engineer may require more passes to ensure the rolling is sufficient.
 - The required speed to reach the required compaction level is 4 mph. The field engineer may ask the operator to maintain that speed or lower it.

Second Otta Seal–Layer Application

- It is necessary to broom the first layer’s surface on the same day that the second layer is constructed.
- Apply a second application of emulsions and aggregate 10 days to three weeks after the initial application, steps 1–3 need to be repeated for a successful second layer.

Post Construction

- One month after construction of the second layer, the final surface must be swept of loose aggregate.
- One month after construction of the second layer, road markings can be added if desired.

CONCLUSION

Iowa County Engineers are hopeful that Otta seal will prove to be a lower cost alternative treatment that will produce lower long-term maintenance costs. Construction costs for double Otta seal applications are shown in Table 2. Prices varied based on the cost of the local

TABLE 2 Double Otta Seal Construction Costs

County	Cherokee	Buchanan	Louisa	Ringgold
Costs	\$55K/mi	\$62K/mi	\$82K/mi	\$43K/mi



FIGURE 3 Spring 2019.

aggregate used by each county and the distance the contractor had to travel to perform the work. One contractor performed the work in all the counties except Ringgold County. Ringgold County had its own construction equipment and applied the Otta seal with its own forces.

Construction of Otta seal in Iowa to date has gone well. Using the preliminary QA/QC procedures has produced a very good product that appears to be performing as predicted. TR-753 will continue to monitor the performance of the projects built in 2018. Some positive aspects of Otta seal realized through construction are the following:

- Flexibility with respect to the use of locally available natural aggregates for producing the graded aggregate. Thus, crushed or uncrushed aggregate or a combination of both can be used in the construction of Otta seal.
 - Construction can be performed with county equipment and labor.
 - Existing contractors can construct Otta seal projects without needing to use any special equipment.
 - Lowered maintenance costs due to reduced need of blading and adding of aggregate to Otta seal roads.
 - Enhanced durability of pavements and aggregate treated with Otta seal.
 - Greatly reduced loss of aggregate (dust) of Otta seal constructed on existing aggregate surfaces.
 - Favorably public acceptance of Otta seal due to reduction of dust on exiting aggregate roads.

Performance of Chip Seal Treatments in Two Different Climatic Regions

ADRIANA VARGAS-NORDBECK
National Center for Asphalt Technology

MICHAEL C. VRTIS
BENJAMIN WOREL
Minnesota Department of Transportation

Over time, new pavements deteriorate because of the effect of traffic loads and the environment. Pavement preservation treatments, such as chip seals, are a cost-effective alternative for extending the service life of the pavement without incurring costly rehabilitation or reconstruction activities. Chip seals are preservation treatments that can help protect the pavement structure, reduce the rate of pavement deterioration, improve skid resistance, and address minor surface problems. As part of the National Center for Asphalt Technology and the Minnesota Road Research Facility Pavement Preservation Study, chip seal test sections were placed on low-volume roads in Alabama and Minnesota. The two locations were selected to represent different climate conditions. Lee County Road 159 in Auburn, Alabama, is subjected to a warm, wet, no-freeze climate, while County State Aid Highway 8 is located in a cold, wet, freeze area in Pease, Minnesota. Treatments have been in service for approximately 6 years in the southern sections, and 2 years in the northern sections. During this time, cracking, roughness, rutting, and macrotexture data were collected periodically to evaluate pavement performance. The results determined that cracking is a predominant form of distress for these treatments. While the treatments are not expected to address rutting or roughness, the results indicated little variation in the case of the southern treatments, and an increasing trend in international roughness index in the northern sections, likely related to the appearance of thermal cracking. Macrotexture data may be used to assess the functional life of the treatments.

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Chloride-Stabilized Roads Versus Thin Bituminous Surface *A Comparative Analysis*

ALELÍ OSORIO-LIRD
ÁLVARO GONZÁLEZ
ALONDRA CHAMORRO

Pontificia Universidad Católica de Chile

Given the increasing use of chloride stabilizers to improve the level of service and structural capacity of low-volume roads (LVRs), and the alternative to use thin bituminous surface (TBS) for these roads, this paper addresses a comparative analysis between both alternatives. The objective was to perform a comparative analysis of the characteristics of chloride-stabilized roads (CSR) and TBS roads in terms of serviceability and structural performance. To achieve this goal, road sections built with both alternatives were analyzed in the field and laboratory. In the field, CSR presented higher values and variability of international roughness index than TBS during the analyzed period. Overall surface performance presented values slightly lower for CSR than TBS, but with similar variability. In the laboratory, California bearing ratio (CBR) for unbound materials and CSR samples presented similar behavior. Costs per kilometer for TBS roads are 2.3 to 4.2 times higher than those for CSR, considering different alternatives of TBS with CSR structures built with magnesium chloride and sodium chloride. Given the outcomes of the study, the decision between CSR and TBS roads should be made considering the service conditions in terms of climate and traffic demand and expected level of service, considering that CSR could present more variability overtime, but also a more cost-effective alternative compared to TBS for LVRs.

INTRODUCTION

Background

Rural roads play a crucial role in the economic and social development of societies, linking rural communities to education, health services, and markets (1). In developing countries, rural roads are commonly low-volume roads (LVRs) designed to meet the social and economic needs of the rural population (2). In these countries, rural roads represent more than 80% of the total road network, carry 20% of the total motorized traffic, and provide access to the majority of the population to main roads (3). LVRs have a traffic with less than 400 annual average daily traffic (4).

LVRs are broadly classified as sealed or unsealed depending on whether they have a sealed surface, which is related to the traffic volumes they serve as well as the productive and functional characteristics of the network (5).

The Chilean LVR network consists of earth, unbound granular materials, and sealed roads, which include stabilized granular materials, and thin bituminous surfaces (TBSs). Figure 1 shows typical cross sections of structures used in LVRs in Chile. Unsealed LVRs (Figure 1a)

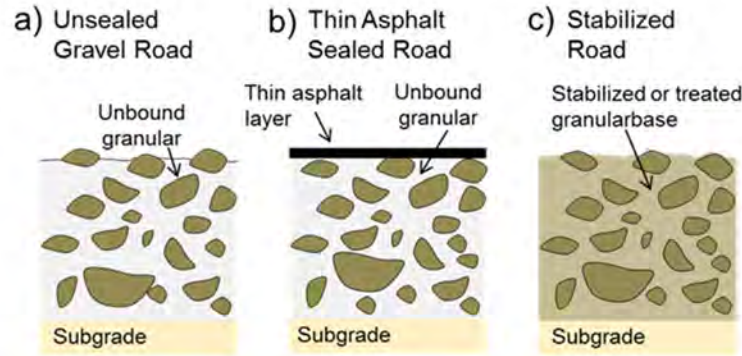


FIGURE 1 Typical cross sections of structure used in LVRs in Chile (9).

have a single unbound granular material (UGM) layer over the existing soil or subgrade. TBS roads have a thin asphalt treatment on top of the UGM layer (Figure 1b) that protects the underlying materials from traffic and climate. Stabilized roads (Figure 1c) have between 1% and 5% stabilizing agent added to the UGM that agglomerates and cements the fine particles, improving the shear strength and cohesion for specific climate and traffic conditions (6–9).

Stabilized roads are commonly built where good quality materials are not locally available and the hauling distance to quarries is too far. Chemical stabilization is then used to improve the mechanical behavior of soils for LVR construction (10). Stabilized rural roads present lower deterioration rates and longer service lives compared to unsealed roads; however, the performance significantly depends on the type of agent used and the thickness of the treated layer (11).

Soil stabilizers are classified into two categories: traditional stabilizers such as lime, fly ash, cement, and bituminous products; and nontraditional products of a variety of chemical agents (10). Dust palliatives, such as sodium and magnesium chlorides, are also used as chemical stabilizers when properly mixed with the natural soil in thicknesses greater than 15 cm. Stabilizing the gravel surfaces reduces dust, improves safety, and reduces washboarding and raveling, which in turn leads to reduced maintenance costs, stream sedimentation, and aggregate depletion (12). Chlorides stabilize granular materials mainly through the hygroscopic mechanism, absorbing and retaining the air relative humidity, reducing the rolling on face erosion, and increasing durability (13).

The stabilization of granular materials with chlorides has increased in Chile in the last decade, and currently is commonly applied, mainly in arid areas. In the order of 7,000 km of public roads have been treated with chlorides in the north of Chile (14). The increase in using chlorides for stabilization is explained by the advantages of low construction costs, low maintenance costs, reduction of vehicle-dust emissions, and a good quality perception from road users (13).

TBS roads are a good alternative when a better riding surface is needed without an additional structural strength. However, indirectly TBS protects the unbound granular layer (Figure 1b) from surface-water infiltration and aggregate loss, indirectly providing additional strength to the road. TBS roads are classified in three categories, based on their function and road layer in which is applied. In Category 1 TBS is applied directly on subgrades as dust palliative; in Category 2 TBS is applied on top of the unbound granular layer; and in Category 3 TBS is applied on top of granular base and subbase, instead of a hot-mix asphalt layer (12). This research refers to the Class 2 TBS (Figure 1b) and is referred to as TBS roads.

Studies have centered their efforts on identifying the main factors affecting the deterioration of specific types of sealants or stabilizers and on performance models of specific distresses (15, 16). The most known are the Highway Design and Maintenance Standards Model, developed by the World Bank during the 1980s; the Road Investment Model for Developing Countries, developed by the Transport and Road Research Laboratory; and the Australian Road Research Board models developed in Australia (17).

Recent studies addressed the performance of LVRs, including chloride-stabilized roads (CSR) and TBS roads, as the development of methodology for evaluation of unpaved road in general and sealed roads in particular (9, 13, 18–20), the development of condition performance models for network-level management for sealed roads in Chile (11), and for sealed granular pavements in Australia (21).

Given the increasing use of chloride stabilizers to improve the level of service of LVRs, and the alternative of TBS roads, this paper addresses a comparative analysis between both alternatives (13). The findings of the analysis provide relevant information to stakeholders for decision making on the selection of LVR type and materials.

Objective and Scope

The objective of this study was to perform a comparative analysis of the characteristics of CSR and TBS roads. To achieve this goal, the performance of road sections built with both alternatives were analyzed. The specific objectives of the research were (1) to analyze the field indicators as international roughness index (IRI) and overall condition indexes for each alternative, (2) to present laboratory strength characteristics of CSR and TBS roads granular layers, and (3) to present average construction and maintenance costs. The study was limited to LVR sections located in different regions of Chile. Stabilized granular materials studied are limited to CSR with sodium and magnesium chloride, and TBS roads are limited to Category 2 TBS.

METHODOLOGY

Characteristics of Analyzed LVR

The structure of unsealed LVRs consists of 15 cm of a single compacted layer of selected unbound granular material over the natural soil. CSR consist of sodium- or magnesium chloride–stabilized granular materials (Figure 1c); TBS roads consist of an unbound granular material with one type of TBS (Figure 1b), including cape seal, surface treatment, and double surface treatment.

Data Collection

The field data collection was carried out using the window method for sampling road sections with different ages to obtain the data required within the period of the study. This method has been widely used in road engineering (22–24).

The sections evaluated in the study are located in different regions of Chile (Figure 2). CSR sections are located in the north, and TBS mainly in the central and south regions of the country.

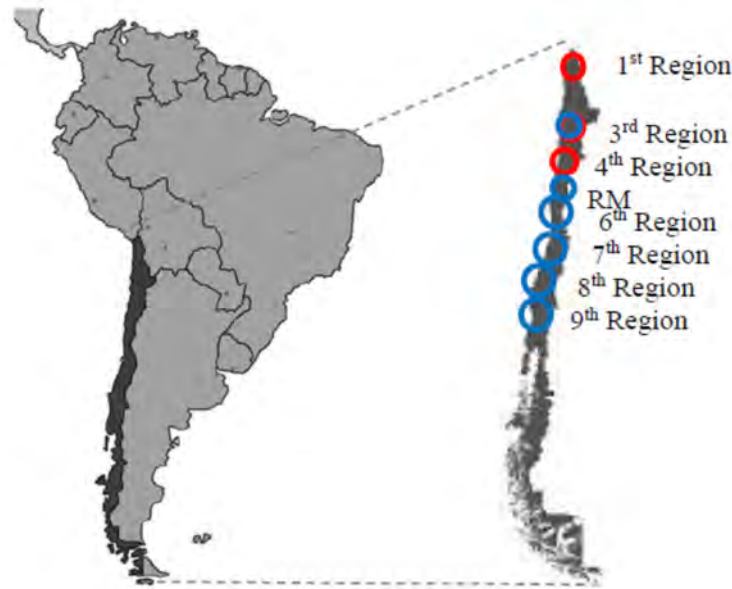


FIGURE 2 Regions of Chile where the data were collected. TBS in blue, and CSR in red.

The climate in the north, central, and southern areas of Chile is different. In the north (1st to 4th regions) the dry climate is classified as mid-latitude desert by the Köppen Climate Classification System (KCCS), with a rainy season of less than 4 months, a maximum monthly precipitation less than 50 mm, and an annual mean monthly precipitation less than 20 mm. The central region features a Mediterranean climate and is classified as mid-latitude summer-dry by KCCS, with the rainy season between 4 and 8 months, a maximum monthly precipitation between 50 mm and 400 mm, and an annual mean monthly precipitation between 20 mm and 200 mm. In the south (6th to 9th region) the climate is humid and classified as mid-latitude winter-dry by KCCS, with a rainy season of more than 8 months, a maximum monthly precipitation of more than 400 mm, and an annual mean monthly precipitation of more than 200 mm (20, 22, 25, 26).

IRI and Overall Condition Indexes Comparison

In order to perform a comparative analysis of field indicators of CSR and TBS roads, IRI and overall condition indexes, the following experimental design was proposed. The IRI, overall condition indexes, and the age of CSR and TBS roads were the factors studied. IRI and overall condition indexes were the dependent variables, and the age was the independent variable.

The IRI values were evaluated in the field, using the Roughmeter III (RIII) device, widely used in unsealed LVR. The RIII is a World Bank Class 3, portable response-type device, which provides roughness results by directly measuring the axle displacement and is equipped with an integrated GPS unit (21). The IRI was evaluated in meter/kilometer. The evaluations were performed at a range of speed between 40 and 60 km/h. The IRI measurements were carried out on 1.5 and 2 km road sections, but the data processing was for the section as well as for sample units within the roads. In total, 77 and 155 sections of TBS and CSR were evaluated, respectively.

The overall condition indexes selected to evaluate roads were the Stabilized Road

Condition Index (SRCI) for CSR and PCI_{ST} (pavement condition index for treatment surfaces) for TBS. The first was developed and validated in a previous stage of this project and considered pavement sections of sealed roads (18, 27). The second is an index developed for surface treatments in Chile and was validated for TBS in a previous stage of this project (18, 23). Both indexes represent the overall condition of unpaved roads from field measurements, consisting of numerical value on a scale from 1 (worst condition) to 10 (best condition). The data used for the condition indexes calculation are collected from road sample unit of dimensions: 50 m long and between 2.5 and 4.0 m wide. The sample unit should be a homogeneous section of the road. The SRCI value is calculated using Equations 1 and 2, and PCI_{ST} is obtained with Equation 3.

$$\text{SRCI} = 9.51 - 0.30 \text{ IRI} - 0.09 \text{ CR} - 1.98 \text{ PT} - 0.22 \text{ ER} - 3.72 \text{ CW} \quad (1)$$

$$\text{SRCI} = 10 - 1.48 \text{ ER} - 2.81 \text{ CW} - 4.64 \text{ RA} \quad (2)$$

where

IRI = international roughness index (m/km);

CR = corrugations, measured as the mean vertical deformation observed in a section (cm);

PT = potholes, measured as total square meters observed in a sample section, calculated as the product of the mean diameter (m), typical depth (m), and number of potholes in a sample section;

ER = erosion, a dummy variable (1 if either erosion depth or width is greater than or equal to 5 cm, 0 otherwise);

CW = crown condition, the average between drainage and transverse profile condition; both defects are rated as 0 when observed in good condition, 0.5 in fair condition, and 1 in poor condition; and

RA = raveling, as the percentage of the surface of the sample section.

$$\text{PCI}_{\text{ST}} = 10.7 - 0.64 \text{ IRI} - 0.06 \text{ RT} - 0.05 \text{ PT} - 0.045 \text{ CR} - 0.02 \text{ BL} - 0.01 \text{ RA} \quad (3)$$

where

IRI = international roughness index (m/km) measured with RIII;

RT = rutting, or transverse deformations caused by loose aggregate, measured as the mean vertical deformation observed in a section (mm);

PT = potholes, measured as the total square meters observed in a sample section, calculated as the sum of the surface of the quadrilaterals that contains the potholes in a sample section;

CR = percentage of cracks, as a weighted sum of linear cracks, fatigue cracks, and thermal cracks;

BL = bleeding, as a percentage of the surface of the sample section; and

RA = raveling, as a percentage of the surface of the sample section.

The Ministry of Public Works of Chile provides information about the year of construction of the roads. The age of the roads varied from 0 to 10 years for CSR and from 2 to 13 years for TBS roads. The sections of roads were chosen based on information available about

the age of construction, and those that had not received maintenance since construction.

The data were processed with descriptive statistics and boxplots to find outliers within the field data collected. These sections were studied considering technical criteria in order to decide the elimination from the database, for example, sections presenting an IRI value for CSR very low for advanced age, as $IRI=1.5$ m/km for a road with 9 years of service. These values could be explained by an unknown maintenance after construction or an evaluation error.

The comparative analysis was performed with the data without the outliers as well as with the difference of means per age, when the information of age for both alternatives was available.

Laboratory Tests

During construction of TBS and chloride-stabilized road sections samples of unbound granular materials were collected from different sites (9). The sealed plastic bags with material samples were transported to the laboratory. The maximum particle size of the material samples was 2", with 74% passing the 4.75 mm (#4) sieve, 30% passing the 0.475 mm (#40) sieve, and 9% passing the 0.075 mm (#200) sieve. The optimum moisture content of the nonplastic, crushed unbound granular material was 5.7%, and the maximum dry density was 2.10 t/m³. Two samples of unbound granular material were later mixed with 80 kg/m³ of chlorides commonly used for road stabilization in Chile. The first sample was stabilized with 80 kg/m³ of magnesium chloride, and the second with 80 kg/m³ of sodium chloride. Finally, the material samples at their optimum moisture content were compacted with a modified proctor hammer in a CBR mould. Later, CBR tests were conducted on the specimens. In addition to the CBR tests on chloride-stabilized materials, control or reference tests without stabilization were conducted on the unbound granular materials

Costs

The focus of this article is the performance of TBS and chloride-stabilized roads; however, the authors performed a general construction cost analysis for both types of LVR. To avoid the effect of local currency, the analysis used relative costs (i.e., the relative construction cost of one kilometer of chloride-stabilized road compared to the construction cost of one kilometer of TBS). Data from the Ministry of Public Works (18) were available and updated to current values using known economic indicators like the cost of a crude oil barrel and the cost of the U.S. dollar (USD) relative to local currency. Data were available for chloride roads and different types of TBS roads. Initial construction costs and maintenance costs were obtained.

RESULTS ANALYSIS

IRI Comparison

The RIII could process the data evaluated every 10 m, with a minimum length of 50 m. The frequency of 10 m was chosen. The data processing of each file created by the RIII consisted of an analysis of variability of the section (length between 1.5 and 2 km). The sections with a coefficient of variation over 0.3 were not considered in the set of data analyzed. Then, a

representative sample unit of 50 m was selected for each section, considering the mean as the representative value. The data collected of IRI values for CSR and TBS after this data processing are presented in Figure 3. In general, the IRI values for CSR present higher values than TBS and higher dispersion.

A descriptive statistic and boxplot analysis was performed on both sets of data to identify outliers. Once the outliers were analyzed, and eliminated in some cases, the resulting boxplot is shown in Figure 4. It is observed that IRI values for CBS are higher than TBS overtime and present higher variability.

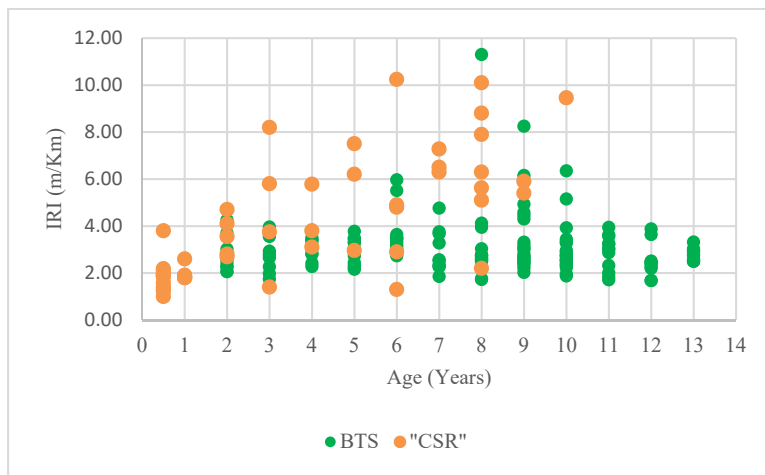


FIGURE 3 IRI data collected for CSR and TBS roads.

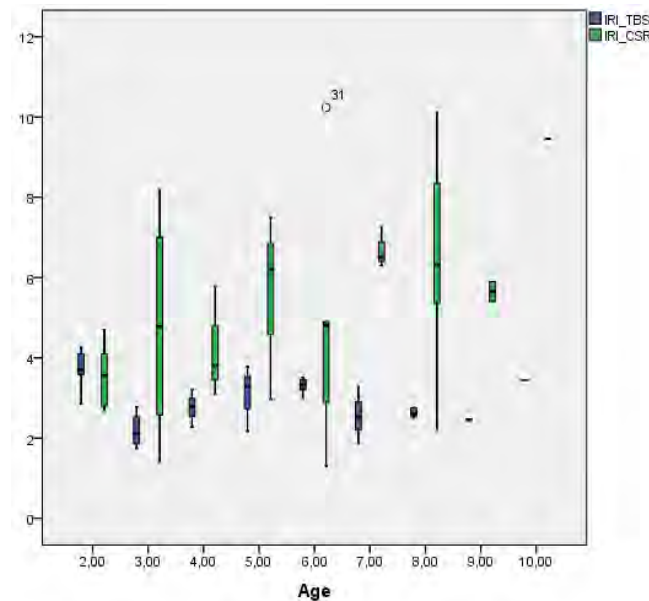


FIGURE 4 Boxplots of IRI values for CSR (green) and TBS roads (blue).

The normal distribution of IRI values was analyzed using an analysis of goodness of fit for each year of the analysis using the Shapiro-Wilk test due to the values of less than 50. This analysis was important to corroborate if a t -test could be applied to analyze the difference between IRI means. From this test it is observed that p -value (Sig.) is greater than 0.05 in all cases, but in Age = 1 for CSR IRI. Therefore, a t -test for the difference of means could be applied to confirm the difference of means for each year to confirm the difference in IRI values for CSR and TBS, mainly for the ages where the values seems to be similar.

The results of the t -test for differences in means indicates that IRI means values of CSR and TBS roads for ages 2 and 6 do not present significant differences for a confidence level of 95%; however, the values for age 4 present significant differences. The statistical values are the following:

$$\text{Age} = 2 \rightarrow t = 0.405 < t_{\text{crit}}(0.025,19) = 2.093$$

$$\text{Age} = 4 \rightarrow t = -2.624 > t_{\text{crit}}(0.025,10) = 2.228$$

$$\text{Age} = 6 \rightarrow t = -0.462 < t_{\text{crit}}(0.025,17) = 2.111$$

Overall Condition Indexes

The SRCI and PCI_{ST} were evaluated in the field on 19 and 45 sections of CSR and TBS roads, respectively. Equations 1 and 2 were used to calculate SRCI from field data collection, depending of the availability of IRI values for each sample unit. A representative sample unit of 50 m was selected for each section. The data collected for CSR and TBS after this data processing are presented in Figure 5. In general terms, overall condition indexes for CSR show higher values than for TBS roads.

A descriptive statistic and boxplot analysis was performed to both sets of data to identify outliers. Once the outliers were analyzed, and eliminated in some cases, the resulting boxplot is shown in Figure 6. On the contrary of what happen with IRI values, overall condition indexes for both types of roads do not present a clear tendency about higher values one than another. In addition, it presents high variability within specific ages of the roads, at different ages for each type of material.

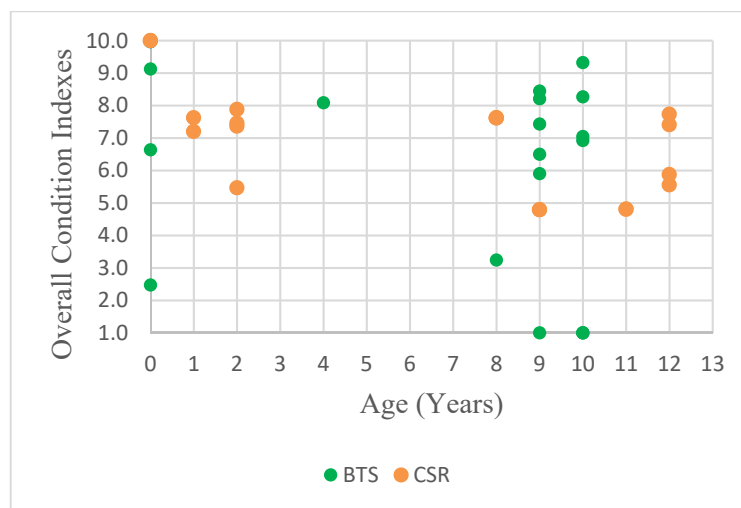


FIGURE 5 Overall condition index values for CSR and TBS roads.

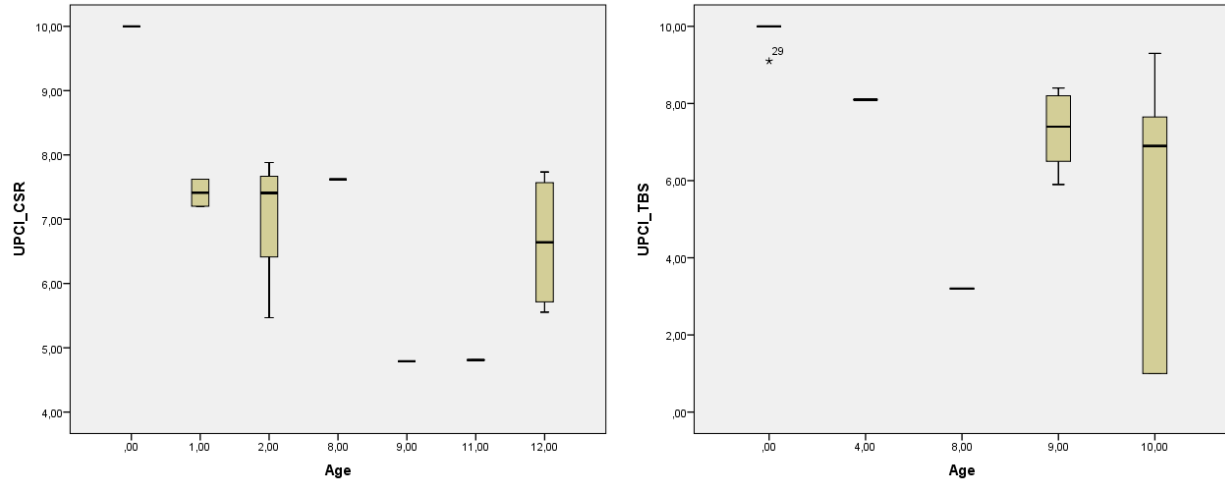


FIGURE 6 Boxplots of overall condition index values for CSR (*left*) and TBS (*right*) roads.

CBR Comparison

Table 1 presents the results for the CBR test on the unbound granular materials without stabilization or reference materials and the chloride-stabilized materials. Results show a small reduction in the CBR of the material treated with chlorides. In other words, the conducted laboratory CBR tests do not explain the good performance of the chloride-stabilized materials in the field.

Costs

Figure 7 presents relative costs of one kilometer of chloride-stabilized road and one kilometer of different TBS commonly used in Chile. As explained above, relative costs, instead of absolute costs, are used to avoid the effect of currency. The relative cost of chloride-stabilized roads was set to 1.0. Results show that the construction cost of chloride-stabilized roads is approximately one-third of the TBS cost per kilometer. TBS costs range from a relative cost of 2.3 to 4.2, depending on the quality of the selected treatment for construction.

TABLE 1 Laboratory Results of Unbound and Stabilized Granular Materials

Hexahydrated Magnesium Chloride (kg/m ³)	Sodium Chloride (kg/m ³)	CBR (%)
0	0	86
80	0	76
0	80	82

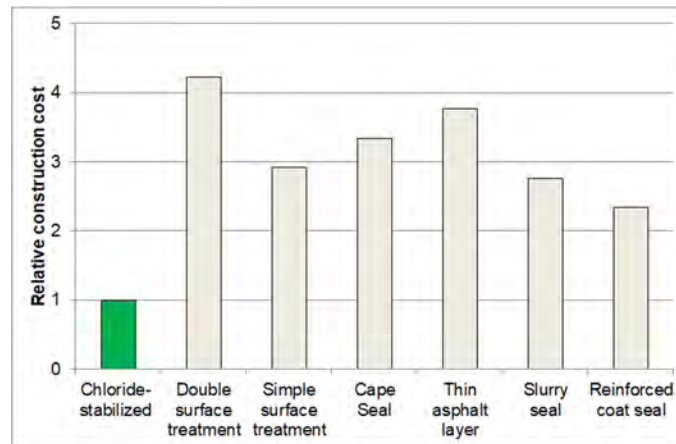


FIGURE 7 Relative construction costs of chloride-stabilized roads and different types of TBS roads.

In addition to construction cost analysis, maintenance costs data were collected from the Ministry of Public Works from LVRs with an average annual daily traffic of less than 400 vehicles. In the analysis, maintenance activities like motor grade reshaping, surface spraying with water or chloride brine, pothole patching, and crack sealing were included. The present value of all the costs related to maintenance were calculated for two and four years (Figure 8). Again, the chloride-stabilized maintenance cost was set to 1.0, and the maintenance cost of TBS was calculated as a relative cost. Results show that for the traffic level studied, chloride-stabilized roads have a maintenance cost higher in approximately 8% than TBS roads. Unfortunately only generic costs of TBS roads were available instead of detailed costs for different TBS types.

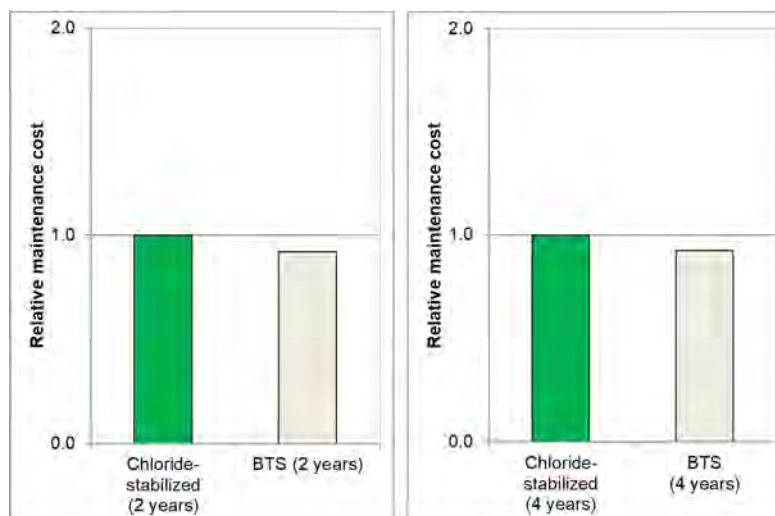


FIGURE 8 Relative maintenance costs of chloride-stabilized roads and different types of TBS roads.

CONCLUSIONS

The comparison of indicators, characteristics, and costs for CSR and TBS leads to the following conclusions:

- CSR presents higher values of IRI than TBS during the analyzed period, having no significance difference at the beginning of the analyzed period and at age of 6 years. Additionally, CSR IRI values present higher variability than TBS IRI values when the samples were analyzed by age of construction.
- Overall condition indexes present values slightly higher for TBS than CSR, but this tendency is not clear, and both alternatives present variability per year of analysis at different ages for each type of material. However, analysis with more sections is needed to obtain stronger conclusions about the comparison of the overall condition index for these types of LVR.
- CBR for unbound materials and CSR samples present similar values, meaning that the mechanical behavior should be similar; however, the empirical behavior of CSR in the field shows better performance over time than unbound granular materials. This may be due to maintaining the cohesion between aggregates for longer periods and preventing the aggregate loss.
- The costs per kilometer for TBS roads are 2.3 to 4.2 times higher than for CSR, considering different alternatives of TBS with a general CSR. Maintenance costs for CSR are 8% higher than TBS.

Considering these conclusions, a higher variability of CRS in terms of IRI will affect the ride quality and comfort of users, as well as the cost users; but if the condition is analyzed considering other distresses, as the included in the overall condition indexes, both alternatives presented similar variability. Furthermore, the difference in construction costs is high and important when the decision about which alternative should be used, but maintenance costs showed little difference. Therefore, the decision between CSR and TBS roads should be made considering the service conditions in terms of climate and traffic demand, considering that CSR could present more variability over time comparing with TBS, but could be a cost-effective alternative for LVR, which was not included in the scope of this paper.

Further analysis of life-cycle cost is required to study in detail the costs and benefits of both alternatives and find the best scenarios when each alternative is better to apply.

ACKNOWLEDGMENTS

The research team acknowledges Fondecyt/Conicyt for funding the project Fondecyt 11121535 “Development of Performance Models for Network Level Management of Sealed Rural Roads.” Special acknowledgments to the Department of the Ministry of Public Works who support the project and companies Sal Lobos and Salmag. Support from the National Research Center for Integrated Natural Disaster Management CONICYT/FONDAP/15110017 is sincerely appreciated.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: study conception and design, Alelí Osorio-Lird and Alondra Chamorro; data collection, Alelí Osorio-Lird and Alvaro González; analysis and interpretation of results, Alelí Osorio-Lird and Alvaro González; draft manuscript preparation, Alelí Osorio-Lird, Álvaro González, and Alondra Chamorro. All authors reviewed the results and approved the final version of the manuscript.

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Assessing Bio-Based Fog Seal for Asphalt Pavement Preservation

BO YANG

YANG ZHANG

HALIL CEYLAN

SUNGHWAN KIM

Iowa State University

INTRODUCTION

All types of roads, including those with asphalt pavements, steadily deteriorate over time because of repeated mechanical (traffic) and climatic loadings. Pavement preservation consists of applying a suitable treatment on deteriorated roads to maintain good conditions and extend their service lives (1, 2). Fog seal is a low-cost application of liquid asphalt or emulsion derived from petroleum or coal tar to slow down microcracking propagation, prevent oxidation, and seal against water infiltration. The conventional fog sealants need heating before spraying on the pavement surface, and the recommended spray temperature should be between 52°C and 71°C (125°F and 160°F). Although such petroleum-based traditional fog sealers have been successfully used to maintain road surfaces for many years, they not only need a long curing time, which results in delayed traffic opening, but they can also cause health issues from chemical components such as polycyclic aromatic hydrocarbons (3, 4). Furthermore, the use of fossil fuel-based products increases the risks associated with an energy crisis and environmental contamination (5, 6).

In recent years, a few bio-based fog sealers have been developed as sustainable alternatives to traditional petroleum-based sealers; soy-based fog sealant derived from agricultural oil is one such product. The manufacturers of the bio-sealant claim that it protects asphalt from oxidation, potholing, edge rutting, and cracking and can extend the life of paved asphalt surfaces when applied every 3–5 years (7); the other advantages and disadvantages are summarized in Table 1. States such as Missouri and Ohio have reported success in using bio-based products for county road preventive maintenance (7, 8). Even though the reported observations include quick shedding of water from roadways treated with bio-sealant while retaining the skid resistance of normal pavement, documentation of construction and performance experience is limited.

Based on the successful use of bio-sealant in other states, this study aimed at evaluating a bio-based product as a fog sealant for low-volume asphalt pavements in Iowa. With the intent of checking the effect of such bio-sealant on skid resistance, pavement-marking retroreflectivity, water absorption, and permeability, the construction process and consequent field and laboratory investigations based on varied sealant spray rates over a 2-year period were documented.

TABLE 1 Benefits and Limitations of Using Bio-Sealant for Fog Seal (7, 8)

Benefits of Using Bio-Sealant	Limitations of Using Bio-Sealant
<ul style="list-style-type: none"> • Resistance to deterioration <ul style="list-style-type: none"> – 3–5 additional years of service life. – Reduces oxidation. – Penetrates deep into asphalt. – Adding polymers to the asphalt cement. • Improvements to surface <ul style="list-style-type: none"> – Seals hairline cracks. – Helps maintain skid resistance. – Reduces moisture penetration. – Reduces potholing and edge rutting. • Financial considerations <ul style="list-style-type: none"> – Does not affect line stripping. – Is not removed by snowplowing. – No heating, carbon negative. – Reduces life-cycle costs. 	<ul style="list-style-type: none"> • If a road is in good shape, bio-sealant should be applied every 4 to 5 years. If it is in fair shape, it should be applied every 2 to 3 years, as long as the road is not ravelling. If the road has alligator cracking, bio-sealant cannot repair the damage and should not be used. • Applying bio-sealant calls for dry conditions and a dry road with temperatures above 40°F. Bio-sealant should never be applied in wet or freezing conditions.

CONSTRUCTION AND EXPERIMENTAL APPROACHES

Bio-Based Fog Sealant

The soy-based bio-sealant used in this study is a black liquid with a nondescript slightly citrus odor. This product has a viscosity of 5 to 20 s at room temperature, similar to the flowability of water, and no heating needed before application. It is 88% bio-based, with 40% obtained from soybean oil. By making use of agricultural and recycled materials, this bio-sealant is a nontoxic and environmentally friendly alternative to petroleum-based sealing agents with competitive price. It contains some polymers and common admixtures in traditional asphalt emulsion used to improve pavement flexibility under colder conditions.

The typical spray rate of bio-sealant can vary from 0.045 to 0.091 l/m² (0.01 to 0.02 gal/yd²), which is lower than for the rate of traditional fog sealant [0.45 to 0.82 l/m² (0.1 to 0.18 gal/yd²)] (9–14). The relatively low rate of bio-sealant is due to its good followability. Bio-sealant can reduce not only the need to use petroleum-based products in pavement maintenance, but also the need for using bitumen in the manufacturing of new asphalt by causing the road surface to last longer.

Site Installation

The sites selected for bio-sealant installation were located near Toronto in Clinton County, IA, and included a 4,506-m (2.8-mi) section of asphalt-surfaced road in E63/Y32 and an 805-m- (0.5-mi-) long asphalt-surfaced section through the City of Toronto. These sections were part of a two-lane low-volume road. Each lane was 3.05 m (10 ft) wide with a 0.91-m- (3-ft-) wide sand-paved shoulder on each side. The test sections at the installation site were divided into five subsections and are shown in Table 2. These three spray rates, which are different from the typical rates, were selected to investigate the effects of high rate on the pavement surface.

TABLE 2 Construction Information About Bio-Sealant Installation

Section	Length, m (ft)	Spray rate, l/m ² (gal/yd ²)
Control section (CS)	30.5 (100)	0 (0)
Treated section No. 1 (TS 1)	305 (1,000)	0.136 (0.03)
Treated section No. 2 (TS 2)	305 (1,000)	0.113 (0.025)
Treated section No. 3 (TS 3)	305 (1,000)	0.091 (0.02)
Remaining section (RS)	4,366 (14,324)	0.091 (0.02)

The application of bio-sealant in Clinton County, IA, began on June 29, 2016, during dry and clear weather. Before application, all road surfaces were swept and cleaned, and the boundary marking lines for each section were painted. A vehicle equipped with an automatic spray machine was used to spray the bio-sealant. During application, the vehicle speed typically ranged from 8 to 16 km/h (5 to 10 mph).

After the application, the bio-sealant-treated lane exhibited a darker color than the untreated lane; this difference in appearance disappeared after a few days. During construction, the pavement marking was applied along with the bio-sealant materials, but no obvious reduction in visibility of the marking was observed. Additionally, the bio-sealant-treated section did not exhibit free liquid standing on its surfaces, indicating that the bio-sealant could be quickly absorbed by the pavement surface because of its natural properties. Based on this characteristic, a bio-sealant-treated road can be opened to traffic within 30 min after application, somewhat more rapidly than when applying traditional fog sealers (3). In summary, the documented construction process showed that the application of bio-sealant is easy to perform and does not require extra energy for heating of the sealant, and the treated road section can be opened to traffic quickly. From these perspectives, it is a cost-effective technology.

EXPERIMENTAL PLAN

Field Investigations

To document the performance of bio-sealant-treated roads, several field visits were conducted to measure retroreflectivity of pavement marking and surface friction of pavement, including skid number (SN) and British pendulum number (BPN) for the bio-sealant installation site within the first 2 years after application. These parameters play important roles in safe driving (15). In this study, a retroreflectometer, a locked wheel skid tester, and a BP tester were used to measure the retroreflectivity, SN, and BPN, respectively, by following related specifications (16–18).

Laboratory Testing

To perform the laboratory testing for hot-mix asphalt (HMA) specimens, four cores with a 10.16-cm (4-in.) diameter were taken from the CS, TS 1, TS 2, and TS 3 in the site in 2017, and the same number of cores were taken in 2018. All HMA specimens were brought to the laboratory and sawed into 5.08 cm (2 in.) thickness, and then were oven-dried at 52°C (125°F) to obtain the constant mass for water absorption and air permeability measurements. ASTM D2726 was

followed to measure water absorption, and an air chamber device was used to measure air permeability (19).

RESULTS AND DISCUSSION

The results of retroreflectivity and friction are shown in Figure 1. For measurements of retroreflectivity, the results indicated that the application of bio-sealant did not cause a significant reduction of retroreflectivity at the early stage after application (Figures 1a and 1b).

For measurements of SNs, the eastbound lane (Figure 1c) and the westbound lane (Figure 1d) exhibited significant decreases in skid resistance within the first week after application. After several months (between July 2016 and May 2017), the skid resistance was restored to its original condition. The measured BPN values from May 2017 to March 2018 indicate that surface friction was in a stable stage. Decreased surface friction is because of the filling in of the pavement surface texture by fog sealant (20, 21). With continuous tire wear, the fog sealants were worn away from the surface, resulting in restoration in friction (22). In consideration of the average SN drop from 63 to 49, the minimum SN value before application of bio-fog seal must

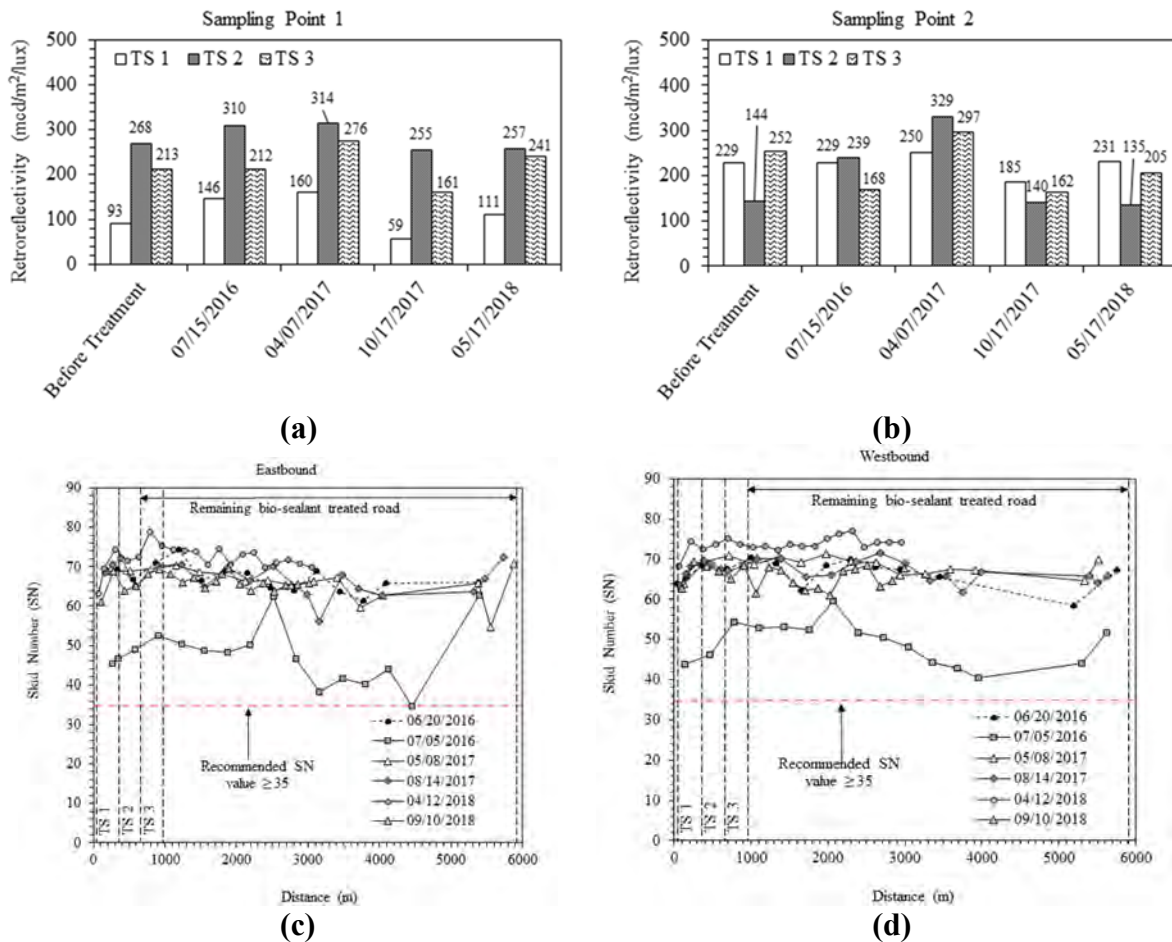


FIGURE 1 Results of (a) retroreflectivity of point 1; (b) retroreflectivity of point 2; (c) SN of eastbound; and (d) SN of westbound.

be 50 to avoid the SN dropping below the minimum requirement of 35 (23). The friction results indicated that the application of bio-sealant could lead to a reduction in surface friction at an early stage, although after several months the friction could be restored.

The results from water absorption and air permeability tests shown in Figure 2a and Figure 2b, respectively, reflect the lower rate of water absorption and air permeability in TS 1 and TS 2 compared to that in specimens from the control section. The thickest/highest rate of application resulted in the lowest rate of water absorption and air permeability for specimens taken from the first year (2017) and the second year (2018), reflecting the greater void-filling in bio-sealant-treated specimens. From the perspective of pavement preservation, lower water absorption and permeability are desirable since they can prevent water infiltration into pavement structures and thereby minimize damage caused by seasonal variations such as freeze–thaw cycles.

CONCLUSIONS

Traditional petroleum-based fog sealers have been successfully used for many years, while alternative nontraditional fog sealers derived from agricultural matter, which have more cost-effective and environmentally friendly potentials, have not yet been properly investigated. In this study, current practice in the use of fog seal was reviewed and summarized. Additionally, a bio-based fog sealer was applied to a selected asphalt pavement section at various spray rates over a 2-year evaluation interval. The detailed construction procedures were documented, and the key findings from field investigations and laboratory tests can be summarized as follows:

- Retroreflectivity of pavement marking decreased immediately after fog seal application using bio-sealant, but was restored to its preapplication level in 2 weeks.
- While a short-term decrease in friction was observed after bio-sealant application, friction requirements were met throughout and returned to their original levels within 11 months.
- The minimum SN value of a road before application of bio-fog seal must be 50.

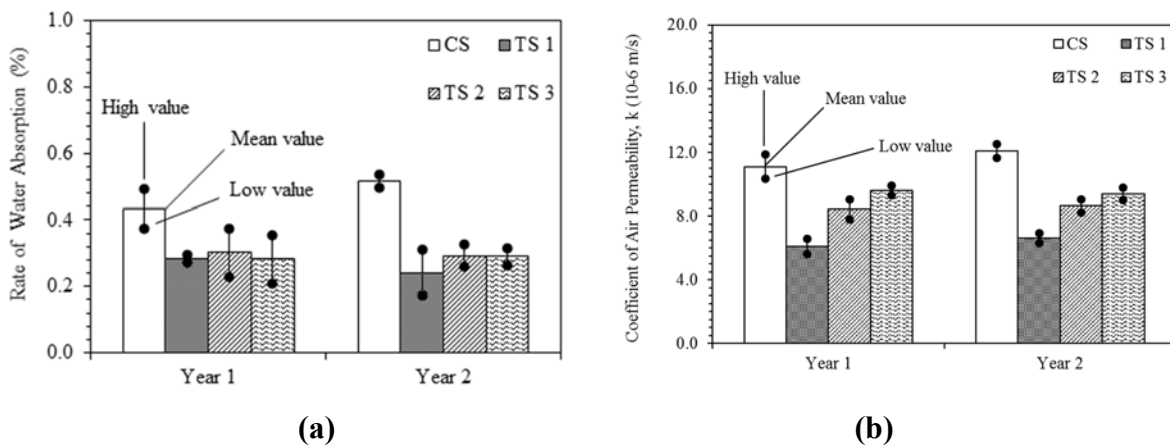


FIGURE 2 Test results of (a) water absorption and (b) air permeability (1 m/s = 3.28 ft/s).

- Laboratory results indicate that specimens treated with a higher bio-sealant spray rate are associated with lower water absorption and permeability.
- If the permeability is a critical issue in some roads, the highest bio-sealant spray rate of 0.136 l/m² (0.030 gal/yd²) is practically applicable based on field and laboratory performance test results. The middle level of 0.113 l/m² (0.025 gal/yd²) is also acceptable based on financial consideration.
- The construction of bio-sealant without heating process is rapid and cost-effective and needs only 30 min to open traffic. However, the dropped surface friction could be a concern to roads that have a low SN before application.
- Although the 2-year evaluation indicated that bio-sealant could seal voids in the pavement and resulting negative impact on retroreflectivity and friction could be restored, their function on friction maintenance should be evaluated in the following years.

ACKNOWLEDGMENTS

The authors are grateful to Roger Boulet, Chris Brakke, Khyle Clute, Kent Ellis, Vanessa Goetz, Kevin Jones, Jason Omundson, Jeffrey Schmitt, Jim Schnoebelen, Fred Thiede, Francis Today, Danny Waid, and Bob Younie from Iowa DOT, Todd Kinney from Clinton County, and Mark McCulloh and Doug Standerwick from Bargaen, Inc.

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**Role of Preventive Treatments in
Low-Volume Road Maintenance Program**
A Full-Scale Case Study

FARHANG JALALI
ADRIANA VARGAS-NORDBECK
MOSTAFA NAKHAEI

National Center for Asphalt Technology at Auburn University

The lack of information on reliable life-extending benefit for pavement preservation treatments led the National Center for Asphalt Technology to conduct a preservation study applying preservation treatments on a low-volume county road in Auburn, Alabama, which began in 2012. The primary objective of the study is to develop performance models for preservation treatments to be used in pavement management systems. Performance measures—including rutting, cracking, and roughness—are collected on a regular basis to the present date. Among all the collected measures, only cracking shows an upward trend and therefore was used in this study as the performance index for the analysis. The present study aims to implement a semi-parametric survival analysis technique to assess the effectiveness of treatments used, and to investigate associated risk factors. The study confirms that preservation strategies significantly decrease the risk of failure when compared with a “do-nothing” scenario. It was found that initial condition, treatment family, recycled material usage, and crack sealing application have a significant impact on future deterioration. It is also concluded that survival analysis techniques are useful tools in aiding decision makers in the selection of proper treatment choices.

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https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

**Effect of Preservation Treatments on the
Structural Performance of Pavements**
*A Study from the National Center for Asphalt Technology's
Low-Volume Preservation Sections*

MD RAHMAN

ADRIANA VARGAS-NORDCBECK

National Center for Asphalt Technology at Auburn University

INTRODUCTION

Pavement preservation is a cost-effective strategy that can extend pavement life and improve functional condition. This proactive approach has been adopted by many agencies as an alternative for maintaining road networks with limited resources. The benefits of applying pavement preservation treatments to good-candidate sections have been documented mainly in terms of surface distress and ride quality; however, there have been limited studies that focus on the effect of these treatments on the structural performance of the pavements.

The National Center for Asphalt Technology at Auburn University has 25 low-traffic-volume preservation test sections, including two control sections, on Lee County Road 159 in Auburn, AL. The treatment sections include standalone treatments and a combination of treatments with multiple applications. At the time of construction, the road had been in service for 14 years, and the treatments were applied without any structural condition restoration activity. At one end of the test site, there is a quarry and asphalt plant that provides an accurate number of truck traffic data collected from the weigh station at that facility. The sections were constructed in the summer of 2012 and have been in service approximately 6.5 years without any additional rehabilitation or maintenance works. Along with the functional condition, the structural condition of the pavement section is being monitored periodically over time. A falling weight deflectometer (FWD) has been used to measure the deflection basin parameters at specific locations of each treatment section, all of which were randomly selected at the beginning of the study.

The objective of this study is to investigate the contribution of pavement preservation treatments for maintaining the structural condition of the pavement. The performance data collected over time are also used to forecast the future performance of the treated pavement and measure the contribution of the treatments to the life extension of the pavement sections.

BACKGROUND

The benefits of pavement preservation treatments have typically been studied in terms of the extension in service life imparted to the existing pavement. Peshkin et al. (1) developed guidelines for the selection of preservation treatments of high-traffic-volume roadways based on

traffic levels, pavement condition, climate, work-zone restrictions, expected performance, and costs. The authors noted that treatments normally do not add structural benefit, but some may address or delay the appearance of low-severity fatigue cracking. Arabali et al. (2) proposed a decision matrix for the selection of appropriate preservation treatments. The relative benefit was quantified in terms of change in pavement condition index, which is obtained from visual distress surveys. Other studies have defined the life-extending benefit as the difference in the time required to reach a threshold performance indicator value for treated and untreated sections (3, 4).

Few studies have focused on the long-term structural benefits of applying pavement preservation. Howard (5) studied the structural capacity of pavement sections with seal treatments by evaluating the change in the effective structural number over time. The results showed that chip seal treatments and scrub seal treatments preserved the structural number of the pavement better than the structural number on the control sections, with no appreciable differences between seal types. Ji et al. (6) also performed FWD testing to evaluate the structural adequacy of pavements treated with microsurfacing. The results indicated that microsurfacing can maintain the structural condition of the roadways when applied to suitable candidates.

The present study focuses on quantifying the effect of the treatments over the structural condition indices of the pavement. The base damage index (BDI), the base curvature index (BCI), and the area under pavement profile (AUPP) are the three deflection basin parameters analyzed in this study. BDI, BCI, and AUPP represent the structural condition index of the base, subgrade, and an estimate of strain under the pavement structure, respectively. Horak et al. (7) and Talvik et al. (8) performed extensive studies on the benchmarking of the deflection basin parameters, which have been found representative of the structural condition of the pavement. The individual layer moduli and composite pavement modulus has been calculated over time through backcalculation. Pavement layer moduli backcalculation methodologies developed from Irwin (9), AASHTO (10), and Smith et al. (11) have been adapted for the backcalculation of the moduli in this study. The effective structural number (S_{Neff}) is calculated following the AASHTO (10) equation using the backcalculated composite pavement modulus and pavement thickness profiles collected from ground penetrating radar data.

METHODOLOGY

FWD testing was performed at two locations in each section. At each location, three repetitions at three load levels (6, 9, and 12 kips) were applied. The deflection-measuring geophones were spaced at 0, 8, 12, 18, 24, 36, 48, 60, and 72 in. from the center of the load plate. Measured deflections were standardized to a 9-kip (40-kN) load and corrected to a pavement temperature of 68°F (20°C) before the calculation of the deflection basin parameters. Table 1 shows the deflection basin parameters, the equations, and the value ranges for different extents of damage as recommended by Smith et al. (11).

The prediction and forecasting of the deflection basin parameters were performed using a seasonal autoregressive integrated moving average (ARIMA) with simple exponential smoothing of the forecasted data. The seasonal model can be easily explained in the format of ARIMA (p, d, q) \times (P, D, Q) model with constants, where $p, d,$ and q are the number of autoregressive terms,

TABLE 1 Deflection Basin Parameters and Recommended Ranges (3)

Parameter	Equation	Layer of Significance	Zone	Values
BDI	$BDI = d_{12} - d_{24}$	Base layer	Severe	> 16
			Warning	8 ~ 16
			Good	0 ~ 8
BCI	$BCI = d_{24} - d_{36}$	Subgrade	Severe	> 8
			Warning	4 ~ 8
			Good	0 ~ 4
AUPP	$AUPP = \frac{(5d_0 + 2d_{12} + 2d_{24} + d_{36})}{d_0}$	Surface layer	—	

NOTE: d_0 , d_{12} , d_{24} , and d_{36} are the deflections measured at 0-, 12-, 24-, and 36-in. offset from the center of the load plate for standard 9-kips equivalent impulse loading at 68°F.

number of nonseasonal differences needed for stationarity, and number of lagged forecast errors, respectively. P , D , and Q are the same parameters as p , d , and q , but they are used in those models where the seasonal effect is very prominent. Statistical analysis software (SAS) was used for the forecasting. To investigate the time and treatment effect on the deflection basin parameters, the longitudinal data analysis method was used with mixed effect modeling with SAS at a 0.05 significance level. The following treatments were included in this study: chip seals (single, double, and triple layer; single layer with crack sealing), microsurfacing (single and double layer, single layer with crack sealing), and thinlays (PG 67-22, PG 67-22 on foamed recycled base, PG 76-22).

RESULTS

A 120-months BDI projection has been performed based on the existing 67 months of data where all the sections have a lower BDI value than the control section. Figure 1 shows the BDI values over time. The 67th month represents the current condition, while values extending from that point to 120 months were projected using the methods described in the methodology section.

Figure 1 shows that; the control section has the highest BDI at the current time. The untreated section is projected to yield a BDI higher than 8 mils (~ 200 microns) at approximately 83 months of service, which means maintenance or rehabilitation work will be required at that time according to Smith et al. (11). All treated sections, with the exception of the single-layer microsurfacing with crack sealing, would be in good condition even at 120 months of service. The plot shows a brief summary of the base layer condition over time for each type of treatment.

BCI is the measurement of damage in the subgrade. The lower the BCI value is, the better the subgrade can spread the load to the soil. As per FHWA, Smith et al. (11) mentions an optional requirement of maintenance and repair of the subgrade layer if BCI values rise higher than 4 mils.

AUPP, being the representative measurement of strains under pavement layer, can help modeling of rutting and fatigue cracking. Kim and Park (12) developed a model to measure

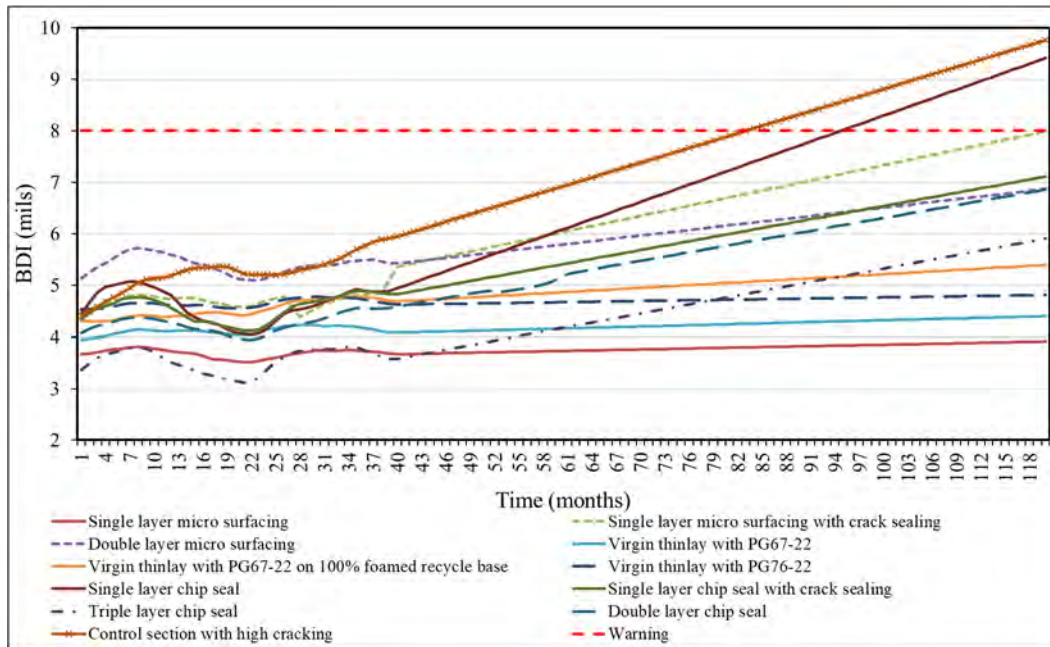


FIGURE 1 BDI for different treatments and projection up to 120 months.

tensile strain at the bottom of the asphalt concrete layer (ϵ_{ac}) constructed over aggregate base:

$$\log(\epsilon_{ac}) = 1.034 \log(AUPP) + 0.932.$$

Using AASHTO’s 1993 equation for composite pavement modulus (E_p) and layer thicknesses collected from ground penetrating radar profiles, the effective structural number (S_{Neff}) was calculated over time. Using the ARIMA forecasting model, the S_{Neff} was projected up to 10 years of service. After 10 years of service, most treated sections have an effective structural number higher than the control section. Considering $a_1 = 0.44$, the overlay thickness can be saved up to a minimum equivalent thickness of 2.25 in. (57 mm). Figure 2 shows the difference between the S_{Neff} value of different treatments and control section, measured at 6.5 years and predicted after 10 years of service. The positive values signify the better structural condition of the treated section than the control section while the negative value signify the vice versa.

Longitudinal data collected over the time of service for the BDI and BCI parameters were tested to observe the treatment, time, and combined effect performing mixed modeling at $\alpha = 0.05$ significance level with an assumption of time being a discrete variable. The p -values for the varied wheel paths and traffic directions were compared for the parameters. BCI values, which represent the subgrade structural condition, showed a strong time effect for the chip seal sections. The thinlay sections showed a strong treatment effect and combined effect (time \times treatment) for the BCI values. The BDI values for the thinlays showed a significant treatment effect for the thinlay sections. Table 2 shows the comparison between control and treated sections for different parameters at the current time of 67 months and a projection at 120 months. The parameters being considered are BDI, BCI, strain at the bottom of asphalt concrete (AC) layer (ϵ_{ac}), and effective structural number (S_{Neff}). The values show the difference of the parameters between

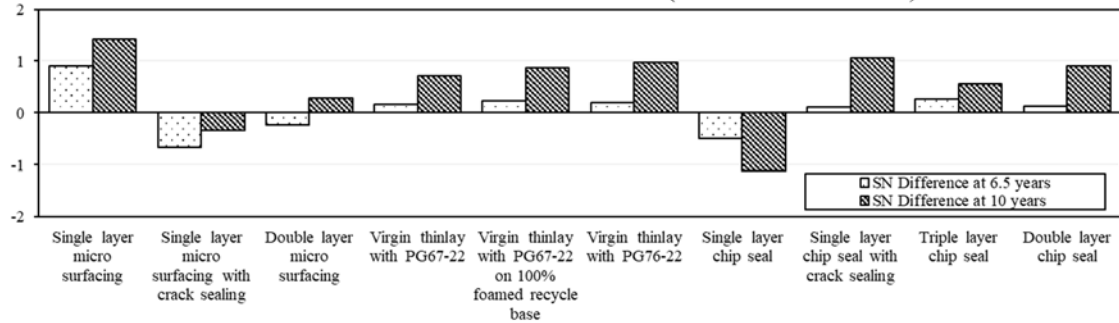


FIGURE 2 SNeff differences of treated sections to control sections (Treated SNeff, Control SNeff).

TABLE 2 Difference Between Parameters (Control, Treatment)

Treatment Name	Differences (Δ = Measured Parameter at Control Section - Measured Parameter at Treated Section)							
	Current Difference @ 67 months				Projected Difference @ 120 months			
	BDI	BCI	ϵ_{ac}	SNeff	BDI	BCI	ϵ_{AC}	SNeff
Single-layer microsurfacing	3.21	1.20	2.88	-0.90	5.85	2.48	5.49	-1.41
Single-layer microsurfacing with crack sealing	0.90	1.00	5.88	0.66	1.78	2.05	8.89	0.35
Double-layer microsurfacing	1.15	1.06	4.94	0.23	2.89	2.41	7.39	-0.28
Virgin thinlay with PG 67-22	2.79	0.87	2.73	-0.16	5.36	2.11	6.56	-0.72
Virgin thinlay with PG 67-22 on 100% foamed recycle base	2.08	0.67	2.14	-0.22	4.37	1.88	5.05	-0.87
Virgin thinlay with PG 76-22	2.28	0.76	2.34	-0.20	4.96	2.03	4.80	-0.98
Single-layer chip seal	0.84	0.58	2.64	0.50	0.35	0.98	4.88	1.13
Single-layer chip seal with crack sealing	1.52	0.86	2.70	-0.11	2.65	1.77	4.07	-1.05
Triple-layer chip seal	2.76	0.57	-1.32	-0.27	3.86	0.40	-7.16	-0.56
Double-layer chip seal	1.74	1.14	3.45	-0.12	2.90	2.17	4.74	-0.90

the control section and the treated section. Positive BDI, BCI, and ϵ_{ac} values as well as the negative SNeff signify a better performance of the treated sections, while the vice versa means a worse condition of the treated section than the untreated control section.

CONCLUSIONS

In this study, the structural performance of pavement preservation treatments was evaluated based on the corresponding deflection basin parameters for individual pavement layers. The selected treatments—chip seals, microsurfacing, and thinlays—were found to have a statistically significant effect on the structural performance of the pavement. Single-layer chip seal, single-

layer microsurfacing with crack sealing, single-layer chip seal with crack sealing, double-layer chip seal, and triple-layer chip seal were the treatments found to have the biggest contribution in delaying the structural deterioration of the base layer and extend the life of the base layer up to 109 months, depending on the treatment applied. Single- and triple-layer chip seals can extend subgrade life up to 73 and 15 months, respectively. All the treatments discussed in this study show the reduction of tensile strain at the bottom of AC layer up to 10 microstrains after 10 years of service compared to the control sections. The S_{Neff} values for the single-layer microsurfacing, chip seals, and all thinlays can save a minimum overlay thickness of 2.25 in. at the end of 10 years of service. Data continue to be collected for this ongoing study with the intent of improving the developed performance models.

ACKNOWLEDGMENTS

The authors thank the Departments of Transportation of Alabama, Colorado, Georgia, Illinois, Kentucky, Michigan, Minnesota, Mississippi, Missouri, New York, South Carolina, Oklahoma, Tennessee and Wisconsin, and FP² Inc. for their sponsorship of this project.

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Geotechnology

GEOTECHNOLOGY

Geographic Information System Semi-Automated Landslide Inventory and Assessment Tool

TERESA D. MORALES

Forest Service Regional Office, Missoula, Montana

INTRODUCTION

The intent of this presentation is a step-by-step analysis through the GIS Semi-Automated Landslide Inventory Tool developed by OSU for the USFS. The tool was developed by Ben Leschchinsky and Michael Olson of Oregon State University to assist U.S. Department of Transportation and land management agencies with conducting landslide inventories across large landscapes. The tool uses lidar or U.S. Geological Survey (USGS) digital elevation models (DEMs) to help identify head scarps from old slides. Once a head scarp has been positively identified and classified, the extents of the landslide are mapped (starting at the head scarp) using a Contour Connecting Method (CCM) algorithm. Since the algorithm is based in a GIS environment, the outputs from the analysis can be overlain with other datasets such as a road network, popular recreational areas, sensitive habitat, etc. Using this analysis in the context of pertinent datasets allows land managers, specialists, and line officers to make informed management decisions regarding landslide hazards.

METHODOLOGY

As noted above the tool uses lidar or USGS DEMs. The resolution of the raster data is reduced to avoid misclassifying potential head scarp candidates. Selection of appropriate resolution is user identified. Once the appropriate resolution is determined slope raster and hillshade derivatives are generated to aid in visual classification and verification. The analysis then identifies locations of convexity and concavity (rough indicators of head scarps and deposits) across the landscape. Locations of concavity and convexity are then compared to potential rock outcroppings and stream crossings. All potential rock outcroppings and stream crossings are removed from consideration. Remaining areas of convexity are identified as potential head scarp candidates. The tool then generates head scarps from these candidates. Candidates can also be removed or added as deemed appropriate by the user. Once candidates are positively identified, the CCM algorithm is used to generate the extents of the landslide initiated from the head scarp.

The tool may generate multiple potential landslides. The results should be closely evaluated and verified against field investigations.

FINDINGS

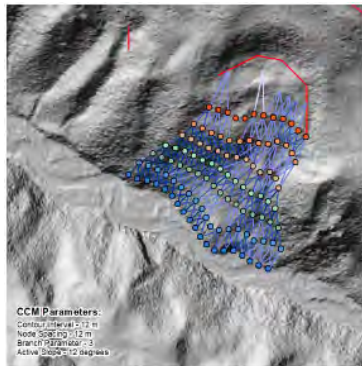


FIGURE 1 Example of CCM output results.

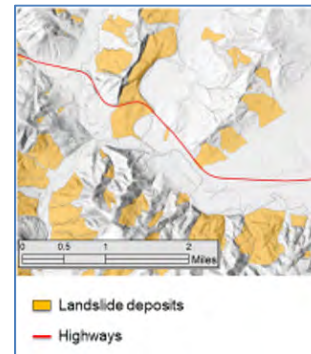


FIGURE 2 Example of inventory results.

CONCLUSIONS

- Tool is well organized and relatively easy to learn.
- LiDAR data performs much better than USGS DEMs
- The tool is useful in doing a coarse inventory of potential landslides. Positive identification of past landslides can be indicative of future landslides (i.e. peak vs. residual strength).
 - The results should be carefully evaluated and field verified by trained professional.
 - Tool is effective in identifying head scarps but evaluation/verification of results can be time consuming.
 - Since analysis is conducted in a GIS platform the output results can be evaluated in the context of other data such as a road network, popular recreation sites, sensitive/critical habitat, etc.

ACKNOWLEDGMENTS

- Ben Leshchinsky, Ph.D, Associate Professor of Forestry, Forest Engineering, Resources & Management, Oregon State University.
- Michael Olson, Ph.D, Associate Professor of Geomatics School of Civil and Construction Engineering, Oregon State University.
- Michael Bunn, Ph.D Candidate, School of Civil and Construction Engineering, Oregon State University.

Cold Regions and Climate Change

Development and Pilot Installation of a Scalable Environmental Sensor Monitoring System for Freeze–Thaw Under Granular-Surfaced Roadways

DERYA GENC
JERAMY C ASHLOCK
BORA CETIN
PAUL KREMER
Iowa State University

The freeze–thaw cycle is one of the major sources of damage to granular-surfaced roadways, especially in areas where the timing of heavy agricultural traffic coincides with that of spring thawing. To help local roads agencies plan better for annual budgets and frost embargos, it is useful to be able to predict the frost depth and number of freeze–thaw cycles under a given roadway based on continually updated weather and soil data. Computational modeling can help in this regard, and may be conducted by collecting data on weather and the thermal and hydraulic properties of the soil—as well as soil temperature, moisture, and suction—and using the data directly in the analyses. In order to obtain accurate field data for model calibrations and predictions, an appropriate sensor network and data acquisition system must be planned and installed. This article details the development and installation procedures for one such system of sensors for subgrade temperature, water content, and matric suction, and presents lessons learned throughout the process. Various issues are discussed relating to selection of the sensor and data acquisition system, laboratory and field checks, borehole-sensor installation tools, and post-installation troubleshooting and monitoring. To ensure a successful installation beneath the granular roadway, laboratory and field trials were first performed. Salient details of a pilot installation in Hamilton County, Iowa, are provided to guide others developing and scaling similar subgrade sensor systems.

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Comparison of Three Degree-Day Models for Application of Spring Load Restrictions on Low-Volume Roads

HEATHER J. MILLER
CHRISTOPHER CABRAL
RICHARD BERG
FROST Associates

MAUREEN A. KESTLER
U.S. Army Corps of Engineers

INTRODUCTION

Major highways and interstates are designed to withstand damage from traffic during spring thaw; however, low-volume roads can be highly susceptible to damage from such springtime traffic. To reduce potential spring-thaw damage, many road management agencies apply spring load restrictions (SLRs), which restrict the allowable load on the road during critical periods when the pavement is most susceptible to damage. Table 1 presents a summary of the approaches used by transportation agencies for SLR timing (1).

The Aurora Consortium is sponsoring a pooled-fund study in which two of the approaches listed in Table 1, Methods 5 and 6, are being evaluated via a field demonstration. Several protocols and/or model predictions are being validated against observed subsurface temperature profiles and measurements of pavement deflection at instrumented sites in five highway jurisdictions [AK, MI, ND, Ontario (ONT), and WI]. This abstract presents interim results from evaluation of three degree-day threshold/protocols (Method 5). Ultimately, results from this research will be used to help understand the reliability, benefits, costs, and risks of these approaches for SLR timing.

METHODOLOGY

Several degree-day models and protocols have been reported in the literature for determining SLR timing. Initial efforts in this regard were by Mahoney et al. (2). Their method has been revised and several variations used since that initial effort. The most notable revision, by Van Deusen et al. (3), is the procedure currently used by Minnesota Department of Transportation (MnDOT) (4, 5). These and several other methods are based on cumulative thawing indices (CTI) calculated as the accumulation of degree-days computed from average daily air temperatures. Degree-days are generally calculated as the difference between the daily average temperature and some reference temperature. The daily air temperatures used in the models are ideally obtained from a weather station at the site, but may also be obtained from a nearby National Oceanic and Atmospheric Administration or National Weather Service weather station, a state-owned road weather information system or similar station, or a combination of sources. These models are relatively

TABLE 1 Methods for Applying Spring Load Restrictions

	Method	Comments
1	Fixed dates based on long-term experience.	Because of year-to-year variations in freezing and thawing, damage could still occur. Or hauling may be restricted because of given date when conditions are fine for hauling without damage.
2	Inspection/observational approaches, where field personnel observe changes in the roadways such as water seepage from cracks, other indicators of pavement distress, or a worst-case scenario of major rutting, cracking, and breaking up of asphalt.	Too late; damage has already occurred.
3	Monitoring changes in pavement load-bearing capacity as indicated by deflections measured in falling weight deflectometer (FWD) tests.	State DOTs typically own one FWD, possibly two, and can test only a sample of roadway segments statewide. Most other road management agencies or municipalities, such as cities and counties, do not own FWDs.
4	Monitoring subsurface temperature and/or moisture profiles beneath roadways.	Excellent for conducting studies, but as a means of monitoring for SLR and winter weight premium placement and removal, site specific.
5	Degree-day thresholds/protocols based upon air temperature data to set SLR and winter weight premium dates.	Can be very simple to use.
6	Predictive models based on atmospheric weather data to predict subsoil temperature profiles.	Can range from somewhat simple to very complex to use requiring complex input. More complex models can cost a fair amount.

simple to apply and can be accomplished using spreadsheets. When coupled with accurate weather forecasts, this approach has an advantage over Methods 2, 3, and 4 (Table 1) because estimates of when to apply the SLR can be provided 5 to 10 days in advance.

Most degree-day methods use a reference temperature other than 32°F, the freezing point of pure bulk water, to consider differences between the air and pavement surface temperatures and increases in incoming shortwave radiation during the spring and early summer. Three degree-day threshold protocols were studied for the Aurora project. The first method, outlined in detail by MnDOT (4, 5) uses a variable reference temperature to account for increased solar gain during the late winter and early spring. The MnDOT protocol recommends placing the SLR when the CTI exceeds 25°F days.

While several researchers have found that the MnDOT protocol and reference temperatures work well for regions located at about the same latitude as Minnesota, other investigators have suggested that local calibration may be required for sites at different latitudes. The second protocol used in this study was developed by researchers at Lakehead University and is described in detail by Pernia et al. (6). This method also uses the MnDOT reference temperatures and equations for computing the CTI, but suggests using site-specific CTI threshold values for applying SLRs, which must be calibrated by determining the CTI values corresponding to the dates when the thaw depth exceeds 12 in. For the ONT site, the Lakehead model was calibrated by Pernia et al. (6) based on several years of data. For the other four

Aurora demonstration sites, the authors used Year 1 data (2014–2015) for calibration and then ran this model in predictive mode for Year 2 of the study (2015–2016). Site-specific CTI threshold values (in units of °F days) were the following:

- AK: 80.5
- MI: 101.1
- ND: 64.2
- ONT: 129.6
- WI: 49.2

The third approach used in this study uses a relatively new model developed by Rajaei et al. (7) for estimating pavement surface temperatures, based on the latitude of the site and measured daily average air temperatures. Rajaei et al. suggested using pavement surface temperatures estimated from their model to compute daily and cumulative thawing indices for use in SLR application. Because this approach accounts for changes in latitude (and thus changes in solar gain during the early spring), a potential advantage of this approach is that it may eliminate the need for local calibration of reference temperatures for CTI calculations. Rajaei et al., however, did not recommend a threshold value to be used for that pavement temperature–based CTI, nor did they correlate thawing index values with thaw depth profiles beneath any of the roadway stations in their study. Therefore, in a complementary study, several investigators conducted research to accomplish the following:

- Validate the Rajaei et al. (7) model over a wider range of geographic regions using measured air and pavement surface temperatures at several roadway sites and
- Determine whether a consistent CTI threshold could be determined that would be appropriate for SLR application for all the sites regardless of location.

Eftekhari et al. (8) studied several sites in Alaska and in the contiguous United States (CONUS) and found that the Rajaei model yields reasonable estimates for pavement surface temperatures over a wide latitudinal range, although they noted that the Rajaei model slightly underpredicted pavement surface temperatures (7). Eftekhari et al. (8) correlated CTI values computed using pavement surface temperatures from the Rajaei model with measured thaw depths and suggested that a CTI threshold value of 30°F days might be appropriate for SLR application (based on a target window between the onset of thaw and the date when the thaw depth exceeded 12 in.). This third approach for SLR application was also evaluated at the Aurora field test sites.

FINDINGS

SLR start dates from the three different degree-day protocols were compared with measured frost and thaw depths. The number of days that the SLR start dates fell early or late relative to a 12-in. thaw depth are summarized in Table 2 and Figure 1. The findings suggest that the protocol based on the Rajaei pavement temperature model generally comes closest to suggesting SLR start dates on or slightly before the dates when the thaw depth reaches a depth of 12 in. In this study,

TABLE 2 Number of Days that Predicted SLR Start Dates Fell Early or Late

	# Days Early (-) or Late (+)		
	MnDOT	Lakehead	Rajaei/Eftekhari
AK 2015	-36		-1
AK 2016	-2	7	
MI 2015	-6		-5
MI 2016	-1	5	0
ND 2015	-3		-1
ND 2016	-3	1	3
ONT 2015	-25	7	-19
ONT 2016	-16	19	-15
WI 2015	-1		0
WI 2016	-6	6	-1
Average	-9.9	7.5	-4.3
Std Dev	12.0	6.1	7.5

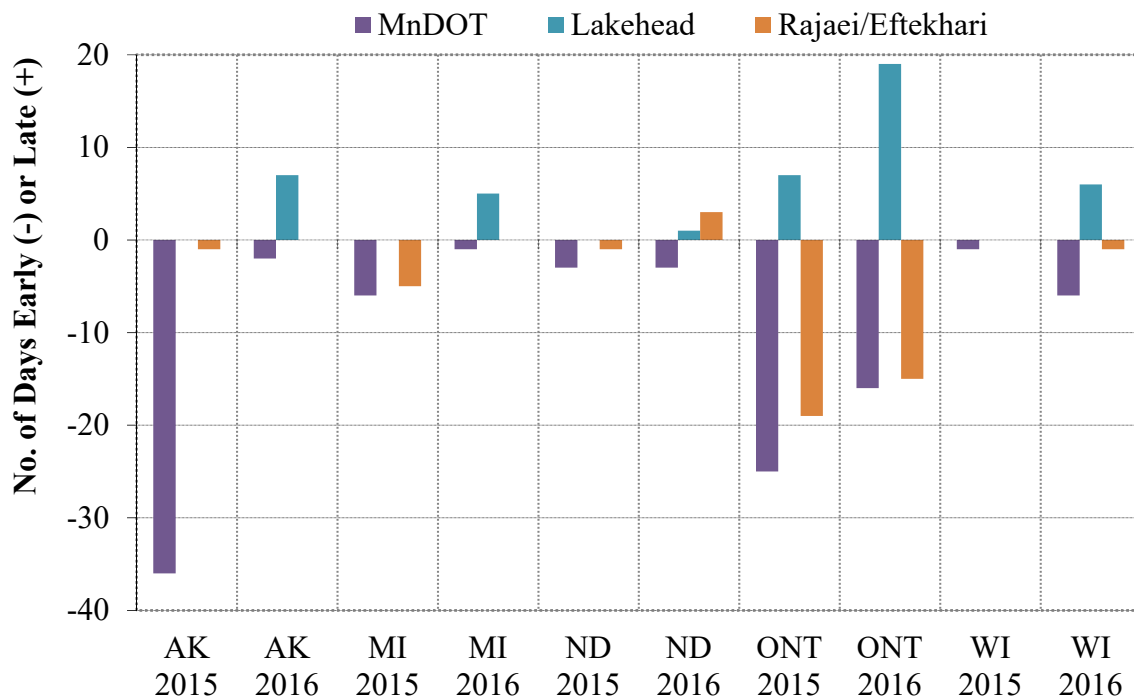


FIGURE 1 Number of days that predicted SLR start dates fell early or late.

the MnDOT method was most conservative, and the Lakehead University protocol was least conservative (with suggested SLR start dates consistently falling after the 12-in. thaw depth was reached).

For the 2015 spring at the Aurora site in Alaska, as well as the more extensive set of data from Alaska sites reported by Eftekhari et al. (8), the new protocol based on the Rajaei pavement temperature model greatly improved SLR predictions compared to the MnDOT protocol. In the Eftekhari et al. (8) study, the MnDOT method predicted SLR start dates before the onset of thaw for 98% of the freeze–thaw season and site combinations in Alaska. This early prediction bias was expected, since the MnDOT reference temperatures were calibrated in Minnesota, where solar gain increases are observed much earlier in the springtime compared to those in Alaska.

In 2016, there were insufficient data available at the Aurora site in Alaska to run the new protocol. For the remaining Aurora sites in CONUS, the new (Rajaei/Eftekhari) protocol provided only modest improvement over the MnDOT method. Interestingly, none of the models performed well for either study year at the Ontario site, suggesting that perhaps there was some site-specific anomaly there.

CONCLUSION

Results of this study suggest that the MnDOT method, as well as a new protocol based on the Rajaei pavement temperature model, both perform well for predicting SLR application dates in CONUS. Both models are slightly conservative and predicted SLR start dates within 6 days before the date when the thaw depth reached 12 in. at the study sites in CONUS. As expected, the new protocol performed better than the MnDOT method at higher latitudes such as Alaska. The Lakehead University protocol was least conservative, with suggested SLR start dates consistently falling after the 12-in. thaw depth was reached at the study sites. Coupled with the fact that the Lakehead method must also be calibrated on a site-specific (or perhaps regional) basis, it is considered not as desirable as the other two degree-day protocols evaluated in this study.

When using any protocol for SLR application, one must keep in mind that variations in cross section, solar radiation, elevation, etc., can vary substantially along the length of any road, and particularly along LVRs. Since one instrumented site is typically used for obtaining air temperatures (and/or other input parameters) for SLR timing protocols, it is important that the site be as representative as possible so that the SLR application date is appropriate for the entire region over which it is applied.

ACKNOWLEDGMENT

This research was supported by Aurora SPR-3(042) Project 2012-05 under a contract with the USDA Forest Service.

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Accuracy Assessment of Satellite-Based Freeze–Thaw Retrievals at Low-Volume Roads in the United States

SIMON G. KRAATZ
JENNIFER M. JACOBS
ESHAN V. DAVE
JO SIAS

University of New Hampshire

HEATHER J. MILLER

University of Massachusetts Dartmouth

Accurate identification of soil freeze or thaw (FT) is important for road management, because it greatly affects a road's load-bearing capacity. Despite low-volume roads (LVRs) being more susceptible to damage because of FT transitions compared with high-volume roadways, relatively few LVRs are monitored via temperature data probes (TDP). Frequent and global space-borne retrievals of soil FT states may be valuable to fill this observational gap. The NASA's Soil Moisture Active Passive (SMAP) instrument provides FT retrievals up to twice a day, approximately corresponding to the top 0 to 10 cm of soils. This study compares SMAP FT data to TDP data at LVRs located in the contiguous United States (CONUS) and Alaska using hourly data obtained from the Meteorological Assimilation Data Ingest System for the 2016, 2017, and 2018 winters. Overall, SMAP FT retrievals show promise in distinguishing between cold and warm roads. For all cases, the median road temperatures corresponding to SMAP frozen and thawed retrievals were clearly below or above 0°C, respectively. SMAP 6:00 p.m. observations perform better than the 6:00 a.m. observations with overall accuracies of 76% in CONUS and 81% Alaska. However, SMAP's accuracy for frozen conditions is below 50% in CONUS indicating that SMAP has a warm bias compared with the TDP sites. These preliminary results suggest that the SMAP FT states have potential value for road management.

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When Should Agencies Post Weight Limits on Local Highways in New York State?

DAVID P. ORR
Cornell University

Most highways in New York State are maintained by local agencies, so decisions about operations and maintenance for low-volume roads are critical. Local highway officials use existing tools and methods to manage their highway networks. Posting of weight limits is done for a variety of reasons such as structural limits on bridges, setting of truck routes through a community, and seasonal posting. A simple low-volume pavement model was developed using with the Cornell Pavement Frost Model to review various approaches to seasonal posting of roads and streets. Using a simple model for timing of seasonal posting can be a very effective tool for a local highway agency. The Cornell Local Roads Program developed such a spreadsheet tool currently being tested by local towns in New York. The tool only requires a simple pavement model and average air temperature data.

To view this paper in its entirety, please visit:
https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

Construction of Chemically and Mechanically Stabilized Test Sections to Reduce Freeze–Thaw Damage of Granular Roads

JERAMY C. ASHLOCK

YIJUN WU

BORA CETIN

HALIL CEYLAN

Iowa State University

CHENG LI

Changan University

INTRODUCTION

Granular-surfaced roads in seasonally cold regions regularly experience damage and degradation due to freeze–thaw cycles and steadily increasing traffic loads. Repair and maintenance of such roads can consume significant portions of budgets from counties and secondary roads departments. Several approaches are used to combat these types of moisture-related damage, including

- Temporarily spreading rock on the affected areas;
- Lowering or improving drainage ditches;
- Tiling;
- Bridging the areas with stone and geosynthetics covered by a top course of aggregate or gravel;
- Coring boreholes and filling with calcium chloride to melt lenses and provide drainage; and
- Regrading the crown to a slope of 4% to 6% to maximize spring drainage.

However, most of these solutions are aimed at dealing with frost boils after they occur.

To help prevent or minimize the occurrence of freeze–thaw damage-related problems in the first place, the Iowa Highway Research Board (IHRB) has supported several previous and ongoing research projects. In the previous Phase II IHRB Project TR-664: Low-Cost Rural Surface Alternatives: Demonstration Project (I), several stabilization methods were implemented over 17 test sections in Hamilton County, Iowa. Data was collected on construction and maintenance costs, and extensive laboratory tests, field tests, and field photographic surveys were conducted. The most-effective and economical stabilization methods for the soil and climate conditions of Iowa were identified, with several of the methods greatly improving the longevity and performance of the roadway materials.

For the ongoing IHRB Project TR-721: Low-Cost Rural Surface Alternatives Phase III: Demonstration Project additional mechanical and chemical stabilization methods were used to construct a total of 33 additional test sections in four counties distributed geographically around the state of Iowa in August through October 2018. Through the upcoming winter and spring

seasons, the test section performance will be documented using extensive field tests and surveys. The goal of the ongoing project is to assess the effectiveness and relative costs of the additional stabilization methods for improving performance and minimizing freeze–thaw damage, under a wider range of climate conditions, subgrade types, and aggregate sources. To aid other stakeholders interested in using the stabilization methods, this paper details some of the construction equipment and methods used to build the 33 test sections.

METHODOLOGY

The stabilization methods used in the current project include five chemical stabilization methods in Washington and Hamilton counties and six mechanical stabilization methods in Howard and Cherokee counties (Table 1). Additionally, two of the mechanical methods (optimized gradation with clay slurry and aggregate columns) were also used in Washington and Hamilton counties, as the Technical Advisory Committee was particularly interested in assessing the performance of these two economical methods in all counties.

To design the test sections, extensive laboratory tests were conducted including sieve and hydrometer, Atterberg limits, slaking, California bearing ratio, proctor compaction, and unconfined compressive strength tests on soil samples that were taken from the test sites and mixed with the stabilizers when appropriate. After construction of the test sections, field tests including dynamic cone penetrometer, falling weight deflectometer, light-

TABLE 1 Types and Locations of the 33 Field Test Sections Used in This Study

Stabilization Method		Counties			
		Howard	Cherokee	Washington	Hamilton
None (control section)		X	X	X	X
Mechanical	Aggregate columns (2)	X	X	X	X
	Optimized gradation with clay slurry (3)	X	X	X	X
	Ground tire rubber (eliminated)	X			
	Recycled asphalt pavement mixed 50/50 with aggregate (4)	X	X		
	2-in. slag surface above 2-in. existing aggregate base (5) (two slag sources)	X+X	X+X		
	4-in. slag surface (5) (two slag sources)	X+X	X+X		
Chemical	12 in. Type I/II cement-treated subgrade			X	
	4 in. Type I/II cement-treated aggregate surface course (6)			X	X*
	Silicic acid, sodium salt concentrated liquid stabilizer (SASS-CLS)			X	X
	Non-ionic concentrated liquid stabilizer with neutral pH (NI-CLS)			X	X
	Ionic concentrated liquid stabilizer (I-CLS)			X	X

NOTE: X = Section constructed in this county. X* = Section will be constructed in this county.

weight deflectometer (LWD), nuclear density gauge, and Colorado State University dustometer tests were performed. Additionally, smoothness and friction measurements were made using the Roadroid cell phone app. The field test results will be compared to those after the spring thaw to track changes in the strength, stiffness, and general performance of the test sections. Visual surveys with photos will also be conducted after periods of thawing and precipitation to assess performance. Maintenance requirements will also be tracked using survey reports completed by grader operators and county engineers.

FINDINGS

Based on the laboratory tests, the optimum compaction moisture contents and dry densities were determined and used to design the test sections. Suggested construction methods were drafted and then modified as necessary according to the actual soil and weather conditions encountered, equipment available, county crew experience, and other challenges commonly encountered in the field. The research team communicated and worked with the liquid stabilizer manufacturers, slag providers, and clay slurry provider to ensure that their recommended construction procedures were followed as closely as possible. Representatives from the manufacturers of the SASS-CLS and I-CLS (see Table 1), as well as both slag providers and the clay slurry provider traveled to the site to oversee construction and provide guidance. Information on the final as-built construction procedures and equipment are provided below to aid other secondary roads departments who may wish to use the same or similar stabilization methods. More detailed lists of construction procedures will be presented in the project's final report.

All of the chemical stabilizers except for the 12-in. cement-stabilized subgrade section were mixed using a 60-in. wide RoadHog mounted on a Caterpillar 938M Wheel Loader and attached to a water truck by a hose system. The portland cement was first spread on the road surface using a spreader truck and the liquid stabilizers were added directly to the water tank before mixing. For all sections except the aggregate columns, tow-behind rubber tire rollers were used for compaction, along with a smooth drum vibratory roller for finishing most of the chemically stabilized sections. The smooth drum roller was also used for the slag sections in Howard County and for all the optimized-gradation-with-clay-slurry sections. Because the three concentrated liquid stabilizers can set up hard in cold weather, it is important to get a smooth finished surface by using the smooth drum roller and tight blading. The 12-in. cement-stabilized subgrade section was mixed using a Caterpillar RM300 Road Reclaimer, followed by compaction with an 86 in. wide pad-foot vibratory compactor.

The optimized gradation with clay slurry sections were based on research described in Li et al. (3, 7). In this approach, the gradations of existing surface materials and up to three potential quarry materials can be entered into a spreadsheet that will determine the optimum mixture proportions to give the tightest particle packing and therefore greatest strength. The clay slurry is used to increase the plasticity to reduce material loss due to water and fugitive dust. The optimization spreadsheet can be downloaded from the Project TR-685 final report webpage given in the references. After blade mixing the optimized proportions of aggregates and forming a 6-in. high windrow on each side, the clay slurry was sprayed by a self-unloading tanker trailer with a custom-fabricated deflector plate, then blade mixed edge to edge with 10 to 15 grader passes. After blade mixing the slurry and aggregate, a light cover of fresh dry aggregate over the

top (two truckloads spread over a 500-ft section) was determined to be effective to minimize sticking of the very wet mixture to the compaction equipment.

Both the steel slag and recycled asphalt pavement (RAP) sections were placed using conventional methods and blade mixed using graders. In Cherokee County, a disc plow harrow was also found effective for mixing the RAP and aggregate together. The slag from Source A had a finer gradation, while that from Source B had a larger and more uniform gradation. Both materials resulted in a good final surface, with the Source A slag packing to a tighter surface due to the higher fines content.

However, their performance through winter freeze–thaw cycles has not yet been compared. Figures 1 and 2 show the various equipment used to construct the test sections.



FIGURE 1 Equipment used for mechanically stabilized sections: (a) disk plow harrow; (b) motor grader; (c) power auger; (d) vibratory compactor; (e) water truck; (f) rubber tire roller; (g) self-unloading tanker trailer spraying clay slurry; and (h) dump truck.



FIGURE 2 Equipment used for chemically stabilized sections: (a) RoadHog reclaimer; (b) water truck with chemical stabilizer added to tank connected to RoadHog; (c) road reclaimer; (d) sheepfoot vibratory compactor; and (e) powder spreader truck.

CONCLUSIONS

All of the stabilization methods except for ground tire rubber resulted in good quality surfaces immediately after construction. The ground tire rubber was incorporated at only 20% by volume in the bottom 2 in. of a 4-in. thick aggregate surface, but created a soft, unstable surface that had to be removed. The clay slurry results in a rather wet construction procedure, but the surface is passable by the end of construction. In all four counties, significant rainfall the same day or one day after construction did not make the sections impassable. Secondary roads crews from all four counties remarked that the optimized gradation with clay slurry sections looked very good (better than the control sections) after a few days of drying, and have continued to hold up well for several months.

In the remainder of the project, the performance of the demonstration sections will be assessed using extensive field tests performed after spring thaws along with photo surveys and condition rating reports completed by grader operators. The costs of constructing and maintaining the various stabilized sections will also be tabulated and compared, to provide insight into their relative effectiveness and economy.

ACKNOWLEDGMENTS

This research was supported by the Iowa Department of Transportation through IHRB Project TR-721. This support is gratefully acknowledged. The findings and opinions expressed herein are those of the authors and do not necessarily reflect the views of the Iowa Department of Transportation. The authors would like to thank the county engineers Nick Rissman, Nicole Stinn, Jacob Thorius, and Brandon Billings as well as their crews for their cooperation and construction of the test sections.

Thanks also to representatives from the SASS-CLS and ISS manufacturers, both slag sources, and the sand company providing the clay slurry, for overseeing and providing input during construction of the related test sections.

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Climate Change Risk Assessment and Risk Management for Transportation Infrastructure in United States National Forests

LESLIE J. BOAK

U.S. Department of Agriculture Forest Service, Pacific Southwest Region

NATALIE J. LITTLE

U.S. Department of Agriculture Forest Service, Intermountain Region

GORDON R. KELLER

Genesee Geotechnical

INTRODUCTION

Climate change poses many serious threats to the nearly 193 million acres of public lands managed by the U.S. Forest Service (1). Most of the existing National Forest transportation system was built over 60 years ago to accommodate timber harvest and log removal. The roads, bridges, and other critical transportation infrastructure needed to access and travel within Forest Service lands were first constructed using the design standards of the 1960s and 1970s. In 2011, the Forest Service developed the *National Roadmap for Responding to Climate Change* and a Climate Change Performance Scorecard (2, 3, 4). The National Roadmap calls for “Protecting infrastructure by modifying or relocating roads, culverts, trails, campgrounds, and other facilities to resist floods and other major disturbances” by using a science-based approach to assessing vulnerability and determining adaptation actions, but it gives each area of the country a chance to develop its own methodology in response to its unique challenges.

The Forest Service is currently responsible for over 370,000 miles of road and 6,200 road bridges. While a comprehensive analysis of the vulnerabilities to the transportation infrastructure would be ideal, the Forest Service has chosen to continue its ongoing activities using the most current science to facilitate adaptation and mitigation strategies. The Forest Service, in cooperation with the U.S. Department of Transportation John A. Volpe National Transportation Systems Center, has developed the *U.S. Forest Service Transportation Resiliency Guidebook* (5). It is based on studies that provide a framework to help field staff assess the most vulnerable transportation assets, which can be incorporated into comprehensive climate change vulnerability assessments at a broad land management scale.

METHODOLOGY

All Forest Service units have been tasked with addressing the impacts of climate change. Methodologies for how unit each assesses, adapts, and/or mitigates the impacts have been carried out in similar ways and adapted to local circumstances. Three different approaches to climate change adaptation are presented in the following case studies.

FINDINGS

Intermountain Region Climate Change Vulnerability and Adaptation Assessment

In April 2015, the Intermountain Adaptation Partnership (IAP) was formed and built on previous Adaptation Partners (www.adaptationpartners.org) efforts. IAP developed a regionwide climate change vulnerability assessment covering the main topics of climate, water resources, fisheries, wildlife, forested and nonforested vegetation, ecological disturbance, recreation, infrastructure, cultural heritage, and ecosystem services. The IAP geographic area includes 12 national forests and 22 National Park Service units. This 3-year concentrated effort to complete a comprehensive climate change vulnerability and adaptation assessment brought together researchers and land managers along with partners to better understand climate processes and how they affect the lands that are collectively managed.

In 2018, the Forest Service published a 15-chapter general technical report (GTR) titled *Climate Change Vulnerability and Adaptation in the Intermountain Region* through the Rocky Mountain Research Station (6). The climate science trends to Year 2100 indicate the Intermountain Region will see increases in annual and seasonal maximum and minimum temperatures. By 2100, the projections show that median minimum and maximum temperatures are projected to rise by as much as 10°F, which in most parts of the region will bring the median minimum temperature above freezing, a critical threshold for snowmelt, which affects runoff.

The infrastructure chapter highlights the potential risks to infrastructure and the surrounding landscape from changing climatic conditions, including from increasing temperatures and extreme weather (Figure 1) (6). The GTR lays out a three-level assessment approach to systematically analyze vulnerabilities to infrastructure from climate change in the Intermountain Region:

1. Maintain an inventory of type and miles of infrastructure. The assessment includes infrastructure summary tables for roads, road bridges, trails, trail bridges, buildings, recreation sites, and dams.
2. Analyze vulnerabilities at regional scales.
3. Analyze vulnerabilities at local or smaller scales.

Assessing vulnerabilities and the establishment of asset rank and purpose will help prioritize planning, funding, replacement, maintenance, and decommissioning. The identification of assets that have a high likelihood of being affected by future climatic conditions can be critical information when developing programs of work. For example, infrastructure, including transportation assets, near or beyond their design life have increased risk to damage from a changing climate, including from flooding and geomorphic disturbance. The most significant damages are often from extreme events.

Adaptation strategies include maintaining an accurate inventory of all infrastructure components and identifying at-risk infrastructure components, increasing the resilience of the transportation system to increased disturbance, and implementing best management practices that take into account climate change.

Similar efforts have been made in the North Cascade Mountains (7). More information may be found at www.fs.usda.gov/goto/cc and www.adaptationpartners.org/iap.

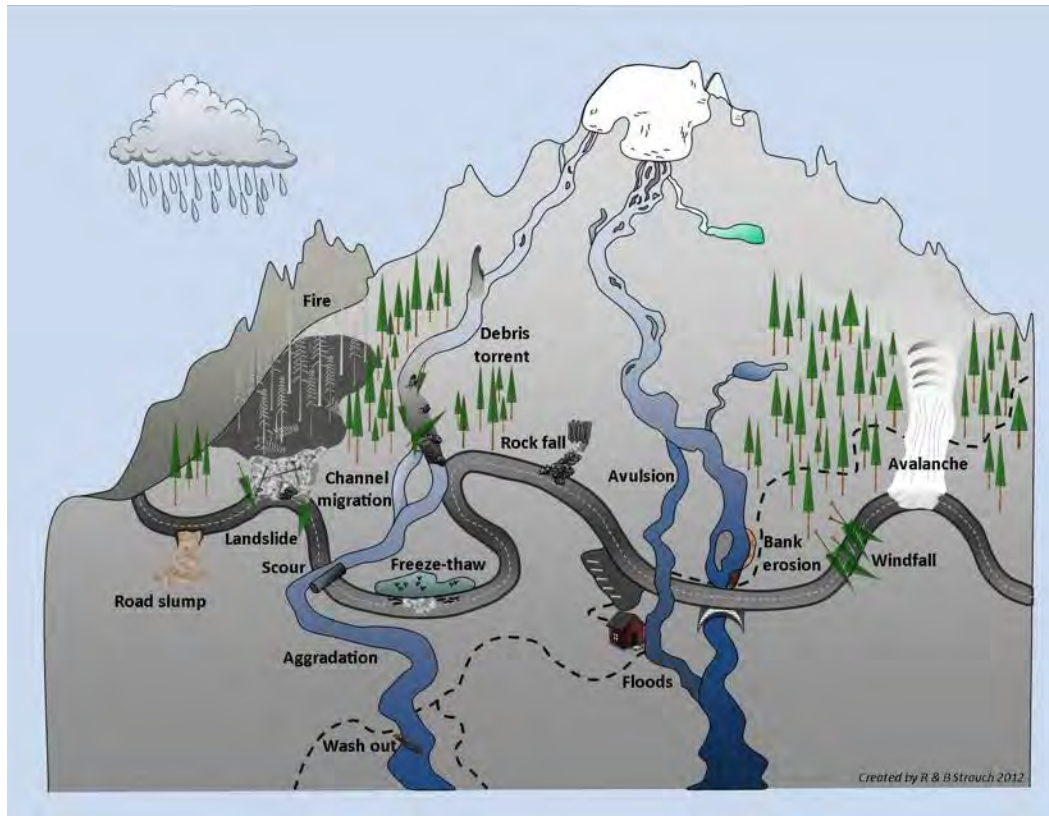


FIGURE 1 Climate-related effects to transportation systems (6).

Pacific Southwest Region Focus Study on the Sierra Nevada Mountain Range

In early 2018, the Forest Service Pacific Southwest Region began a Recreation and Infrastructure Climate Change Vulnerability Assessment for the 10 National Forests that cover the Sierra Nevada Mountain Range. The Sierra Nevada is a biologically diverse and culturally and socially important region of Central and Northern California and provides timber, rangeland, minerals, and recreation opportunities for California. Drought, followed by intense rainstorms, particularly on fire-scarred forests, have led to millions of dollars of damage to roads, trails, culverts, bridges, dams, buildings, and other infrastructure.

Hydrologic impacts and changes can potentially be significant for California's Sierra Nevada region. Climate projections suggest small changes in the total amount of precipitation, with possible increases, but storms will be warmer so that there will be more precipitation as rain and less as snow. Also, storm patterns will be more erratic, runoff peaks will be earlier in the year, and intensities will increase. Increased peak flows will put existing culverts at risk and accelerate erosion and gulying in areas. Sequences of drought and storms, as well as warmer temperatures, will promote more wildfires, followed by debris slide events.

Assessment of infrastructure risk is critical since funding and resources are always limited. Potential problematic areas or sites must be identified and the consequences of damage considered. Risk involves two factors: the likelihood of an event happening and the consequences of that event. This type of risk evaluation is needed to prioritize where limited funds are spent to minimize fire and storm impacts.

While this study is in its infancy, the intent is to produce a vulnerability assessment with a geospatial component of the built environment that complements previous species and habitat-related assessments and will also include socioeconomic issues. Key objectives of this project include the following:

- Synthesize the best available science to assess climate change vulnerability and develop adaptation strategies for recreation and infrastructure resources on National Forests in the Sierra Nevada Mountain Range.
- Develop a framework and tools for managers to incorporate the best available science plus existing/complementary assessments into Forest Service recreation and engineering program assessments.
- Define priority regional and forest-level climate change vulnerabilities so that such factors may be integrated in a cohesive and strategic manner throughout the land management planning process.
- Produce a spatially explicit, peer-reviewed vulnerability assessment, with maps, written to support the needs of FS resource managers.

It is assumed geospatial analysis will help assess the problems, identify areas of risk, and potential problems. Maps of stream corridors, projections of stream bank full width, proximity of roads to streams, expected changes in snowpack depth or duration, fire intensity maps, and soil types can be used to identify hazard risk assessments, while maps of road systems can also identify alternative routes and identify redundancy to key locations. These tools can help identify high-risk areas or sites, potential future problem areas, and resources needed to facilitate repairs after disasters.

Literature reviews of relevant publications, reviews of similar efforts made by other agencies on a state and federal level, and consultation with local personnel as well as key partners in the Sierra Nevada all contribute to the understanding of resources affected by climate change.

Information is being gathered from individuals regarding impacts to resources they have seen and ideas for adapting to the changes and minimizing adverse impacts to infrastructure.

Possible adaptation and mitigation measures include the following: planning for earlier access on forest roads due to less snowpack when subgrade soils are still saturated; ensuring that maintenance is current and road surface drainage measures are functioning properly, minimizing concentration of water; modifying bridge and culvert designs to accommodate larger design flows; addressing potential scour problems at bridges; adding trash racks and culvert diversion prevention measures to deal with increased debris in channels; designing culverts to match channel bank-full width; repairing old retaining structures and filling slopes in marginal conditions; using additional dust palliatives on project roads during droughts; moving facilities away from streams or areas of potential debris slides; and ensuring that critical slopes are well covered with deep-rooted vegetation (8, 9).

U.S. Forest Service Transportation Resiliency Guidebook

Using common themes and methodologies developed in Forest Service case studies across the nation, the Forest Service, with the help of the U.S. Department of Transportation John A. Volpe National Transportation Systems Center, produced the *U.S. Forest Service Transportation*

Resiliency Guidebook in September 2018 (5). The guidebook uses a conceptual framework for general practitioners at a local level to consider the impacts of climate change and prepare for effects to their transportation system. This tool can also be used to help communicate risk associated with climate change to decision makers. A high-level analysis as outlined in the guidebook allows general practitioners the opportunity to rapidly assess transportation vulnerabilities and prioritize high-risk or problem areas. This book provides field-going personnel a process to assess and address climate change impacts on Forest Service transportation infrastructure at local and regional levels. It also highlights ways the Forest Service can make its transportation system more resilient to potential climate change impacts when funding opportunities arise.

CONCLUSIONS

Risk assessments can be used to determine the greatest risks to infrastructure assets; establish a ranking order of vulnerabilities; and help prioritize planning, funding, replacement, maintenance, and decommissioning efforts. Utilization of adaptation strategies such as maintaining an accurate inventory of all infrastructure components; identifying at-risk infrastructure; forecasting changes in snowpack, rain, and stream flows; and implementing best management practices with appropriate road “storm-proofing” measures all lead to a more resilient transportation system. Given funding challenges, a high-level assessment will allow local units to determine how best to focus limited funds and provide asset managers with the tools for wise investment.

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Impact of CO₂ Emissions on Low-Volume Road Maintenance Policy *Case Study of Serbia*

JELENA ĆIRILOVIĆ
IMS Institute

GORAN MLADENOVIĆ
University of Belgrade

CESAR QUEIROZ
World Bank

About 20% of the Serbian national road network has sections with low-volume traffic. These sections are maintained in relatively poor condition since the maintenance budget is typically allocated to the road sections with most traffic. This paper aims to define the appropriate maintenance policy for keeping these sections in “optimal” condition. The traditional approach has been to consider as optimal the condition leading to the minimum sum of road agency costs and road user costs. However, currently there is an emphasis on including environmental cost (greenhouse gas emissions, in particular) into pavement management systems. This extends the concept of optimum by defining it as the maintenance policy leading to the minimum sum of

- (a) Road agency costs,
- (b) Road user costs, and
- (c) The cost to society of CO₂ emissions.

Three potential influencing factors are further analyzed—traffic loading, pavement structural number, and the initial condition of the road section. The World Bank’s Road Network Evaluation Tools model was used to analyze the Serbian low-volume road network and develop the optimal maintenance policy. The results show that the cost of CO₂ emissions plays an important role in calculating the optimal policy, but unlike the high-volume parts of the road network, in the case of low-volume roads, a substantial part of total emissions is related to the production and placement of new pavement layers, rather than from vehicle emissions.

To view this paper in its entirety, please visit:
https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

Making Rural Access Roads More Resilient
*Lessons Learned from Trailing the Climate Change
Adaptation Handbook in Mozambique*

RAQUEL LANGA ABUBACAR

Administração Nacional de Estradas, Mozambique

INTRODUCTION

Like other African countries, Mozambique is facing a challenge of repair and maintaining roads damaged from the effects of climate change (i.e., floods, storms, and cyclones). Climate change–associated road damages have direct socioeconomic effects, particularly to the rural community.

Mozambique has a tropical to subtropical climate, with some semiarid regions in the southwest of the country. A slight increase in mean annual temperature has been observed in Mozambique. The largest increase was observed in the south of the country (1° in 100 years). Average annual rainfall has decreased significantly. The south and coastal regions have experienced an increase in extreme events associated with the effects of climate change.

One focus of the ongoing climate vulnerability project supported by the Africa Community Access Partnership (AfCAP), a research programme funded by UK Aid, is to identify, characterize, and demonstrate the appropriate engineering and nonengineering adaptation procedures that may be implemented to strengthen the resilience of rural roads.

As part of the AfCAP climate vulnerability project, a 50-km gravel (unpaved) road between Mohambe and Maqueze in the Gaza province of Mozambique was identified for the construction of demonstration sections (Figure 1). The road is located along the edge of various natural lakes, making it more vulnerable to the flooding frequently experienced in Mozambique. This road links to a small village that has not more than 2,000 inhabitants who have been besieged during the rainy season. The road is estimated to carry about 25 vehicles per day (vpd) on average, with very few heavy vehicles.

The road also forms part of a climate adaptation program funded by the World Bank. Significant damage was done to this road during the 2013 flooding, and six concrete fords (emergency repairs) were installed to improve passability in 2014, and various short sections were spot regavelled with a blend of sand and calcrete. Although this road had been identified by Mott MacDonald for a climate resilient exercise funded in a World Bank project, their conclusions were primarily related to improving the existing drainage structures.

Four demonstration sections are constructed along the Mohambe and Maqueze road to address climate change–related problems leading to the undercutting of concrete fords, damage to road approaching concrete ford, damage to culverts, and damage to the gravel road surface. The designs and construction program for the demonstration sections are presented in this paper. The long-term performance of the demonstration sections is planned to be monitored over time. The outcomes of the monitoring program are expected to inform on the appropriate adaptation procedures for wider implementation in Mozambique, and possibly elsewhere.

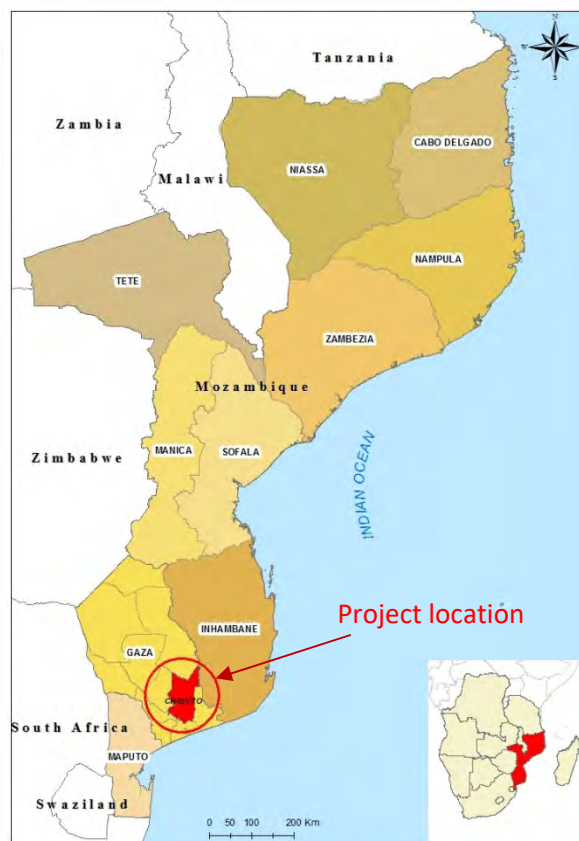


FIGURE 1 Location of the project.

METHODOLOGY

The ongoing project was divided into two phases, namely

- Phase 1: Road assessment, identification of problems, and design of the solutions.
- Phase 2: Construction of demonstration sections.

Phase 1 of the project has been completed. At this moment, construction of the demonstration sections is in the final phase and will be followed by an evaluation of the degree of implementation of the recommendations and, finally, the monitoring of the performance, particularly during the rainy season.

Road Assessment and Identification of Problems

The road was initially visited in September 2016, and a follow-up visit was made in August 2017. The road is unengineered earth road with minimal side drainage and some ineffective mitre drains and made use of the local silty-sand material as the wearing course. The road mostly had only two-wheel tracks and had very few corrugations, indicating the low traffic volumes and speeds. The local sandy materials (reddish and grey) appeared to be suitable for the light traffic

on this road and had been compacted into a hard-wearing course, probably mostly by traffic, although many sections apparently become impassable (slippery) when saturated.

At the northern end of the road, numerous culverts had been assessed as part of the upgrading project. These are spaced at various intervals, but with the flat grades prevailing in the area, many of them appeared to be unable to move the water away from the road effectively.

Very few areas have properly constructed and maintained side drains. Mitre drains are constructed regularly along the road, but not always in the correct positions and usually too short and poorly graded, resulting in ponding of water in the drains and onto the adjacent road.

Numerous culverts had been damaged by flooding. Most of the damage was due to erosion at the exits with undercutting and cracking. However, the incidence of undercutting of erosion protection measures (mostly grouted stone pitching) was evident at many of the structures. This was often due to poor compaction of the embankment material allowing access of water behind the structures as well as surface erosion—improved anchoring and drainage are needed at these structures. The main areas affected are parts of the approach fills that are eroded during flooding, protection works that are left unsupported or washed away, and scour of the stream-bed at outlets (one of the identified demonstration sections was constructed at this location).

A major drainage problem had occurred at km 47. An old borrow pit had become a dam to the north-west of the road section. After a rain, the dam overflows and creates a stream that leads onto the road with no drainage provided. This has caused extensive erosion adjacent to the road as well as severe damage to the road. This area required significant erosion protection and the installation of an appropriate drainage structure.

At km 43, the culvert was showing significant damage due to erosion. There was severe erosion and undercutting beneath the water exit slabs, loss of parts of the grouted stone protection, and erosion of the approach embankment to the south. The approach road on both sides of the culvert was very poor and needed significant lifting and improvement. In addition, the drainage channels were incorrectly handled by the structure and required some realignment. This area was identified as a potential demonstration section.

The damage to the unpaved road surface was seen at several locations, resulting from different causes, including erosion by uncontrolled water and poor compaction of the materials and use of erodible materials as the wearing course.

The concrete ford constructed under the World Bank emergency repair program in 2013 at km 36 has performed relatively well, although the gabion baskets showed significant signs of rust and failure. The road at the northern approach to this ford is badly deformed and eroded and required repair. It was clear that the ford does not extend sufficiently up the slope to avoid erosion of the road during overtopping, and this had to be addressed in a demonstration section.

The undercutting of concrete fords was observed in two locations. It is not apparent whether this was caused by overtopping of the ford by the stream flowing from the east or wave erosion of the unprotected material beneath the concrete. The concrete ford in km 17 was the most affected by the erosion and had lost part of the old concrete. This was addressed in a demonstration section.

Design and Construction of the Solutions

After the road assessment, four demonstration sections were designed and constructed to address the identified problems as described in the following sections.

Section 1: Erosion and Undercutting of Concrete Fords

The recommended solution is to incrementally construct a vertical wall (200-mm thick with 16-mm reinforcement at base at 250-mm intervals) near the edge of the concrete ford slab (on inert material), backfill the voids with an inert fill and a high slump concrete (0.5 m thick), and then extend the concrete slab to the top of the wall. The wall was founded on imported inert material at least 1-m deep and has a concrete protection slab at the base. Figure 2 depicts construction progress by mid-October 2018.

Section 2: Damage to Culverts and Erosion Protection

The recommended solution is to properly shape the road and level off at the top of the culverts with the grade from 5 m to the north of the first culvert to 5 m to the south of the second culvert being perfectly flat. All damaged protection works should be removed. The materials in these areas must be replaced with inert material and compacted to at least 95% mod AASHTO density. The stone pitching should then be repaired with a cement grout filling all joints.

Section 3: Damage to Road Approaching Concrete Ford

Initially, it was proposed that a continuous (i.e., unjointed) roller compacted no-slump concrete slab 150-mm thick and 45-m long (the same width as the existing ford) should be constructed. Instead of constructing the proposed roller compacted concrete, the damage to the road approaching concrete ford at km 23 was repaired by constructing improved gravel road.

Section 4: Ineffective Drainage of Road Surface: Poor Shape and Side-Drains

The recommend solution is to construct an improved gravel road with wearing course material that complies as closely as possible with the standard specification and is placed in a 150-mm compacted layer at not less than 98% mod AASHTO effort. The camber of the wearing course after construction should be 5%. In addition, the culvert at the start of the section should be reconstructed.



(a)



(b)

FIGURE 2 Section 1 construction progress: (a) before start of construction in August 2017 and (b) construction progress by mid-October 2018.

FINDINGS

Although still preliminary, the results of quality control in the field show that the calcrete soils used in the wearing course will have a good bearing capacity, which will guarantee less erosion on the road surface.

During the last rain season, it was noticed that the culverts performed well. The concrete strips that were built around the edge of stone pitching had an important role of protecting the stones.

WAY FORWARD

This project is expected to produce manuals and guidelines with engineering solutions that will improve road maintenance works with low traffic volumes and to address the effects of climate changes on the structures. The results of the demonstration sections will be replicated to other regions with the same type of problems.

The sections performance monitoring plan is being prepared carefully, considering that the characteristics of the demonstration sections differ (pavement, aqueducts, concrete fords). The “guideline for the monitoring of experimental and LTPP sections in Mozambique,” which is being prepared and funded by UK Aid, are anticipated to be used during the performance monitoring.

ACKNOWLEDGMENTS

This paper is the result of a research program that was commissioned by the Africa Community Access Partnership (AfCAP), a research program funded by UK Aid. The authors express their gratitude to the roads authorities in Ghana, Ethiopia, and Mozambique for their inputs into the refinement of the guidelines and handbook.

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Pavement Management

Accuracy of Statewide Pavement Surface Evaluations and Ratings Performed by Local Agencies

SHARLAN R. MONTGOMERY
Nichols Consulting Engineers, CHTD

JOHN E. HADDOCK
Purdue University

The passage of state legislation has promoted and encouraged the development of pavement management plans at the local level. A widespread response to such legislation in some states has been the assessment of pavement condition using the pavement surface evaluation and rating (PASER) method. The specific objectives of this research were to

- a. Assess the accuracy of statewide local agency PASER raters;
- b. Identify PASER values that are difficult to rate; and
- c. Propose a standard tolerance to be used in statewide quality-control evaluations.

Statewide PASER condition data collected by local agencies in Indiana were statistically subsampled and rated by PASER-certified Indiana Local Technical Assistance Program (LTAP) personnel. The local agency ratings were then compared with the LTAP ratings using four agreement tolerances:

1. Complete agreement;
2. Within-1-rating agreement;
3. Within-2-ratings agreement; and
4. Good–fair–poor category agreement.

Local agencies tended to overrate the middle-to-low PASER values (1–6) and slightly underrate the upper PASER values (7–10). This means that budget needs based on data collected by local agencies are likely underestimated. Pavements in poor condition (PASER 1–4)—particularly PASER 4—were assigned the correct good–fair–poor category the least often. Since the good–fair–poor category agreement was the most rigorous tolerance after complete agreement and has the potential to capture significant funding differences appropriate to each category, it is recommended for use as a statewide quality-control standard.

To view this paper in its entirety, please visit:
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PAVEMENT MANAGEMENT

Using Summer Interns to Develop Pavement Management Plans for Local Highway Agencies in New York State

GEOFFREY R. SCOTT

DAVID P. ORR

Cornell University Local Roads Program

The Cornell University Local Roads Program (Cornell) has worked with over 137 municipalities throughout New York State since 1993 in the development of pavement asset management plans. To facilitate the implementation of pavement management plans, Cornell originally used a software package developed by the New Hampshire Local Technical Assistance Program Center. More recently, Cornell worked with a former intern to develop a modern, easy-to-use asset management software focused on pavement management using concepts from the original road surface management system. This software is available to any municipality interested in developing an asset management plan for their pavement. To facilitate the implementation of asset management plans, Cornell matches an interested municipality with a qualified college student intern to organize, evaluate, and develop a pavement management plan specific to the municipality. Over many years, and several iterations of software, Cornell has improved the intern project. The project has helped many local agencies develop management plans that not only help identify the maintenance and construction needs of a roadway network, but also justified an increase in funding in many cases. Currently, the project, known as the Cornell Asset Management Plan Summer Intern Project, annually facilitates approximately 15 to 20 municipalities across New York State in the development of pavement management plans.

INTRODUCTION

Pavement management systems help agencies better understand their transportation network and in doing so can help in the efficient budgeting and planning of maintenance and repairs; thereby extending the life of a roadway network, improving the level of service provided to its users, and allowing for the better use of the limited funds that are available for those activities (1). For many small municipalities in New York State, a comprehensive overview of the roadway network is known only by the highway official in charge of that network, since much of the maintenance history is never documented. When these managers retire, the loss of this institutional knowledge often results in the complete history of a public asset being lost over time. This forces every new road manager to rediscover what can be reconstructed while forging ahead with a trial-and-error approach in managing their infrastructure.

Since 1993, the Cornell University Local Roads Program (Cornell) has offered the Cornell Asset Management Plan (CAMP) Summer Intern Project to all local highway agencies within New York State. In 2013, when funding became available for the development of an upgraded user-friendly pavement management software, Cornell turned to a former student intern and program assistant, Pete Schmalzer, to redevelop the software originally used: Road

Surface Management Systems (RSMS) (2). RSMS was originally developed by the University of New Hampshire in the mid 1980s.

The redevelopment was necessary because of the inconsistencies and issues associated with the original DOS-based format of the RSMS software. It became difficult to manage and operate this older software on today's newer computer operating systems. CAMP was developed on a Windows Access-based database format, which would allow for the software to remain simple in nature and work on a standalone basis (3).

ASSET MANAGEMENT

As a tool for identifying, assessing, and planning, asset management has become a critical means for many municipalities to address their various assets. In New York, common methods encountered and used to develop road maintenance budgets usually include one or more of the following:

- Last Year's Budget, with random increases or decreases;
- Standard Program, where programmed maintenance is conducted on a periodic basis;
- Squeaky Wheel, where the emergency and citizen complaints dictate maintenance response;
- Worst First, where the streets in the worst condition get the work;
- Political Pressure, where politics are used to develop programs and budgets; and
- Gut Feeling, where experience and knowledge of experienced employees are used.

These approaches, whether singularly or in combination, may work when a system is in good condition with adequate funding. In New York, many local roads networks do not have the necessary funding, nor do these above-mentioned methods actually help plan for the future, look to extend the life of the existing pavements, or focus on improving the overall roadway network. With the increased demand for accountability and a decrease in the supply of funding, particularly if an agency has roads in medium to poor condition, it has become increasingly clear that there is a need for the tools to allow for better decision making based on reliable information.

There are many reasons that municipalities resist the implementation of documented management plans including a resistance to change from current practices; the fear of being "discovered" as unprepared; the fear of the supervisor losing "power" by sharing information; and/or lack of manpower and funding to continue program support. In larger organizations, communication between the pavement management manager and other operating units within the agency can be diluted or lost, resulting in the management plan failing to get implemented. To help local agencies in New York State overcome these implementation road blocks, Cornell developed the summer intern project that matches college students with municipalities.

In many local highway departments around New York, the number of employees limits what a department can do in a day. By using a student intern, a municipality can develop a pavement management plan with limited time available from current employees. The use of an intern also allows for an unbiased evaluation of the road network that is based on the conditions of those roads. This allows the entire network to be reviewed, relative to itself, in an unbiased

approach that has been widely accepted by local government officials and the public, when used as a communication tool.

Cornell provides training to the student intern and a municipal employee on how the process works and how to make it successful. In addition, Cornell provides technical support via phone and e-mail and conducts a minimum of two site visits to each location to discuss issues, provide guidance, and ensure the interns are moving forward in the right direction.

Cornell Asset Management Program for Roads and Streets

The Cornell Asset Management Program (CAMP) software is basic and easy to use, yet still provides the necessary information to make good decisions. The newer CAMP software, developed in 2013, is based on concepts in RSMS, but uses more modern computer code and database format (4). The new software is easier to use for the summer interns who set up the management plan for the municipality under the assistance and guidance of a municipal employee.

Factors considered in its development were the following:

- Be easy to learn and use,
- Provide a systematic and organized approach,
- Be workable and realistic, and
- Be understandable and accepted by the local board.

The software is a network-based approach to pavement management that stores and analyzes data, which generates reports that assist in cost-effective decision making. The steps in the CAMP process are the following:

1. Prepare an inventory of the highway network.
2. Assess conditions of the network segments.
3. Develop repair alternatives, and local costs, typically used by the agency.
4. Assign repair alternatives specific to the distresses found during the condition assessment.
5. Prioritize maintenance needs.
6. Generate reports, budgets, work schedules, and work orders.

At the heart of the software are its decision trees and its ability to prioritize repairs and maintenance. A decision tree is essentially a three-by-three matrix with the vertical edge representing severity and the horizontal edge representing extent. For each box of the matrix, there is a deduction value that when selected deducts a specific number from the total condition of the road segment.

Figures 1 and 2 show snapshots of the background of the CAMP software and are a representation of how the software deducts points to calculate the Pavement Condition Index (PCI) and selects Repair Categories for each distress severity/extent level encountered. Figure 1, PCI Deducts, shows a three-by-three matrix; this matrix identifies the amount of points to be deducted for the specific level of severity and extent of each distress. At the same time, a Repair Category (Figure 2) is selected for the corresponding severity/extent. For each distress evaluated, a condition severity and an occurrence frequency on that road segment are determined. This

simultaneously selects the number of points to be deducted and which repair category will address the specific distress in its observed condition. There is a decision tree associated with each distress rated. The software tallies the total number of points for each distress to be deducted for each road segment; this value is then subtracted from 100 to give a PCI value for that specific section. In a case where multiple distresses are identified, the repair category with the more critical distress will take precedent, with the assumption that repairing the more critical distress will also address the lesser distresses that have been identified.

Each distress is caused by a specific type of failure within the roadway; identifying each distress and its condition will help determine the proper repair selection. Repair selections are focused on correcting the problem to minimize the potential for similar issues arising in the future at the same location. This allows the network to be improved for long-term sustainability, facilitating the overall improvement of the roadway network over time.

The 'PCI Deducts' interface features a grid of input fields. At the top left, there is a 'No Distress' label above a single input field containing the number '0'. To the right of this is the 'Extent' label. Below 'Extent' are three input fields containing the numbers '2', '4', and '6'. To the left of these three fields is the 'Severity' label. Below 'Severity' are three input fields containing the numbers '6', '8', and '10'. At the bottom of the grid are three input fields containing the numbers '10', '12', and '14'.

FIGURE 1 Illustrative PCI deduct.

The 'Repair Categories' interface shows a decision tree structure. At the top left, there is a 'No Distress' label above a dropdown menu with options: '41-Defer Maintena', '42-Crack Repairs', '43-Patching', and '44-Surface Treatm'. To the right is the 'Extent' label. Below 'Extent' are three dropdown menus, each with the same four options as the 'No Distress' menu. To the left of these three menus is the 'Severity' label. Below 'Severity' are three dropdown menus. The first two have the same four options as the 'No Distress' menu. The third dropdown menu has five options: '41-Defer Maintena', '42-Crack Repairs', '43-Patching', '44-Surface Treatm', and '45-Overlay'. The fourth dropdown menu has four options: '43-Patching', '44-Surface Treatm', '45-Overlay', and '46-Rehab'.

FIGURE 2 Decisions tree in CAMP-RS.

Prioritization within the CAMP software takes into account multiple factors, including the strategy (or repair category), importance, traffic, PCI, drainage, and roughness. Each factor can be weighted by the user to put greater influence on any of the specific factors. The greater the value of the priority the sooner the repair is recommended.

$$\text{Priority} = k1 * S * \left(k2 * I + k3 * T + k4 * \frac{PCI}{10} + k5 * D + k6 * R \right) \quad (1)$$

where

Priority = priority score;
S = strategy;
I = importance;
T = traffic;
PCI = pavement condition index;
D = drainage;
R = roughness; and
k_i = weighted percentages, 1–100.

In the above equation, the strategy, *S*, referred to as the repair category within the software, is the primary factor that dictates the priority as it is a multiplier for all of the other scores. Since work is scheduled based on this score, the strategy is also the driver of when work will be accomplished. The determination of which strategy will be recommended is made when the status of each distress is chosen. Referring back to Figures 1 and 2, as each level of distress is determined, simultaneously a value to deduct is made for the PCI determination and a repair category is selected. The strategy column in Table 1 shows the values that are used in the priority equation. There are eight repair categories for each surface type within the software (Table 1), with slight variations from each other to address the different levels of repair that may occur for each surface type. Each level of the repair category has a strategy value and a precedence value that allows the user to adjust the emphasis of each.

The underlying schema of the strategy setting is to focus on maintaining roads in good condition while addressing the roads in poor condition as funding allows. By applying a high strategy value to maintenance work and a low strategy value on high-cost repairs, rehabilitation, and reconstruction, the software can prioritize when a repair activity shall be recommended. A higher strategy score will yield a higher priority value, which will recommend the repair be scheduled sooner. Drainage work typically has the highest priority since removing moisture from the road surface and base will help extend the life of the roadway, while reconstruction is a lower priority when it comes to maintaining the road or street (5).

Precedence is the order in which the category is selected within the software, starting with low precedence of defer maintenance to a high precedence for reconstruction. When multiple strategies are identified based on the distresses present, the software will select the repair strategy with the higher precedence value to address the more severe condition, assuming that repair will correct the other deficiencies. Typically, the higher the precedence of a repair category, the lower value of the strategy, which reflects in a lower priority when it comes to scheduling the work. The exception to this in all situations is the repair category for drainage. As mentioned, drainage is a key component in the survival of any road surface, therefore, in managing the extension of a pavement's life, addressing drainage first is always critical. As such

TABLE 1 Repair Categories in CAMP-RS

Unpaved Surfaces	Strategy	Precedence
Drainage work	5	8
Reconstruct	1	7
Regrade	3	6
Reshape surface	4	5
Spot add material	6	4
Dust control	5	3
Spot dust control	6	2
Defer maintenance	1	1

Surface Treated	Strategy	Precedence
Drainage work	6	8
Reconstruct	2	7
Asphalt overlay	3	6
Surface treatment	4	5
Spot repair	3	4
Patching	5	3
Crack repair	6	2
Defer maintenance	1	1

Asphalt Surface	Strategy	Precedence
Drainage work	6	8
Reconstruct	1	7
Rehab	2	6
Overlay	3	5
Surface treatment	3	4
Patching	4	3
Crack repairs	4	2
Defer maintenance	1	1

Concrete Surface	Strategy	Precedence
Drainage work	6	8
Reconstruct	1	7
Rehab	2	6
Overlay milling	3	5
Surface treatment	4	4
Patching	4	3
Crack repairs	5	2
Defer maintenance	3	1

the software is set to have drainage as the highest strategy, allowing the correction of drainage issues to be a priority when scheduling maintenance work.

When determining the importance of a road or street, the software allows the user to select from five levels. To keep it simple and to avoid the impact of a knowledge void, Cornell recommends that the importance be focused on the local roads being evaluated. This means ranking an agency’s highways based on the relative importance to the community. For example, a roadway leading to a hospital or fire house would have a high importance to the community, while a dead-end with a limited number of users, would have a low importance to the community as a whole.

This same approach is also taken when an agency is determining the traffic level. Again, five different levels of traffic are available for the user. For roads where traffic counts are high or where truck traffic is high, agencies are recommended to identify these roads as high traffic, while a road such as a dead-end has a low volume of traffic. Many local agencies have opted to use only three of the categories (1, 3, 5) since most of their roads are low volume in nature. This allows them to simplify their assessments and tailor the software to their needs.

PCI is determined through the deduction of points process described earlier. As the distresses are evaluated during the condition assessment process, each value less than perfect causes a deduct value for each distress. The accumulation of these deducted points are subtracted from 100 to calculate the PCI. An example score is shown in Figure 3.

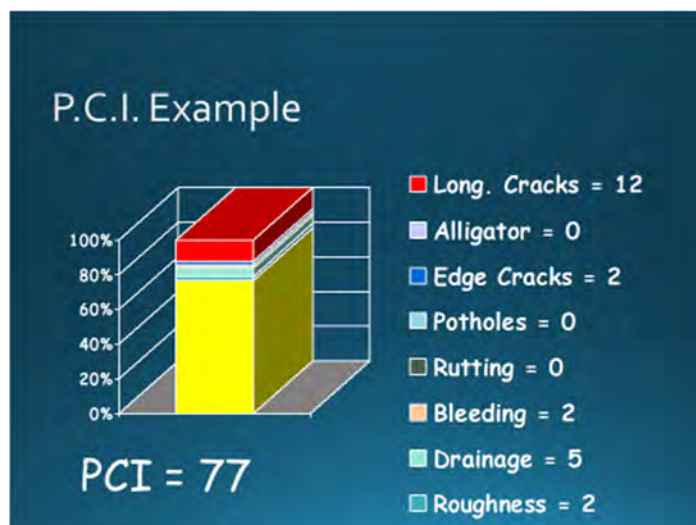


FIGURE 3 PCI example for a highway segment with a PCI of 77.

The PCI used in this software is not related to any other methods of calculating PCIs and is strictly related to the deduction process within the software, but an informal review showed the PCI value is very similar to the pavement condition score used by the New York State Department of Transportation (NYSDOT) in their *Comprehensive Pavement Design Manual* (6). The value determined provides a means for road managers to assess the overall condition of the network while identifying roads within that network that will need additional work to bring the condition rating higher. It is also a good communication tool to provide comparisons between highway segments in the local network.

Maintenance of drainage is one of the more important practices available when managing a road network (7). As such, the CAMP software includes this as an important component in managing the network. Drainage refers to the condition of the roadway and roadside drainage. Roadway drainage includes the road surface cross slope, shoulder build up, and the ability of runoff to leave the surface of the roadway. Roadside drainage includes the effectiveness of the shoulder and ditches, or curbs, gutters, and storm sewer systems. Drainage condition is determined during the condition assessment and is rated based on its severity on whether it is good, fair, or poor. Good is when it functions properly; poor is when it has failed; and fair is when there are issues, but it still functions.

Roughness is the last category used in determining the priority of maintenance of a road section. Again, the approach is to keep the process simple. When deciding the roughness of a road surface, users are advised to use the “coffee cup” test, where the evaluator imagines holding an open-top coffee cup while driving on the road to determine the road surface roughness. This test assumes three levels of roughness; smooth, medium rough, and very rough. With an open coffee cup in mind, a road where the driver could drink from the open cup would be considered smooth. On the other end of the spectrum, if the ride is too rough to even hold the cup of coffee without it spilling, the segment would be considered in the very rough category. Anything in between is considered a medium roughness.

The software also allows the user to increase the impact of a specific feature of the priority equation through the use of weighting factors. These weighting factors, k_i , can be increased from the default of 1 to a maximum value of 100. It is recommended that the user be

confident in the use and results provided by the software before modifying any settings, including the priority weighting factors. Cornell recommends using the default values of 1 until the user has fully developed an initial pavement management plan.

CAMP Database

The CAMP software uses three underlying tables of data:

- Road network inventory;
- Road surface condition; and
- Maintenance, rehabilitation, and reconstruction (MR&R) strategies.

The road network inventory contains the basic information about each road segment in the network. The inventory number must be unique. The basic data collected and stored are illustrated in Figure 4.

The road surface condition table stores the data collected during the visual inspection of the distresses for each segment. This is a critical component since distress evaluation of different distresses are directly related to certain causes of pavement deterioration. Because of this, each of the distress types are linked to proper MR&R strategies, identified as “Repair Strategies” in Table 2. Descriptive scales are used for distress assessment in the form of extent and severity of the distress as described previously. To keep things simple, the extent and severity are only low,

The screenshot shows a software dialog box titled "CAMP-RS - Add New Section". It contains the following fields and controls:

- RIN:** A text input field.
- Name:** A text input field.
- From Street:** A text input field.
- To Street:** A text input field.
- Begin (Miles):** A text input field.
- End (Miles):** A text input field.
- Width (Feet):** A text input field.
- Lanes:** A spinner control with values 1 and 2.
- Surface:** A dropdown menu with options: 0-NA, 1-Other, 2-Unpaved, 3-Surface Treated.
- Shoulder:** A dropdown menu with options: 0-None, 1-Paved - Asphalt, 2-Gravel, 3-Earth.
- Traffic:** A spinner control with values 1 and 2.
- Importance:** A spinner control with values 1 and 2.
- Notes:** A text area for additional information.
- Buttons:** "Save" and "Cancel" buttons at the bottom.

FIGURE 4 Inventory data.

medium, or high. In some cases, only severity or extent is used with a particular distress. This allows survey personnel to use the scales with more consistent results and allows the results to be understood by nontechnical people involved in the decision process. Table 3 shows the distresses evaluated by surface type.

Distresses used in the CAMP software vary depending on the type of road surface. For gravel or unpaved roads, the distresses evaluated include the cross section, potholes, roadside drainage, rutting, corrugations, loose aggregate, dust, and roughness. For surface-treated and asphalt roads, the common distresses evaluated include longitudinal/transverse cracks, drainage, alligator cracking, roughness, patching/potholes, rutting, and bleeding/raveling. For surface-treated surfaces, there is the addition of the distress, treatment condition; while the asphalt surface contains the distress, edge cracking. Concrete distresses evaluated include drainage, roughness, patching/potholes, surface condition, durability “D” cracks, faulting, corner breaks, and joint seal condition.

Distress Definitions (4, 8)

- Gravel or unpaved road surface
 - Cross section: Ability for run-off to flow from the roadway.
 - Potholes: Areas loss of material resulted in bowl-shaped depressions.
 - Roadside drainage: Whether roadside drainage, including ditches, culverts, or shoulders, can properly direct and carry run-off water away from the road.

TABLE 2 Repair Strategies by Surface Type

Unpaved Surface	Asphalt Concrete or Surface-Treated Surface	Concrete Surface
Defer maintenance	Defer maintenance	Defer maintenance
Dust control	Crack repairs	Crack repairs
Additional gravel	Patching	Patching
Reshaping	Surface treatment	Surface treatment
Regrading	Overlays	Overlay milling
Rehabilitation	Rehabilitation	Rehabilitation
Reconstruction	Reconstruction	Reconstruction

TABLE 3 Distresses Evaluated by Surface Type

Unpaved Surface	Asphalt Concrete or Surface Treated Surface	Concrete Surface
Cross section	Alligator cracking	Drainage ^a
Roadside drainage ^a	Long/trans. cracking	Roughness ^a
Corrugations	Edge cracking	Durability D cracks
Potholes	Patch/potholes ^a	Patching/potholes
Rutting	Roughness ^a	Joint seal condition
Loose aggregate	Rutting	Surface condition
Dust	Bleeding/raveling ^a	Faulting
Roughness ^a	Drainage ^a	Corner breaks

^a Only severity or extent is used.

- Rutting: Channels along the wheel paths that cause the run-off to drain along the road surface rather than drain to the edge of the road.
 - Corrugations: Closely spaced ridges and valleys, perpendicular to the flow of travel, and spaced at fairly regular intervals.
 - Loose aggregate: Aggregate particles that have been loosened and moved into berms along the shoulders or center of the roadway.
 - Dust: A hazard created by the loss of fine materials from the road surface by vehicles.
 - Roughness: Irregularities in the roadway surface that adversely affect the comfort of the ride.
- Surface-Treated and Asphalt Roads
 - Longitudinal/transverse cracks: Cracks run parallel or perpendicular to the road centerline.
 - Drainage: Ability for the run-off to flow from the paved area.
 - Alligator cracking: Interconnected crack patterns that resemble alligator skin or chicken wire.
 - Roughness: Irregularities in the roadway surface that adversely affect the comfort of the ride.
 - Patching/potholes: Areas where original pavement has been replaced due to deterioration. Patching does not include properly prepared patches. Potholes are areas where portions of the road surface have broken and loss of pavement has resulted in a bowl-shaped pattern.
 - Rutting: Channels in the wheel paths, causing water to flow along the road surface rather than drain to the edge of the road.
 - Bleeding/raveling: Bleeding refers to excess asphalt material on the surface of the roadway, reducing skid resistance. Raveling is the wearing away of the pavement surface caused by the dislodging of the surface aggregate particles and loss of asphalt binder.
 - Treatment condition: Quality and condition of the surface treatment.
 - Edge cracking: Cracks adjacent and parallel to the edge of the pavement.
- Concrete Roads
 - Drainage: Ability for the run-off to flow from the concrete surface.
 - Roughness: Irregularities in the roadway surface that adversely affect the comfort of the ride.
 - Patching/potholes: Patching refers to areas where the original concrete has been removed and subsequently replaced. Potholes are areas where portions of the concrete have broken and loss of material has resulted in a bowl-shaped depression.
 - Surface condition: The presence and severity of map cracking, scaling, polishing, pop outs, and spalling.
 - Durability “D” cracks: Closely spaced crescent-shaped hairline cracking pattern that occurs adjacent to joints, cracks, or free edge, initiating in slab corners.
 - Faulting: An evaluation of elevation differences across a joint or a crack.
 - Corner breaks: Where a portion of the slab is separated by a crack, which intersects with the adjacent transverse and longitudinal joints, describing approximately a 45-degree angle with the direction of the traffic.

- Joint seal condition: Ability to prevent incompressible materials or water from infiltrating through the joint from the surface.

Repair strategies are preloaded into the software, and decision trees are used to associate them with surface distresses. Each surface distress category has a decision tree that relates each severity/extent condition with each of the strategies. Table 3 shows the repair strategies by surface type.

Accurate estimated unit costs and life expectancies of each repair strategy are a required input. Accurate costs are important in the selection of the most cost-effective alternative. The user needs to develop MR&R strategies actually used by the local agency.

CAMP Summer Intern Project

The CAMP Summer Intern Project matches qualified student interns with interested municipalities across New York State to develop the pavement management plan. The use of an intern helps local agencies to develop a documentable pavement management plan without restricting the ability of the department to conduct work it has been obligated to perform. To ensure that all parties are aware of the goals and how to get there, Cornell provides three days of training on the Cornell campus for both the student and a municipal employee. This training covers a wide range of topics related to pavement design, maintenance, and management. Training includes the following:

- Safety—including proper personal protection equipment—issues and hazards found in the highway right-of-way;
- Roadway structure and materials;
- Concepts of pavement management;
- Pavement distresses and their causes;
- Repair techniques and application for best results;
- CAMP software overview and practice;
- Inventory collection;
- Developing unit costs;
- Selecting repairs; and
- Developing a management plan and budget.

The training also includes field practice in highway sectioning, inventory collection, condition evaluations, and software use to provide actual experience. Providing this hands-on experience gives the student and the municipal employee confidence in moving forward with the project when they return to their specific municipality.

Once at the municipality, the intern researches and gathers the road inventory information from local and state resources; confirms road section boundary lengths and widths; and, with assistance of the municipality, identifies the level of traffic and the importance of each road segment. Properly identifying the traffic level and importance of the road segment to the community is valuable since these parameters are used to set priority in maintenance and repairs.

Once the inventory is gathered, confirmed, and entered into the software, the student proceeds to conduct condition evaluations on each segment. Following the guidance developed

by FHWA, *Distress Identification Manual for Long-term Pavement Performance* (8), the student and a municipal employee typically begin the evaluation process to confirm the evaluations are within the guidelines and consistent. Once the entirety of the road network segment distresses are assessed, these data are entered into the software.

Based on the data collected, the software provides recommended repair strategies and a condition index, from which the student and municipal employee determine the most cost-effective repair to extend the life of the pavement and preserve or improve the level of service of the road network. Critical to this process are the determination of the repair unit costs, from which estimated project costs are generated. Once the repair is selected, the software provides a prioritization of each of the road segments based on the concept of keeping roads in good condition. Annual budgeting using available funding is then used to develop a multiyear maintenance plan.

Once repairs have been selected and prioritized and the initial plan developed, the intern and the municipal employee sit down and prepare a final management plan that outlines which repairs will be conducted. The student then prepares a final report for the agency that walks through the CAMP process, including the inventory and condition assessments, the training, and the findings and recommendations. Included is the final management plan, which outlines the recommended repairs and repair sequence. This report typically outlines the scheduled repairs for the following 5 years, at which time it is recommended that the network be re-evaluated and the plan updated.

TRIALS AND TRIBULATIONS

The goal of the CAMP software was to provide a simple tool that could be used by smaller agencies to better manage their road network. The opportunity to participate in the summer intern project, for many smaller agencies, was the support they needed to help implement a pavement management system. The interns were able to conduct the time-consuming tasks of gathering and coordinating the necessary information into a readily usable tool tailored to each specific municipality that can be built on in the successive years to follow.

Initially, the updated CAMP software had several glitches that created nuisances to the user. In the past 6 years, minor corrections were made to improve the program. Ultimately, most of the correction recommendations were provided by the students who were using the software regularly. From these recommendations, Cornell prepared a list of wish items of more extensive corrections to be fixed when the opportunity presented itself. In addition, many agencies were pleased with the pavement management aspect of the software, but requested the addition of other assets, developed on the same concept of “simple and easy to use.”

Many agencies successfully worked to link the CAMP software to a GIS format. Using the GIS format, many of the agencies were able to better communicate their management plans and overall goals to their elected boards and the public. This better communication often led to an increase in funding to achieve the goals outlined. The reports generated by the students also have proved useful. These reports have been used in subsequent years for planning the work activities on the road network and helped define the work scope and budget needs. In several cases, the municipalities have put these reports on their websites for the community to access, or to refer residents, to the pavement plan to identify when other roads were to be repaired or maintained.

Associated with the use of student interns for the summer came the difficulty of obtaining students. Typically, a student in the STEM (science, technology, engineering, and mathematics) programs is preferred. However, in recent years finding students interested in participating in the internship has been difficult. The demand for this internship tends to fluctuate with the economy, the better the economy the more difficult to find interested students. To overcome this, the requests for participation have expanded to most of the colleges and universities in New York State, expanding from just engineering schools. Qualified students from other colleges and universities outside of New York State were also considered, although not pursued.

Experience has also found that when students are recruited locally (often by the municipality), they tend to take ownership of their work and are more conscientious of the work they are doing. Being local also helps the student find a place of residence easier than a student from out of town, who would need to find a place to stay during the summer. In areas where tourism dominates summer housing, it can be very difficult for an intern to find housing at a reasonable cost. Colleges and universities close to an agency are often used for student interns to find an apartment to sublet for the summer, defraying some of the total cost for housing.

Although there is no cost to the student to participate, as mentioned, the costs for housing and the anticipated pay rate can also be discouraging to some students. Many students have different opinions on the amount they should be paid. For interns in an engineering field of study, the Cornell program recommends the pay scale be in the range of \$12 to \$15 per hour. However, with the rising of minimum wage in some areas, many students opt for the high rate of pay in lieu of the experience of an engineering internship. To compound this, many agencies do not pay their current regular employees in the recommended pay scale range. In these cases, the municipality offers what it can to the students. Typically, the students accept the position, but there have been experiences where the desire for pay outweighs the desire for experience.

Each year the Summer Intern Project is offered to all municipalities in New York State. It is unfortunate that of the 1,599 local municipalities in New York State, only a total of 137 municipalities have participated in the program. In recent years, there has been an increase in the number of returning municipalities. Many of those that are 3 to 5 years into a previously developed pavement management plan request participation in the project again to update their plan. Each agency differs in their reasoning and methods when re-participating; for Cornell, the important aspect is that they are maintaining and continuing with their pavement management program.

Comments provided in project evaluation forms have found that there have been many successes gained from participation. The management plan and report serves as a valuable resource for communicating the pavement management plans of a community to both the elected board and the community. By educating elected officials on the process, several boards have come to understand the critical needs of the highway network and have increased funding, specifically funding aimed at local highway maintenance and reconstruction. Other agencies have discovered that they have substantially more mileage than thought. This was because of the lack of detailed record keeping during a period when multiple housing developments were built within the local jurisdictions. In New York State, this translates into an increase in funding from the state through the Consolidated Highway Improvement Funds (9).

CONCLUSION

The development of and subsequent improvements to the CAMP software were a collaborative effort between multiple municipalities, many student interns, and the Cornell staff. As an inexpensive pavement management tool that is simple to use, it has allowed many smaller agencies to develop their own specific pavement management programs. At less than \$100, it provides an affordable alternative to many of the proprietary software programs that require annual use and maintenance fees. As improvements are made to the software, agencies that had previously purchased the software and/or participated in the summer intern project, are provided with the software upgrades and technical support for no extra fee. This allows the agencies the ability to improve and expand their asset management and have a resource to help them through any issues they may encounter during its use.

At the request of many of the agencies, the next step for the CAMP software is to address some glitches and inconsistencies found within, while expanding the asset offerings to include signs and culverts. At the same time, the software will be geared to link directly to a GIS format to allow the mapping of the assets. These improvements are scheduled to be completed by the spring of 2019.

Looking back, the authors have assembled their experiences, lessons learned, and future plans for the CAMP software and the intern project. As a simple tool to better management, it serves as a valuable tool in communicating the needs of the roadway network. This communication and education has not only gained the support of the public, but has proven to educate several elected boards to increase much needed funding. It is hoped that this review of the project can be used by others when working to develop their own asset management plans or assisting others doing the same.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: oversight of summer intern program, David P. Orr and Geoffrey R. Scott; interpretation of project, Geoffrey R. Scott and David P. Orr; draft manuscript preparation, David P. Orr and Geoffrey R. Scott. All authors reviewed the results and approved the final version of the manuscript.

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PAVEMENT MANAGEMENT

A Framework for Monitoring of Road Agency Performance in Rural Road Asset Management

CHARLES TONGAI BOPOTO

African Community Access Partnership

ROBERT GEDDES

MICHAEL PINARD

CDS Consultants

The Research for Community Access Partnership is providing technical assistance and capacity-building initiatives to foster sustainable improvements in asset management performance in selected rural road agencies in Sub-Saharan Africa. Central to the research methodology is the development of a specification to enable road agencies to assess their performance in asset management as a basis for self-improvement. The specification is based on the development of an objectively determinable “road sector sustainability index” which measures the extent to which six building blocks considered essential for achieving effective road asset preservation are satisfied in practice. Periodic measurements of the condition of the project road networks, coupled with the collection of socioeconomic data, are being used to monitor the trend in road asset value and to assess the effectiveness of, and improvements in, asset management as well as the impact of road condition on the well-being of rural communities. This paper outlines methodologies and tools that have been developed and piloted in four Sub-Saharan African countries to assess and monitor performance in rural road asset management and to achieve improvements over time. It summarizes progress achieved in the first 3 years since the project’s inception. The initial findings of the research indicate that severe institutional, funding, and technical shortcomings exist in the participating countries that preclude sustainable road asset preservation. However, following implementation of the methods summarized here, there is now an increased awareness of the importance of adopting a holistic approach to road asset management using simple and sustainable methods.

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**Cluster-Based Maintenance Management System for
Low-Volume Rural Roads**
Indian Experience

V. SUNITHA

SAMSON MATHEW

National Institute of Technology, India

A. VEERARAGAVAN

KARTHIK K. SRINIVASAN

Indian Institute of Technology Madras

Performance prediction models are needed for a decision support system to decide the budget for maintaining the pavements above the desired threshold performance level. In the present study, performance of low-volume rural roads constructed under the Pradhan Mantri Gram Sadak Yojana program of the government of India in the Tiruchirappalli district, Tamil Nadu State, India, are considered. An extensive visual condition survey of the study roads was conducted for a period of 3 years to assess the pavement and drainage condition. The evaluation was done on a rating scale of 1 (very poor) to 5 (very good). Factor analysis was used to formulate an index called the Visual Condition Index. The present study uses k -means cluster, which is one of the nonhierarchical methods of clustering. Five cluster groups have been developed, and separate performance prediction models were developed for each of these clusters. A separate prediction model was also developed for all the road sections without clustering. The optimization of the maintenance strategies is considered under the scenario of the Necessary Fund Model. The objective is to minimize the budget required for maintaining all the pavement sections above the threshold condition index level. The minimum maintenance treatment required to keep the pavement sections above the desired performance level and the associated cost was found to decrease, when cluster-based performance prediction models were used. The significance of clustering of pavement sections is also elucidated from the optimization results.

INTRODUCTION

In developing countries, like India, the problem of pavement maintenance is of immense significance because of limited resources. As pavements continue to deteriorate due to climate, traffic, and environmental effects, the maintenance of the rural roads through timely planning is the need of the hour under the budget-constrained scenario. The purpose of maintenance is to ensure that the road remains serviceable throughout its design life. Maintenance is important because it extends the life of the road by reducing the rate of deterioration, thereby safeguarding costly investments in rehabilitation, lowering the vehicle operating cost, keeping the road open

for traffic, and contributing to more reliable transport services and sustains social and economic benefits of improved road access (1).

Though clustering has been adopted as an efficient analysis tool in various traffic studies, it has not been integrated with a pavement management system (PMS) for low-volume rural roads. The application of cluster analysis and the development of deterioration models for roads based on clusters and their benefits needs an investigation. The pavement maintenance and rehabilitation (M&R) program should consider the choice of different M&R strategies for the project roads during the entire analysis period, the optimal timing for the application for these maintenance strategies, and the budget requirements for all the sections during the entire analysis period. The requirements of the above information and other constraints, like the threshold level of pavement performance for maintenance, limitations on the number of maintenance strategies in a year, etc., make the pavement management optimization problem more complicated.

The main objective of the present research work is to develop an optimization technique to determine the optimal M&R actions for flexible pavements grouped into various clusters based on the pavement conditions. The importance of or the need for clustering in optimization techniques for selecting the optimal M&R strategies is also studied. This paper focuses on the deterioration and maintenance of low-volume rural roads at the network level. As these roads are low-volume roads, having traffic less than 1 msa (million standard axles); the influence of traffic on the performance is not considered. In the present work, the evaluation of the optimum M&R actions is limited to flexible pavements of rural roads at the district level. The applications of the proposed models are to compute the minimum budget required to maintain the pavement sections at or above the threshold level of performance during the design period. The solution from the optimization problem can be used for planning the budget when combined with better performance prediction models.

REVIEW OF EARLIER WORK

Maintenance is generally defined as a set of well-timed and executable strategies to ensure or expand pavement life. Unlike heuristics and scenario-based methods for deciding M&R strategies, optimization techniques can provide rigorous tools resulting in cost-effective solutions, thus assisting the decision-making process and identifying solutions toward better pavement management practices. Operation research techniques and systems methods provide a scientific basis for decision makers to allocate the resources to maximize the benefit or minimize the cost.

Along this line of research, a decision support system was developed to assist pavement management agencies in M&R planning and implementation considering the road network of Greece (2). A linear model with globally optimal solution was developed for use in management of pavement M&R works. However, the problem of solving the pavement management problem was restricted to only a limited number of M&R actions (3).

An integer programming model in the Oklahoma Department of Transportation was applied for strategic planning of pavement rehabilitation and maintenance; it provided a valuable tool for the highway agencies to manage the network (4). However, integer programming becomes computationally intensive if it is applied to a large-scale road network, particularly with multiperiod decisions of pavement preservation strategies. A cost-effectiveness-based integer programming was done on a year-by-year basis for the preservation of deteriorated pavements in

a road network with the constraints of budget limitations and required pavement serviceability levels (5).

In yet another study, a new methodology was proposed wherein Markov transition probabilities, genetic algorithm, and Monte Carlo simulation have been used to address the issue of optimum decision-making and uncertainty analysis in PMSs (6). A Markovian model was developed for maximizing highway life-cycle cost by considering optimal maintenance strategies for roadside appurtenances over a given planning horizon (7). An integrated PMS has been designated to provide the pavement engineers with an effective decision-making tool for planning and scheduling of pavement M&R work (8).

Another systematic approach was formulated for the pavement maintenance management using policy iteration method to determine cost-effective maintenance strategies (9). While a number of different optimization techniques could be applied, the sequential nature of the problem (repeated applications of M&R actions over different periods) renders dynamic programming as one of the preferred approaches. A framework for a stochastic model, which is applicable where significant uncertainty exists in the deterioration models, has also been developed in another study (10).

The above review indicates that a comprehensive framework for the optimization of a network-level PMS that integrates different M&R strategies duly considering the uncertainties in the pavement performance prediction and maintenance history during the design life is needed. In this context, to reduce the uncertainties in performance prediction models, clustering of data and separate models for each of these clusters has not been adequately explored. To evaluate the optimal strategies, all feasible strategies are to be considered. Global optimization of maintenance strategies is to be performed within clusters and across different clusters. The optimal maintenance strategies to keep the pavement sections above the desired performance levels are to be developed for low-volume rural roads.

DATA COLLECTION

For the present study, 124 roads in the Tiruchirappalli district constructed under the Pradhan Mantri Gram Sadak Yojana (PMGSY) scheme were considered. The surveys were conducted on all 124 roads. The length of each road ranges from 0.5 to 4 km. The data were collected yearly at every 200-m section of the selected project roads for a period of 3 years. Out of this data, for the analysis purpose, 550 road sections were considered. The total length of road sections considered for the analysis was 110 km. The age of the pavement sections of these roads varied from 7 to 9 years.

Extensive data on the performance of these low-volume rural roads under actual traffic, climate, and environmental conditions were collected. These data were summarized for the development of appropriate deterioration models and later for their prioritization in terms of maintenance and selection of appropriate maintenance strategies. To summarize such data, several indices were used in the past, including the pavement condition index and the present serviceability index. In addition to using such data, this study uses a richer range of data that includes distress parameters and drainage parameters. The pavement distress parameters include pothole, crack, and edge break. Apart from this, to assess the drainage characteristics, additional parameters were measured, namely the camber of the pavement; condition, slope, vegetation of shoulder; drainage shape, side slope, and presence of silt; and cross drainage structures based on the settlement, erosion, and closure of openings due to silt and other debris. Therefore, there is a need for

development of appropriate indices considering the distress as well as other relevant parameters in performance prediction, since the accuracy of the indices depends on the extent and accuracy of relevant data.

In this study, an index referred to as the Visual Condition Index (VCI) is used; the index is formulated using the multivariate factor analysis technique. The scale of this index ranges from 0 to 100, where a pavement section with VCI 100 ranks at the top and the section with VCI 0 ranks the last. The weights for the different parameters are decided based on the factor loadings obtained from the factor analysis of the parameters. The detailed procedure of data collection and computation of VCI and its corroboration with field data are described in Sunitha et al. (11) and is not provided here due to space considerations.

GROUPING OF PAVEMENT SECTION BY CLUSTERING

The basic unit for any PMS is a road section. The data storage and manipulation of the pavement sections are tedious, if the road sections are considered individually. The road sections may have varying characteristics, namely dimensions, age, traffic, location, distress conditions, environmental factors, etc. These basic data units are to be grouped on the basis of some homogeneity or similarity of various attributes that are considered in the analysis. For this purpose, clustering of the road sections is done.

A nonhierarchical clustering technique was used in the present study. Due to its simplicity, the *k*-means algorithm is one of the most extensively used methods in this context. The *k*-means clustering algorithm is based on the concept of choosing a preliminary set of centroids and assigning each point to the nearest centroid. In this work, the pavement and drainage condition is used for grouping the pavement sections. The 12 attributes are crack area, pothole area, and edge break length; the left and right shoulder condition and vegetation; left and right side drainage slope and silting, if any; cross drainage condition (opening and settlement); and camber. These attributes are collected from the visual condition survey for 3 years and used in the analysis. In total, the data are obtained for 36 attributes (12 indices x 3 years) for each of the 550 road sections used in the analysis.

In the *k*-means clustering method, the number of clusters are to be specified a priori. In this work, the number of clusters is decided based on two criteria:

1. Minimizing within class variance or maximizing between class variance of clusters.
2. Minimum number of sections in any class should not be less than 5% of the total sample to ensure adequate sample size in each class.

The *k*-means clustering was done for a number of classes varying from 2 to 10. Based on the above two criteria, the number of clusters in *k*-means was determined to be five clusters, and these five clusters are used for further analysis in the development of pavement deterioration models in a subsequent section. The clusters are numbered as 1 to 5 according to the size of the cluster. The cluster composition of these five clusters is shown in Table 1, which specifies the average and the maximum distance from the centroid of the clusters. Among the five cluster groups, Cluster 1 consists of 150 sections; Cluster 2, 148 sections, Cluster 3, 127 sections; Cluster 4, 97 sections; and Cluster 5, 28 sections. Within-group variance of these five clusters varies from 12.312 to 28.872. The detailed explanation about *k*-means clustering is given in this paper (12).

TABLE 1 *k*-means Clusters Composition

Cluster	1	2	3	4	5
Within-group variance	20.414	12.670	12.312	28.872	25.515
Minimum distance from centroid	2.077	1.835	2.140	3.203	2.673
Average distance from centroid	4.357	3.453	3.382	5.218	4.651
Maximum distance from centroid	7.534	6.184	6.618	8.213	9.468
Class size (no. of sections)	150	148	127	97	28
Class size (%)	27	27	23	18	5

PAVEMENT DETERIORATION MODELS

The prediction of pavement condition is essential in any PMS. Models like regression, Markov, stochastic-empirical, etc., have been used by the researchers to predict the performance of pavement sections. In this work, multiple linear regression technique is used for modeling. Separate linear regression models were developed for the different cluster groups. To compare the effect of clustering on the deterioration models, a common deterioration model is developed for all the pavement sections without clustering. The general form of the equation is as given in Equation 1.

$$VCI_{ni} = \beta_2 VCI_{(n-1)i} + \beta_1 age_{ni} + \beta_0 \quad (1)$$

where

- VCI_{ni} = VCI of the pavement section i in Year n ,
- $VCI_{(n-1)i}$ = VCI of the pavement section i in Year $(n - 1)$,
- age_{ni} = age of the pavement section i in Year n ,
- β_2 = coefficient of VCI of the pavement in previous year,
- β_1 = coefficient of age of the pavement section, and
- β_0 = constant.

The coefficients β_0 , β_1 , and β_2 obtained for the model for clusters and nonclustered data are shown in Table 2.

TABLE 2 Deterioration Model Coefficients for *k*-means Clusters

	β_0 (Const.)	β_1 (Coeff. of age)	β_2 (Coeff. of VCI_{n-1})	R^2	p -Value
Clusters					
1	-0.395	-1.247	1.020	0.724	< 0.0001
2	-1.524	-0.972	1.029	0.823	< 0.0001
3	-7.055	-0.566	1.062	0.737	< 0.0001
4	2.155	-1.225	0.908	0.737	< 0.0001
5	5.159	-1.043	0.927	0.813	< 0.0001
Nonclustered Data					
	-5.635	-0.503	1.001	0.801	< 0.0001

Significance Test for the Cluster Models

From Table 2 in the previous section, it is seen that the cluster and nonclustered models are different as their coefficients are varying. To test the significance of this difference, the following F -test has been conducted:

$$F_{cal} = \left(\frac{RSS_{nc} - RSS_c}{p_c - p_{nc}} \right) / \left(\frac{RSS_c}{n - p_c} \right) \quad (2)$$

where

- RSS_{nc} = residual sum of squares for noncluster model,
- RSS_c = residual sum of squares for all the cluster models,
- p_{nc} = number of parameters used in nonclustered model,
- p_c = number of parameters used in cluster models, and
- n = total number of observations.

The critical value is calculated for 95% level of confidence. For rejecting the null hypothesis, F calculated should be greater than the F critical value.

The residual sum of squares (RSS) obtained for the k -means cluster models is shown in Table 3. The sum of RSS obtained for all the five k -means cluster is 21554. The RSS value for the nonclustered model is 58451. The F calculated value is 155, and the F critical value is 1.761. This implies that the k -means cluster models are significantly better model than the noncluster model.

$$F_{cal} = \left(\frac{58451 - 21554}{15 - 3} \right) / \left(\frac{21554}{1100 - 15} \right) = 155 \quad (3)$$

$$F_{cri(12, 1085, 0.05)} = 1.761 \quad (4)$$

$$F_{cal} > F_{cri} \quad (5)$$

From the model coefficients and the significant test results, the need and importance of clustering of road sections is highlighted. The deterioration models would enable development of optimal scheduling of the rehabilitation activities and to determine the funding level required to achieve a desired level of performance in the next section.

TABLE 3 RSS for k -means Cluster Models and Noncluster Models

k -means Cluster	RSS
1	7188
2	1817
3	3173
4	7167
5	2209
Total	21554
Noncluster	58451

FORMULATION AND SOLUTION OF THE OPTIMIZATION MODEL

The focus of this work is to develop decision support models for maintenance management of low-volume rural roads. Specifically, a new approach to compute the optimal timing and type of M&R actions using performance prediction models considering cluster-based and noncluster-based approaches is studied.

In this optimization problem, decision variables (timing and type of maintenance treatments) are integer values, and hence the optimization problem is formulated as a mixed-integer programming problem. The problem features and constraints are modeled through suitably specified directed network and solved using the General Algebraic Modeling System (GAMS) software. The above pavement performance prediction models are used to evaluate the condition of the pavement.

Problem Features

The important features of the proposed optimization problem are the finite horizon as well as consideration of the cluster to which each section belongs. An appropriate deterioration model is considered for the prediction of the performance of the respective cluster of the study section. All feasible combinations of strategies are to be considered so that the solution is global optimum. The results of the optimization model should also differentiate the effect of cluster-based deterioration models. Thus, the proposed methodology addresses several limitations of some pavement performance prediction models in practice.

The common optimization problem objectives specified by different agencies include the following:

1. To minimize the present worth of overall M&R expenditures over the planning horizon;
2. To minimize road-user costs by selecting and programming M&R activities to reduce disruption and delays to traffic; and
3. To maintain the highest possible level of overall network pavement condition with the resources available (13).

It is also possible to combine two or more of these objectives by assigning an appropriate weight factor to each objective.

In this work, the optimization approach for the maintenance strategies is studied to minimize the budget required for maintenance to keep all the pavement sections above the threshold condition index level. Generally, the optimization of the maintenance strategies can be considered for two scenarios. The first scenario is referred to as the Necessary Fund Model and the second as the Budget Bound Model. The first one is to minimize the budget required for maintaining all the pavement sections above the threshold condition index level. The second scenario is to maximize the benefit area under the budget constraint. In the current study, NFM is elicited.

In NFM, the funds needed to ensure a desired condition level of all pavement sections with minimal cost are considered.

Objective Function

The objective function in Equation 6 is to minimize the total maintenance cost during the analysis period.

$$\text{Minimize } \sum_n^N \sum_i^I \sum_j^J C_j \text{len}_i Y_{nij} \quad (6)$$

where

- n = year in the analysis period;
- i = pavement section;
- j = M&R strategies (do nothing, patch work, surfacing, and rehabilitation);
- C_j = cost of M&R action j per lane-km (million Rs);
- len_i = length of the section in km;
- Y_{nij} = binary indicator variable (decision variable) indicating whether action j is selected in Year n for the pavement section I ; and
- $Y_{nij} = 1$, if action j is applied to a pavement section i in Year n ; 0, otherwise.

As the study roads considered are the low-volume rural roads, the maintenance options considered are routine maintenance (crack sealing and filling, pothole patching, earthen shoulder dressing, drain desilting, etc.), periodic maintenance (surface dressing or chip seal), and strengthening (minor rehabilitation). During the patch repair work, the cracks are sealed or filled and the potholes are filled. The surface dressing (chip seal) is considered to be a resurfacing treatment and will not strengthen the pavement. The rehabilitation action includes laying granular base course layer and providing a thin (20 mm) pre-mixed carpet bituminous layer with seal coat as the surface course. The rehabilitation work includes the routine maintenance. For the analysis, the costs of the maintenance actions considered are given in Table 4.

In the necessary fund scenario, the strategy with minimum maintenance cost is chosen from all the possible alternatives. The maintenance cost is the total cost of maintenance required for keeping all the pavement sections above the threshold VCI level specified in the constraint. The threshold value of VCI can be varied according to the need and the budget availability. The maintenance cost is considered for the entire analysis period. The objective function is subjected to constraints specified in the following section.

TABLE 4 Cost of M&R Activities
(Source: Schedule of Rate, Rural Roads Department, Tiruchirappalli district, 2018)

j	Action	Unit Rate (per lane-km) in Million Rupees
DN	Do nothing	0
PA	Routine maintenance	0.15
SU	Periodic maintenance	0.7
RE	Rehabilitation	2.0

Constraints

Non-Negativity and Definitional Constraints

Y_{nij} is the decision variable. It is a binary variable that is equal to 0 or 1 (Equation 7). If the particular action or the maintenance strategy is selected, the Y_{nij} is 1, else it is 0.

$$Y_{nij} \geq 0, Y_{nij} \text{ is a binary variable equal to 0 or 1.} \quad (7)$$

In each Year n , at least one action (including do nothing) and at most one action will be chosen for each of the pavement section. That is, the total Y_{nij} should be equal to 1 for each of the pavement sections in each year.

$$\sum_j^J Y_{nij} = 1 \text{ for all } n \text{ and } i \quad (8)$$

Age of Pavement

When a rehabilitation action is applied, it is assumed that the pavement is likely to perform like a new pavement. Therefore, the age of the pavement is reset to zero immediately after a rehabilitation action as follows:

$$age_{(n+1)i} = age_{ni} + 1, \text{ if action } j \text{ in Year } n \text{ is other than rehabilitation.} \quad (9)$$

$$age_{(n+1)i} = 1, \text{ if action in Year } n \text{ is rehabilitation.} \quad (10)$$

Analysis Period

The optimization was done for short-term planning. The design life adopted for the PMGSY roads are 10 and 15 years. So the analysis period has been limited to 5 years in the present study.

VCI After M&R Action

The performance of the pavement will be improved due to the implementation of maintenance actions. The VCI of the pavement after maintenance actions (E) will be higher than the VCI value before the treatment. The VCI of the pavement after maintenance action is the quantified value of the increase in pavement performance after the treatment. The models were developed to predict the VCI value after the maintenance treatment viz., routine maintenance and resurfacing. The pavement data were collected where the M&R actions of routine maintenance and resurfacing were carried out. The VCI of the pavement sections before and after maintenance treatments have been found out. A linear regression equation is fitted between the VCI value before and after the treatment for the different treatments viz., routine maintenance and resurfacing separately. For rehabilitation, as thick granular base with a thin bituminous surface course is provided; it is assumed that the VCI will reach 100. The models for the various maintenance actions are shown in Table 5.

TABLE 5 Models for the VCI After M&R Actions

<i>j</i>	M&R Actions	Model for <i>E</i>
DN	Do nothing	$E_{nij} = VCI_{nij}$
PA	Patch work	$E_{nij} = 2.5 + 1.05 VCI_{nij}$
SU	Surfacing	$E_{nij} = 54.5 + 0.35 VCI_{nij}$
RE	Rehabilitation	$E_{nij} = 100$

where

E_{nij} = VCI of the pavement section *i* in Year *n* after maintenance action.

Deterioration Models

The general form of the deterioration models adopted for the optimization is given in Equation 1. The deterioration models will be chosen according to the cluster to which the pavement section belongs. A separate model is framed for the nonclustered data. The program is run separately for clustered and nonclustered data for comparison of the results.

Number of Maintenance Actions in the Analysis Period

The number of rehabilitation actions in the analysis period is limited to 1 (i.e. rehabilitation is provided only once in 5 years). There is no restriction imposed for other maintenance actions.

Minimum and Maximum Pavement Condition

The minimum and maximum values of VCI before and after M&R actions (E_{nij}) are 0 and 100, respectively. The maximum VCI value after maintenance is limited to 100. The minimum value of VCI after deterioration is 0.

$$E_{nij} \geq 0, E_{nij} \leq 100 \quad (11)$$

$$VCI_{nij} \geq 0, VCI_{nij} \leq 100 \quad (12)$$

Threshold Value

The minimum value of VCI after M&R action is denoted as *T*. The pavement condition at any stage after maintenance should be above or equal to the threshold value.

$$E_{nij} \geq T \quad (13)$$

The above mixed-integer optimization problem is solved using GAMS software. The problem has been solved with cluster models and noncluster models. GAMS is a software package for numerical optimization of the following types (14):

- Linear programming problems;
- Quadratic programming problems;
- Nonlinear programming problems;
- Mixed-integer problems; and
- Deterministic optimal control problems.

In this work, the CPLEX solver was used because of its efficiency in solving mixed-integer programming problems (15).

SOLUTION OF NECESSARY FUND MODEL SCENARIO

The NFM analysis was done using noncluster models and with k -means cluster models. NFM considers the objective of minimizing the total maintenance cost of all the sections for 5 years of analysis period with the constraints specified.

The concept of NFM is illustrated as follows:

Consider a pavement constructed in 2008 that belongs to Cluster 1 of the latent class cluster group. In the first year (at the age of 3 years), the VCI value of a section was 74. The length of the pavement was 0.200 km.

1. The threshold value of the pavement is kept as 80, i.e., the condition of the pavement after treatment should be above 80. In Figure 1, dotted red colour line represents the threshold value.

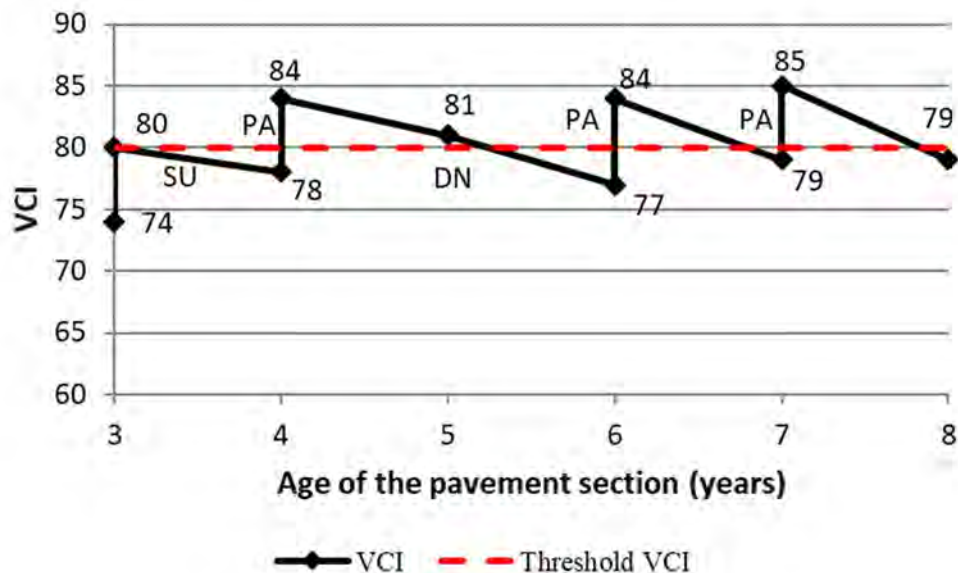


FIGURE 1 Sample performance curve for NFM.
(DN = do nothing, PA = patch work, SU = resurfacing.)

2. At 3 years of age, VCI of the pavement was 74. To keep the VCI above 80, M&R action of resurfacing is suggested, which improves the VCI after M&R action to 80.

$$E_3 = 54.5 + 0.35 * 74 = 80 \quad (14)$$

Refer equation for $j = SU$, in Table 5.

3. The $E_3 = 80$ at age 3 deteriorated to VCI 78 in the next year ($n = 4$). The deterioration model for latent class cluster – 1 is adopted for the estimation of VCI.

$$VCI_4 = -5.060 - 1.000 \times 4 + 1.085 * 80 = 78 \quad (15)$$

Refer model for Cluster 1 in Table 2.

4. To maintain the pavement condition above VCI 80, the M&R action of patch work is adopted at $n = 4$.

$$E_4 = 2.5 + 1.05 * 78 = 84 \quad (16)$$

Refer equation for $j = PA$ in Table 5.

5. At $n = 5$, the pavement deteriorates to VCI = 81 only which is above the threshold value. So at $n = 5$ there is no treatment (do nothing).

$$VCI_5 = -5.060 - 1.000 \times 5 + 1.085 * 84 = 81 \quad (17)$$

$$E_5 = 81$$

Refer equation for $j = DN$ in Table 5.

6. Similarly, the VCI and the VCI after M&R actions has been calculated for the rest of the period. To maintain the pavement condition above 80, at $n = 6$ and 7, M&R action of patch work is applied.

$$VCI_6 = -5.060 - 1.000 \times 6 + 1.085 * 81 = 77 \quad (18)$$

$$E_6 = 2.5 + 1.05 * 78 = 84 \quad (19)$$

$$VCI_7 = -5.060 - 1.000 \times 7 + 1.085 * 84 = 79 \quad (20)$$

$$E_7 = 2.5 + 1.05 * 79 = 85 \quad (21)$$

$$VCI_8 = -5.060 - 1.000 \times 6 + 1.085 * 85 = 79 \quad (22)$$

The performance curve for the particular pavement section is shown in Figure 1. The maintenance cost is calculated depending on the M&R actions.

For this pavement section,

$$C = (C_{SU} + C_{PA} + C_{DN} + C_{PA} + C_{PA}) \times len \quad (23)$$

$$= (0.70 + 0.15 + 0 + 0.15 + 0.15) \text{Rs. million/km} \times 0.200 \text{km}$$

$$= \text{Rs. 0.23 million} \quad (24)$$

Rates adopted are as given in Table 4.

In NFM, the aim is to keep the pavement section above the threshold value throughout the analysis period. The results of the optimization problem with the threshold value of 80 for all the pavement sections are discussed. The threshold value 80 means, the VCI of all the pavement sections should be greater than or equal to 80, i.e., all the pavement sections should be in very good condition. The optimal maintenance strategies will be chosen based on the above objective with the constraints. The minimum budget required for maintenance of the pavement sections in very good condition is given in Table 6. The optimization results for noncluster models and cluster models vary. This variation is due to the difference in the deterioration model coefficients of noncluster and cluster models.

The minimum budget for 110 km obtained using noncluster model was Rs. 238.45 million. For cluster models, the minimum budget was Rs. 202.64 million. That means the minimum budget required per kilometer of the road section for 5 years is Rs. 2.17 million per km for noncluster model and Rs. 1.84 million per kilometer for cluster models. The minimum budget required for cluster models is much lower than the noncluster model, which shows a reduction of 15% for *k*-means cluster. The results reflect that the minimum budget requirement varies, when more accurate cluster-specific deterioration models are used. The budget required for the maintenance strategies in each year for various cluster and noncluster models are shown in Table 7. It can be seen that the budget required in the first year is Rs. 174.92 million (73%) considering noncluster data and Rs. 184.69 million (91%) considering cluster data. As can be seen in the Table 7, the budget required is reducing over the years. This also shows the implication of investing more money in the earlier stage of the service life of a pavement section, which will reduce the fund requirement subsequently. In other words, budget required per kilometer in the first year is Rs. 1.59 million per kilometer and Rs. 0.03 million per kilometer in

TABLE 6 Minimum Budget Required for the Threshold Value of 80

Deterioration Model Type	Minimum Budget (Million Rs.)	% Variation of Budget with Noncluster Model
Noncluster models	238.45	—
<i>k</i> -means cluster models	202.64	15.02

TABLE 7 Budget Required in Each Analysis Period for the Threshold Value of 80

Analysis Year	Yearwise Budget for Various Deterioration Models (Million Rs.)	
	Noncluster Models	<i>k</i> -means Cluster Models
1	174.92	184.69
2	43.39	6.62
3	12.95	4.23
4	4.14	4.11
5	3.05	2.99
Total	238.45	202.64

the fifth year for noncluster models. Similarly, using cluster models, the budget required for the first year is Rs. 1.68 million per kilometer and Rs. 0.03 million per kilometer in the fifth year.

CONCLUSIONS

In the present study, a decision support system has been developed for the optimal maintenance management of rural road network. A new index is formulated considering the distress and drainage condition data. Separate deterioration models are developed considering the various clusters formed by *k*-means clustering technique. Different M&R strategies were considered for the different pavement sections. Separate performance prediction models were developed for each of the *k*-means clusters and the latent class clusters. A general model was also developed for all the road sections without clustering. The coefficients obtained for the cluster models and noncluster models are found to be significantly different. The above finding highlights the need and importance of clustering of road sections for maintenance management instead of considering universal performance prediction models.

The results obtained for the NFM to ensure that all the pavement sections have a threshold VCI value of 80 and above, show that the budget required for the *k*-means cluster is 15% lesser than the budget required for a noncluster-based model. Thus, the minimum maintenance required and the associated cost to keep the pavement in specified condition was found to decrease if cluster models are used. In the analysis of M&R strategies, the budget needed for the first year is 73% for noncluster-based models and 91% for cluster-based models. In the subsequent years, the budget required is less. This implies the benefits of early maintenance. The pavement can be kept in good condition by investing more funds in the early service life, which will ensure good structural adequacy and will result in improved performance and lower maintenance cost during the remaining service life.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: study conception and design, V. Sunitha, A. Veeraragavan, K. Srinivasan, and S. Mathew; data collection: V. Sunitha; analysis and interpretation of results, V. Sunitha; draft manuscript preparation, V. Sunitha, A. Veeraragavan, K. Srinivasan, and S. Mathew. All authors reviewed the results and approved the final version of the manuscript.

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**Envisioning a New Way of Managing
Low-Volume Roads in Québec, Canada**
Impact of Decommissioning on Sediment Yield

KARLELLE GILBERT
VICKY LAROCQUE
OLIVIER FERLAND
SYLVAIN JUTRAS
Université Laval

INTRODUCTION

Rights-of-Way and Low-Volume Roads

On Québec's public land, water management is an important part of road planning. There are approximately 400,000 km (250,000 mi) of roads and an average density of 1.2 stream crossings per kilometer (2/mi) (1). A large proportion of these roads are low-volume roads constructed by energy or forest-harvesting companies. Those roads are used intensively during a short period and are no longer needed except for maintenance or vegetation control. In some regions, time before next usage can go up to 10 years. In Québec, regulatory norms for those roads are the same as for every road on public land. While there are many norms regarding construction, there is a lack of norm regarding maintenance of the roads once the activity for which they were built is over. It has caused a number of low-volume roads to be abandoned over time.

A recent study conducted on 13 watersheds has shown that only 21% of the roads present were properly maintained (2). When vegetation naturally grows back, this lack of maintenance can become an accessibility issue for local users. In other cases, it can lead to erosion and washouts. The major problem appears when sediments washed out from undermaintained roads reach the hydrographic network. Because stream crossings are the contact point between roads and streams, they are hot spots for sediment input. The same study revealed that, unfortunately, 54% of stream crossings were in a mediocre state or worse (2).

Impact on Aquatic Habitat

Poorly maintained stream crossings are a threat to aquatic habitats because they can cause obstruction of the stream and become barriers for fish passage (2–5). Another common problem with stream crossings in Québec (mainly culverts) is that they often become sites for beaver (*Castor canadensis*) dam construction. These dams can cause failure of the stream crossings and flooding of the roads and can lead to significant washouts. Massive inputs of sediment to the stream can be harmful for fish by smothering spawning beds and macroinvertebrates on which they feed, causing mortality in species such as brook trout (*Salvelinus fontinalis*) and Atlantic salmon (*Salmo salar*). In the same way, siltage of stream beds can impact other aquatic vertebrates such as salamanders (4).

Rather than abandoning those roads, some managers deliberately choose to close them permanently by removing all stream-crossing structures. However, other users continue to cross the stream without proper infrastructure, causing erosion and sediment input to the stream (6). To address this problem, decommissioning of low-volume roads using improved fords is being considered. Improved fords are structures where the banks and bed of streams are stabilized with rocks to provide a stable driving surface so that vehicles can cross directly on the streambed. Improved fords were selected for this study because their construction is simple and low cost, they require minimum maintenance, and they represent low environmental risks while maintaining access to the territory. While informal monitoring has been conducted to assess the impacts of improved fords on water quality and many best management practice (BMP) guides are available in the United States on how to build these structures, not many scientific data exist regarding real impacts of improved fords.

Objectives of the Study

That is why four researchers from two Québec universities and two energy companies (Énergir and Hydro-Québec) have put together a study team. This team's main goal is to measure the impacts of decommissioning low-volume roads compared to traditional management methods. To do so, the research project is divided into four main objectives:

1. Develop a method for designing improved fords using hydraulic modeling;
2. Measure the impact of improved fords on sediment input;
3. Measure the impact of improved fords on fish passage; and
4. Make an economic analysis of decommissioning as a management method.

The ultimate goal is the production of a BMP guide that will help to implement decommissioning as a management method on low-volume roads in Québec.

The experimental designs for Objectives 3 and 4 are still being refined, and the first data will be acquired in 2019. A brief summary of Objective 1 concerning fords design is being presented. But since a lot of information is already available on BMP for construction of fords, this paper focuses on Objective 2, which addresses the impact of improved fords on sediment input to the stream. It is divided into two objectives: (1) measure sediment input from vehicle crossing and (2) measure sediment input from construction of the improved ford.

METHODOLOGY

At present, four stream crossings have been selected for the study. They are located in Pessamit, a First Nation reserve in Côte-Nord, Québec. On each site, there used to be a culvert that got washed out due to natural deterioration, beaver dam construction, or high flow. They are located on watersheds varying between 4 and 20 km² (1.5 and 7.7 mi²). Study sites were selected according to water depth (max. 50 cm), river width (max. 8 m), and proximity to usable road network. Local residents used the site to cross the stream regularly without any stabilization of the river bed or banks. Improved fords have been constructed between October and November 2018. Figure 1 and Figure 2 show two of the sites before and after construction.

Design of the Improved Fords

For Objective 1, hydraulic modeling using HEC-RAS software was performed in order to design the improved fords. The diameter of the rocks used and the length of riprap was determined by the shear force of the flow with a recurrence period of 10 years at the location of the crossing. Streambed level after construction was designed to be lower than the initial streambed level to ensure fish passage even in low-flow periods. The rocks used were clean and angular. Construction work was done in less than a day for each crossing and only required an excavator and a dump truck to get the material on site.



FIGURE 1 Site 51 before and after construction work, August 2018 and October 2018.



FIGURE 2 Site 101+100 before and after construction work, November 2018.

Measurement of Sediment Input

To measure sediment input from improved fords, two things are taken into account: sediment input from vehicle crossing and sediment input from construction of the structure itself. Passage-induced sediment was measured before construction (on natural ford) and will be measured after construction (on improved fords) in summer 2019. The methodology used is the same for both cases.

On each site, a water depth sensor (KPSI 710S) was placed at the bottom of the stream taking readings every minute. A cross-sectional profile of the stream was made using a high-precision altimeter (Zipline PRO-2000). Flow velocity was measured using a portable velocity meter (Hach FH950). Water flow was then computed.

Turbidity sensors [Analite NEP9510 (0-5000 NTU)] were installed upstream and downstream from the crossing, taking readings every second. The median value was computed for every minute. Water samples were taken manually close to the sensors at different turbidity values. On each site, 30 water samples of 500 mL were taken downstream from the stream crossing and 10 were taken upstream, where variability is minimal during construction time. An automatic water sampler (ISCO 6712) also took 500 mL samples every 5 min during construction to ensure that samples were taken at a gradient of turbidity values.

Water samples were later filtered using 0.7 μm filters (Millipore) and dried (at 65°C for 48 h). Total suspended sediment (TSS) were weighed to get TSS concentration for every sample. A regression between turbidity and TSS concentration will be calculated. Total induced sediment load for the duration of the construction work will be calculated using the method described in Lane and Sheridan (7) and Lewis and Eads (8).

CONCLUSION

Results obtained will be used to compare sediment input from vehicle passage on natural and improved fords. Sediment load associated with the construction of the improved fords will also be compared to sediment input from washouts on abandoned roads to compare each management method's impact on the aquatic ecosystem. Data from summer 2018 are still being analyzed, and more data will be acquired in 2019 to draw conclusions that are more representative. Thus, no results are available for publishing at the moment.

For now, data have been collected on sites where environmental damage had already occurred due to the washout of culverts and road material. Of course, the ultimate objective of the project is to be able to think ahead and implement alternative stream crossings in the planning stage of every road construction project. For the years to come, the research team will focus on measuring the impact on sediment input of a combination of temporary bridges (for the intensive use period) and improved fords (for low-volume traffic). This will help build a new long-term vision of road management in Québec.

This research project is a unique partnership opportunity for energy companies, universities, and provincial and federal government involved. It also has led to the formation of five graduate student programs in water management methods, which is of direct interest for the industry. Results obtained will help improve management methods in Québec's public land while taking into consideration operational constraints and environmental impact. We thank our partners for their expertise and support.

For any insights, recommendations, or questions, please do not hesitate to contact us at Karelle.gilbert.1@ulaval.ca.

ACKNOWLEDGMENT

We thank our main industrial partners, Hydro-Québec and Énergir, for their interest in the project as well as for their technical expertise and financial support. This project would not be possible without the support of Québec's Ministère des Forêts, de la Faune, et des Parcs and the Ministry of Fisheries and Oceans Canada. This project was funded through the Collaborative Research and Development program of the Natural Sciences and Engineering Research Council of Canada.

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Road Surfacing 2

Nuggets from Minnesota's Research, Implementation, and Deployment of Pervious Concrete in Low-Volume Roads

BERNARD IGBAFEN IZEVBEKHAI

Minnesota Department of Transportation

In a series of research and implementation initiatives, Minnesota Department of Transportation (DOT) constructed various pervious concrete test cells and monitored their strain, acoustics, ride quality, and hydraulic conductivity at MnROAD research facility. Subsequently, Minnesota DOT constructed a boat ramp in Detroit Lakes, while City of Shoreview constructed 3,200 ft of city streets with full-depth pervious concrete. The MnROAD Test cells were instrumented with strain gauges, thermocouples, watermarks, and maturity data-loggers whose information was loaded directly to a database. Early results facilitated improvement in subsequent mix design and maintenance practices. Mechanical and durability properties exhibited statistically significant difference from normal concrete. This research also accentuated the detrimental effects of clogging on mechanical and acoustic properties. The city streets provided higher benefit–cost ratio than the conventional asphalt option with drainage infrastructure. The Detroit Lakes boat landing and filtration system is still providing its design function to the community after 10 years without any visible distresses. Certain acoustic properties and ride quality were successfully restored and improved by diamond grinding of one of the MnROAD Test cells. This paper establishes the adequacy of pervious concrete for local roads when the subgrade soils are granular and when adequate proactive maintenance is performed.

INTRODUCTION

Background

Sandberg (*1*) defines pervious concrete as one in which the porosity is the higher than 18%, where semipervious pavements exhibit porosities of 15% to 18%. Functionally, a pervious concrete contains a matrix of communicating voids in a nonsegregated distribution of cavities that allows downward and lateral flow of water through them. Whereas porosity is employed in the definition of pervious concrete, the tortuosity, which is a function of the actual void distribution and the imparted aggregate configuration, has additional influence on the flow properties of the media. Consequently, a gravimetric evaluation or definition of pervious concrete placing the density at 100lb/ft³ to 125lb/ft³ must be complemented by a description of the communicating voids either directly or by proxy, through acoustic properties. Voids are described as communicating when they are at such optimal distances and of such optimal size that tortuosity is minimized without compromising intergranular bond. The porous structure can attenuate acoustic impacts by diffraction of incident sound at the surface and through multiple reverberations within the cavities. Certain watersheds had instituted limits to the percentage of stormwater allowed into certain lakes and rivers. Since pervious concrete pavements exhibit high hydraulic conductivity, they were deemed suited for certain local or low-volume roads (LVRs)

where they also provide cost savings through the minimization of hydraulic structures. Pursuant to this challenge, the Aggregate and Ready Mix Association of Minnesota collaborated with Minnesota DOT to initiate studies on pervious pavements to facilitate quick deployment. This led to a series of research initiatives (2–5). This paper thus catalogues the previous research initiatives and discusses monitoring in the test cells. Low-noise characteristics are among the merits of pervious pavements (1).

Synthesis

At the incipient stages of pervious pavement mix designs and structural designs, it was regarded as no-fines concrete. The initiatives that were built at the time by others and up until the pervious driveway at MnROAD (Cell 64) and the pervious sidewalk were characterized by zero fine aggregate content (2). Subsequently it was opined that the fine aggregate may increase the paste strengths between cavities and around aggregates (3). This led to the use a gravimetric 6% sand in the mix design of the Local Road Research Board–initiated MnROAD Test cells in 2008 (5). Additionally, the pervious overlay had in addition to 6% sand content some cellulosic fibers and polypropylene fibers. The former actually deployed nanoscopically into the mix, while the polypropylene provided fiber reinforcement of the paste. These additions were believed to have strengthened the intergranular bonds and the interfacial bond with the substrate. Evaluation of rheological properties went through series of developments, while agencies used the nuclear backscatter method for in situ density evaluation. In a closely packed aggregate system, if the aggregates and surrounding mortar are idealized as spheroidal and of a single radius r , it can be shown that the optimum packing efficiency as of a body-centered cubic approximates to 75%. Maximum porosity is thus $1-0.75$ which is approximately 25% when voids are communicating. Additionally, Neithalath (6) showed that beyond that optimum void content there may be a reduction in strength. Izevbekhai (7) showed that if γ is the bulk density of a porous layer and γ_{np} is unit of a nonporous layer, consider a unit volume of the pervious matrix it can be shown that

$$\gamma (V_v + V_s + V_e) = \gamma_{np} (V_s + V_e) = \gamma_{np} (1 + V_v) \quad (1)$$

where

V_v = volume of porous cavities,

V_s = volume of solid, and

V_e = volume of entrained air.

Since $V_v + V_s + V_e = 1$, then bulk density $\gamma = \gamma_{np} (1 - V_v)$. Consequently,

$$\text{Porosity } (n) = V_e + V_v \text{ from where Porosity } (n) = (1 - (\gamma / \gamma_{np})) + V_e \quad (2)$$

The unit weights are determined from the nuclear density backscatter method, and V_e is estimated from the specified entrainment λ proportioned to the solid matrix alone (γ_{np} / γ),

$$V_e = (\gamma_{np} / \gamma)\lambda \text{ and consequently } n = (1 - (\gamma / \gamma_{np})) + (\gamma / \gamma_{np})\lambda. \quad (3)$$

Thus, porosity is important, but is more predictive of hydraulic conductivity when construction practices minimize segregation and poor mixing. Metamorphosis has occurred also in the measurement of hydraulic conductivity. Initially, a sand cone device was used by researchers at MnROAD, but subsequently many practitioners developed single- and double-ring infiltrometer (2, 3). Minnesota DOT developed a valve-less infiltrometer, which was later modified to a valve-controlled falling head “Perveammeter” (5). Along with the Perveammeter, the term “dissipated volumetric rate” (DVR) was also developed by MnROAD researchers as a measure of hydraulic conductivity based on the dimensions of the tube (8). The challenge to measure acoustic benefits led to the acquisition and modification of an impedance tube for sound absorption measurements. Analyzing the principle behind the impedance tube (5), the following can be shown:

$$|R_p(f)|^2 = 1 - \frac{1}{k_r^2} \left| \frac{P_r(f)}{P_d(f)} \right|^2 \quad (4)$$

where

K_r = spreading factor,
 P_r = reflected energy, and
 P_d = incident energy.

The output of a sound absorption coefficient is typically in the form of the sound absorption at any of the seven one-third-octave-frequencies.

Objectives

This paper discusses the various pervious concrete initiatives and the resulting implementation initiatives in Minnesota between 2005 and 2017. It describes the technologies employed in monitoring these test cells and test sections and discusses the various performance variables that characterized the monitored test cells.

RESEARCH SIGNIFICANCE RESEARCH INITIATIVES

Research Significance

Whereas the body of knowledge on pervious concrete had addressed mix design construction issues, there is a conspicuous paucity of long-term performance analysis and a correlation of such performance to rheological mechanical and environmental variables. There is also a dearth of actual trafficked pavement sites. Pursuant to this, some comparative analysis between pervious and nonpervious concrete (built at the same time) is performed. A discussion of long-term performance of test cells and built infrastructure is provided. Pervious pavements are primarily designed to facilitate stormwater infiltration and noise reduction. However, these benefits are constrained by clogging of the pores. It is easy to perform in situ hydraulic conductivity and acoustic measurements to ascertain the hydrologic and acoustic health of pervious concrete. Consequently this research takes a holistic approach: research implementation and long-term performance.

MnROAD Research Facility

The MnROAD mainline consists of a 3.5-mi 2-lane Interstate roadway carrying live traffic. The mainline consists of various pavement designs constructed in 1993, but currently has 70 test cells. Parallel and adjacent to the mainline on Interstate Highway 94 is the LVR, a 2-lane, 2.5-mi closed loop where traffic is restricted to an operated 80 kilo-pound 18-wheel, 5-axle-tractor/trailer making 80 laps a day for 5 days a week.

Pervious Concrete Driveway MnROAD Cell 64

Cell 64 was constructed in late September 2005 in a partnership agreement with MnDOT and the Aggregate Ready Mix Association of Minnesota. Although detached from the MnROAD mainline and LVR, Cell 64 is a part of the overall MnROAD facility. It is located on the south side of the MnROAD pole barn, cut out of a bituminous parking lot. A 60-ft by 16-ft test cell surrounded by a 2-ft concrete curb on all sides, this cell was designed for an anticipated 23% air and zero-slump. A special roller vibrator (Figure 1) was used during placement to avoid bleeding and loss of porosity. An L-shaped 4-in. perforated pipe enclosed within an aggregate filter collected the infiltration and facilitated volumetric evaluation of infiltration discussed by various authors (3–5). The final iteration of this infiltrometer known as the Minnesota DOT Perveammeter is shown in Figure 6. It was loaded two repetitions per day, 5 days a week by the 80 kilo-pound 5-axle semitrailer. The pervious concrete driveway was subsequently monitored, but most of the driveway became irredeemably clogged after 4 years of no vacuuming (2).

Pervious Concrete Test Cells

Cell 85 and Cell 89 were full-depth pervious concrete test cells on granular subgrade and cohesive subgrade respectively. The mix was designed with a porosity between 15% and 18%,



FIGURE 1 MnROAD pervious concrete driveway: placement and motorized screed compaction of pervious driveway.

unit weight less than 135 pcf, and 7-day flexural strength of 300 psi. Construction began on October 17, 2008, using fixed form pavers and roller screed. Each cell was instrumented with vibrating wire static strain gauges, maturity sensors, thermocouples, and water marks at various depths to detect strain, maturity, and freeze–thaw cycles. The difference between the two cells is the full infiltration scenario with a granular subgrade in Cell 85, and a detention system with the cohesive subgrade in Cell 89 (Figure 2). The pervious pavement in both cells is 7 in. thick, over 4 in. of railroad ballast and 12 in. of a gap-graded base. It is noted that in order to accommodate paving traffic, the 4-in. railroad ballast had been placed in replacement of a 4-in. undercut from the 12-in.-thick 1-in. nominal-sized single-graded porous base.

The cell was allowed to cure for 28 days after using a biodegradable curing compound called “Confilm” and two layers of polyethylene sheeting. Prior to construction, subsurface exploration revealed an aquiclude 32 ft from the surface at the west end of the two test cells to be constructed. This tapered rapidly to the east end of the test cell, where the same aquiclude made up of very dense clay was only 3 ft from the surface. Test Cell 85 was therefore built as it were over a granular subgrade, while Test Cell 89 at the east end was built over a clay subgrade. The concrete mix design was common to both cells made up of granite aggregate and 6% sand to enhance bond. This was preceded by the placement of 6-in. curbs with 10-ft joint spacing that was adopted in the pervious concrete as well, using a tooled joint indentation made over the layers of polythene sheeting after the application of biodegradable curing compound. Mix design and other materials specification details were copiously presented in a detailed report (5). These test cells were monitored for 9 years and ultimately were replaced for other experiments in 2017. Monitoring and observations are discussed in a subsequent section.

Pervious Overlay

This initiative built a 4-in. pervious concrete over a pre-existing nonpervious concrete substrate and constructed french drain shoulders to allow for lateral drainage as MnROAD Cell 39 in 2008. The concrete industry, through the Concrete Pavement Technology Center, had proposed this as a low-noise overlay solution. The mix design consisted of 3/8-in. granite aggregates 6% sand and for further bond enhancement polypropylene and cellulosic fibers (9). Until the pavement was replaced in 2017, it received many repetitions of LVR traffic and experienced

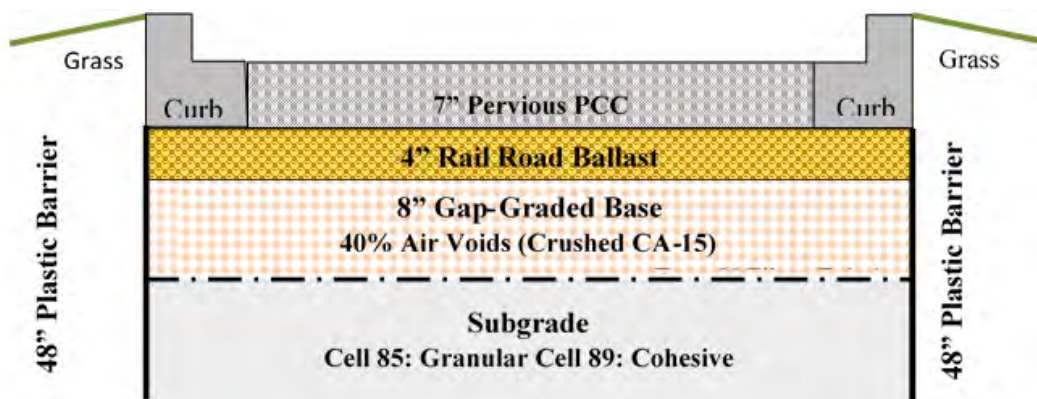


FIGURE 2 Cross-section through full-depth pervious test Cells 85 and 89.

gradual debonding from the substrate, but no longitudinal cracks as in the full-depth cells. During paving, the screed introduced 10-ft scallops that presented as poor ride quality. The surface was diamond ground in 2013 and showed marginal improvements in acoustics in the environmental lane, but measurable improvements in the traffic lane. Both environmental lane and traffic lane experienced tremendous ride quality and hydraulic conductivity improvement.

DEPLOYMENT AND OBSERVATION

Deployment initiatives included the MnROAD sidewalk, the City of Detroit Lakes boat landing, and the City of Shoreview City streets. The Minneapolis cul-de-sac project preceded the mix design and monitoring technology developments and traveled extensively after the first winter. The raveled material clogged the pores that were never vacuumed. That was another example of the potential for irreversible clogging if vacuuming is not initiated early.

MnROAD Pervious Sidewalk

A sidewalk with various pervious mix designs was placed at the MnROAD facility in 2006. This sidewalk provided a basis to examine aggregate types, sand versus no-sand and the use of fibers. It ignored excavation to remove unsuitable soil and represented minimal efforts homeowners would make to build sidewalks. The only removals were to the bottom of the pervious aggregate base. This initiative created the improvements of pozzolanic substitution and addition of 6% sand. However some deployment in the form of an industry-conceived and -built sidewalk at the same research facility had preceded this mix design improvement. The driveway consisted of 4-in. pervious concrete over 8-in. pervious CA 50 base. CA 50 consists of 1-in. single-sized aggregate. There were six segments of the sidewalk. Segment 1 (12 ft long) consisted of pervious concrete made from limestone. Segment 2 (21 ft) was made pervious concrete containing gravel aggregates. Segment 3 was made up of pervious concrete from 3/8-in. granite. Segments 4 and 6 were the nonpervious control, and Segment 5 was pervious concrete with granite aggregates. Not much was done in the area of soil correction or to ensure that the subgrade was drainable (Figure 3). Some aspects of the driveway did not withstand freeze–thaw cycles and were removed very early.



FIGURE 3 Pervious concrete MnROAD sidewalk with cross-section (*right*).

City of Detroit Lakes Boat Landing

Minnesota DOT built a boat ramp and filtration system along the US Highway 10 Frontage Road tangential to the banks of the Detroit Lakes in the City of Detroit Lakes in 2008. The pavement used the MnROAD Test cell mix design to build the pavement (cross-section in Figure 5) as a unique filtration design. The pavement used the MnROAD Test cell mix design to build the pavement (cross-section in Figure 5) as a unique filtration design. Boats from the lake drip water into the pervious concrete that flows by gravity through the porous Minnesota DOT CA-50 base into a separating geotextile that is inclined at a 1:6 slope to a detention pond (Figure 4) (5). Water in this pond is treated periodically and released into the Detroit Lakes. This unique filtration system has been functioning for more than 10 years. Hydraulic conductivity has decreased since construction, but not irreversibly as postvacuuming results are still better than prevacuuming results.

City of Shoreview City Streets

City of Shoreview built the Woodbridge neighborhood local roads using pervious concrete pavements in 2008. The pervious concrete was designed for an anticipated 23% porosity and zero-slump. The design (Figure 5) included a 7-in. pervious concrete pavement built over

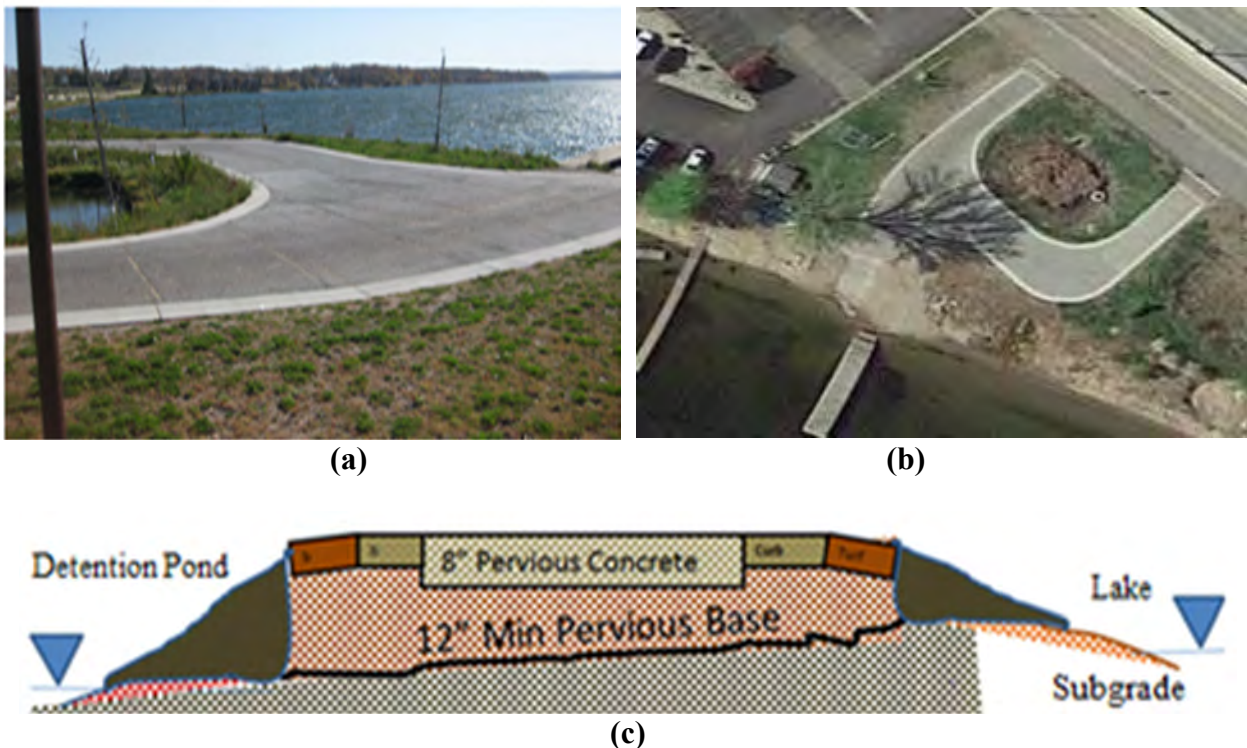


FIGURE 4 Detroit Lakes boat landing and filtration system: (a) portion of the boat landing showing joints; (b) aerial view of boat landing on U.S. Highway 10; and (c) cross-section showing simple filtration system design.

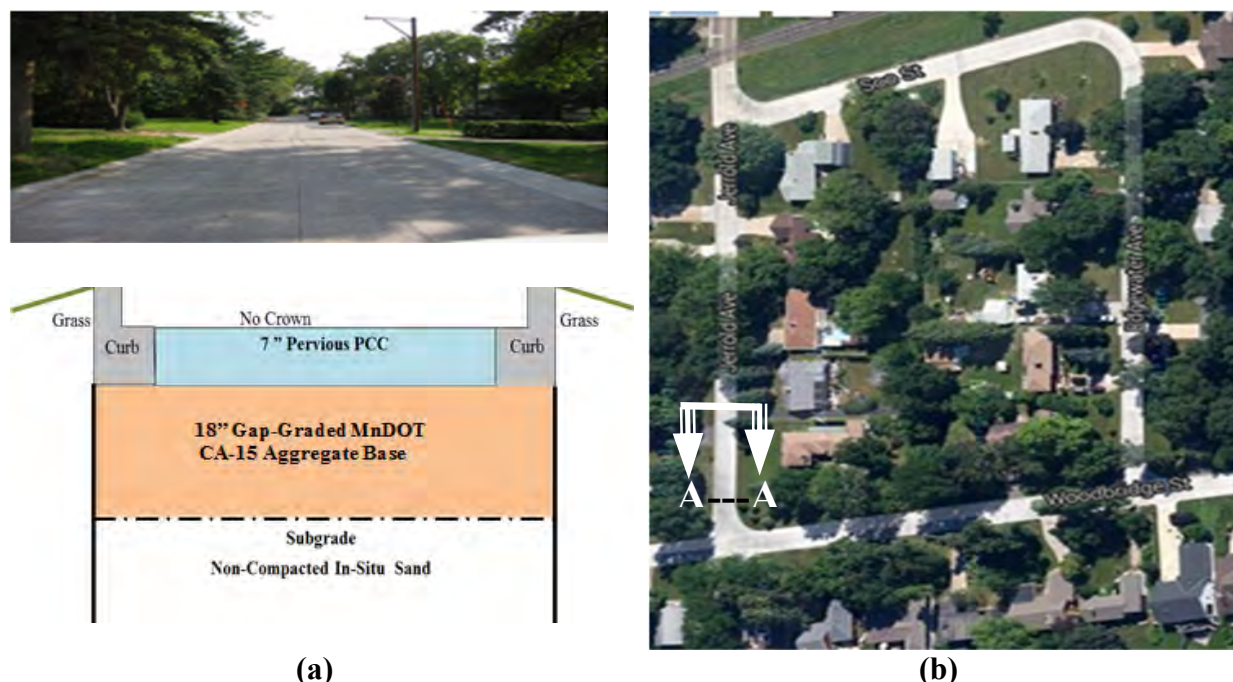


FIGURE 5 Aerial view of pervious pavements in Woodbridge neighborhood Shoreview: (a) section AA through Jerrold Street and (b) aerial view of Woodbridge pervious cells.

18 in. of pervious base. In the proximity of Owasso Lake, the subgrade materials are very drainable sands that allowed direct infiltration of runoff (Figure 5). This deployment was carefully monitored by Minnesota DOT researchers for 7 years until a detailed 7-year performance report was rendered (10). Researchers compared the cost of pervious concrete to the nonpervious alternative with hydraulic infrastructure. The City conducted a cost analysis with two confliction options (10). The first option was a bituminous reconstruction with hydraulic structured, and the second was the pervious concrete without hydraulic structures. The decision machinery showed the pervious concrete to be of lower investment cost. Researchers conducted a life-cycle cost including vacuuming every quarter and diamond grinding the 10th year (10). They also showed lower life-cycle cost for the pervious concrete alternative (10).

MONITORING

For convenience the monitoring is discussed under MnROAD monitoring, which is a routine testing at the MnROAD facility, and the network test section monitoring outside of MnROAD.

Monitoring at the MnROAD Facility

The MnROAD Test cells were instruments with thermocouple trees consisting of watermarks and thermocouples arranged as branches on a stem driven in to the subgrade from the pavement surface providing freeze–thaw cycles. Each test cell was also instrumented with vibrating wire strain gauges that provide top and bottom strain in the concrete. Previous research discussed the freeze–thaw cycles accentuating how pervious pavements facilitated geothermal balance (3, 4, 11). Sensor

readings as with all other cells in the MnROAD facility were downloaded directly into the database. Typical routine monitoring at the MnROAD facility includes skid resistance (ASTM-E274) ride measurement, sound absorption measurements, falling weight deflectometer, and distress surveys. On some occasions, MnROAD operations had borrowed the grip tester for some years and made continuous measurements round the test cells. Ride measurements are conducted at least three times a year through all the test cells in a continuum. Files are later cropped into pavement test cell groups. Hydraulic conductivity tests began with the sand cone and developed to the single-ring infiltrometer that was improved into a gate-valve-Perveammeter (Figure 6) (5). Impedance tube for sound absorption in situ is based on the degree of absorption than the pavement surface does to a white noise intensity supplied by the source at the top end of a tube when the tube is placed over the pervious pavement. For ride measurement, the lightweight profiler at the time of monitoring was outfitted with a line laser and a triple laser so that the effect of texture would be detected when such textures exhibited anomalous ride properties. Such features were found to be more pronounced in longitudinal textures, whereas pervious pavements texture is isotropic. At least three times a year MnROAD operations measures tire-pavement interaction noise with the On-board Sound Intensity Method (OBSI) (5). All acoustic datasets are analyzed in the 3rd octave band. Other technologies used are also shown in Figure 6.

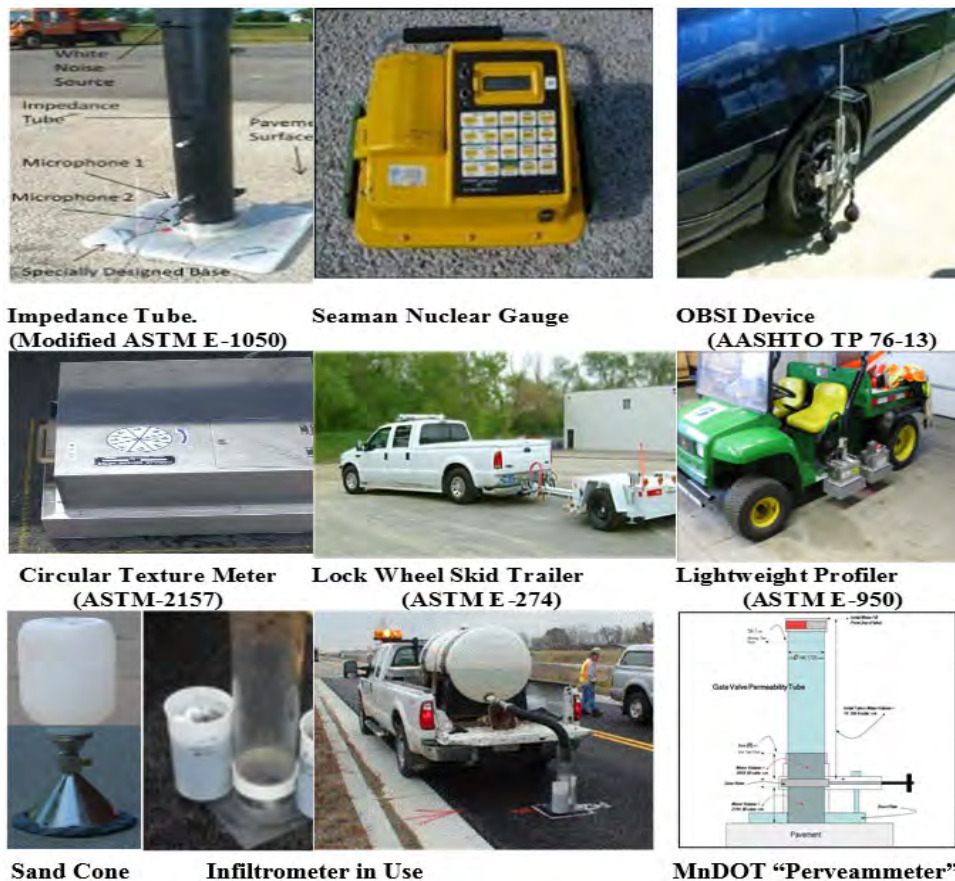


FIGURE 6 Surface evaluation technologies, sand cone, single infiltrometer, and gate-valve Perveammeter built by Minnesota DOT researchers for hydraulic conductivity measurement.

Deployment Test Section Monitoring

As needed by the researcher, sound absorption, hydraulic conductivity, and ride quality (IRI) measurements (where feasible) were performed twice a year on average on the deployment sections exterior to MnROAD. OBSI was not measured in the city street as the speed limit would not allow the testing speed of 60 miles per hour in such a community, even if traffic was diverted. Some general observations from monitoring are discussed in a later section.

DISCUSSION

Many characteristics of pervious concrete have been discussed by author and by others (3, 5, 9–11), but some major observations have neither been mentioned nor observed until now. This research provides the advantage of long-term monitoring and the corresponding long-term performance information. This section succinctly discusses associated key observations. Many pervious concrete initiatives were initiated in Minnesota, but those where Minnesota DOT's expertise was not sought did fail early because of inadequate mix design and absence of vacuuming.

Skid Resistance of Pervious Concrete

Pervious concrete provides as much skid resistance as the any skid-resistant texture in the network. Skid resistance from the grip tester measured in pervious overlay Cell 39 pervious and pervious full-depth Cell 85 and Cell 89 were compared to those from Cell 53 transverse broom built at the same time in the traffic lanes in LVR. The pervious pavement surface exhibited higher friction numbers than the nonpervious surface (Figure 7). Here, in a reliability analysis, the 63.2nd percentile grip number (GN) is proxy for the characteristic value μ of coefficient of friction of the pavement surface. Results showed a characteristic GN of 0.47 for full-depth pervious, 0.55 for pervious overlay, and 0.3 for transverse broom drag.

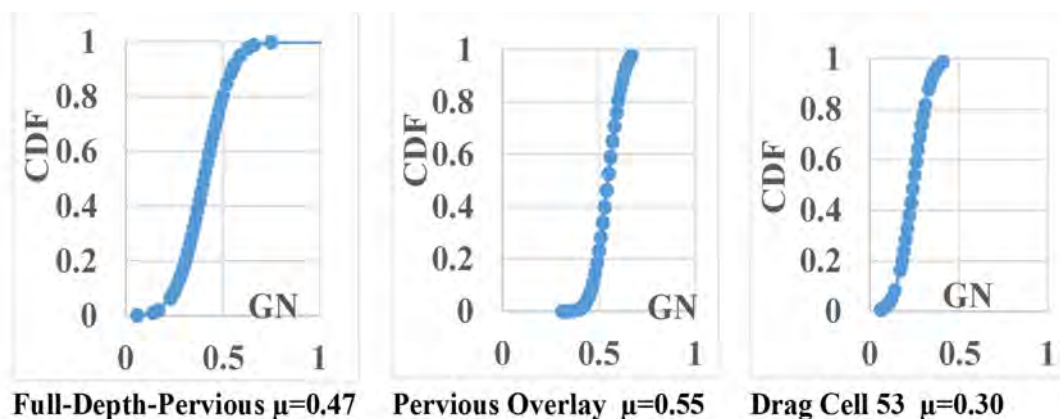


FIGURE 7 Skid resistance CDF rank for GN.

Mechanical Properties of Pervious Concrete

During construction of pervious test cells, test cylinders and beams were made with the prevailing industry standard, cured and tested for compressive and flexural strength. The 7-day strength of pervious specimens were 90 to 99 percentile rank of the 28-day flexural and 80 to 95 percentile rank of 28-day compressive strength unlike the nonpervious specimens, where 7-day strengths were in all cases 85% to 100% of the 28-day flexural and compressive strengths. The 7-day, 21-day, and 28-day strengths were statistically analyzed using the cumulative density function. Results (Figure 7) showed a much higher overlap of pervious and nonpervious compressive strength distributions compared to the tensile strength distributions. From this experiment, pervious concrete tended to be more susceptible to tensile stresses than compressive stresses. However the curves (Figure 8) show reduced flexural and tensile strengths for pervious pavements. Aggregates are not completely encircled by paste in pervious concrete. Consequently, in pervious pavements the interfacial transition zones are even thinner and weaker than in conventional pavements pervious pavements. However, they proved sufficient strength for low-volume traffic, parking lots, and drainage systems (5).

Diamond Grinding As a Maintenance/Improvement Strategy

In a most recent initiative, the pervious concrete overlay cell was diamond ground. Although pervious pavements are among the quieter pavements, there is a preponderance of features such as texture direction and texture orientation that when optimized render nonpervious pavements as quiet as and even quieter than pervious pavements. In this regard a diamond grinding initiative only marginally improved OBSI (measured tire–pavement interaction noise) in the cells that were not traffic loaded that were consequently less deteriorated than the traffic-loaded adjacent lane. In that lane, however, the improvement in acoustic properties due to diamond grading was significant. The diamond grinding improved ride quality considerably, but had very little impact

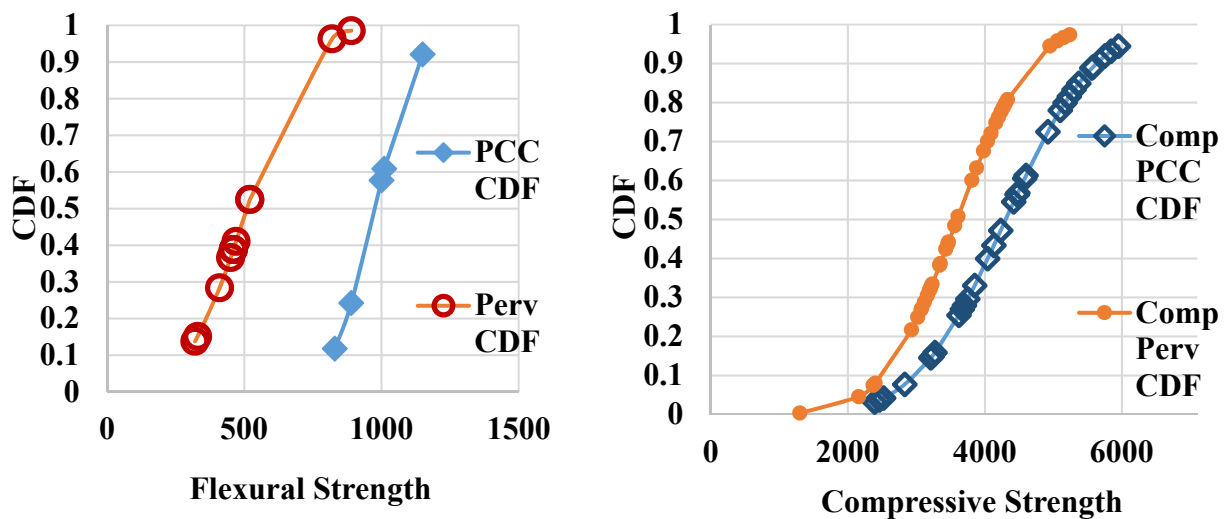


FIGURE 8 Compressive and flexural strength (pervious versus portland cement concrete).

on friction since friction of pervious pavements is comparable to the 80–99 percentile range of friction numbers in the Minnesota network (7).

Long-Term Drainage Properties

DVR varied significantly throughout each cell, suggesting uneven material consistency. However, flow rate (DVR) was generally higher in the cell built over sand than in the cell built over clay subgrade (Figure 9). DVR was a convenient measure of the rate of discharge through the pavement using the single-ring infiltrometer. This observation was associated with the distress survey observation that showed early raveling of the cell built over cohesive subgrade (Cell 89) as well as pronounced cracking on the wheel path of the traffic lane. There were cracks in Cell 85, but they showed up after 7 years. The degree of raveling was insignificant in Cell 85 compared to Cell 89. These distresses were minimal in the boat landing and the city streets of City of Shoreview that were built on granular subgrade.

Sound Absorption and Clogging

Sound absorption proved to be a proxy for structural and environmental health of a pervious pavement and showed tremendous correlation to hydraulic conductivity and durability since lower sound absorption implied clogging and susceptibility to freeze–thaw damage (12, 13). Figure 9 (middle) shows a comparison of ratios of sound absorption (α) of pervious to nonpervious at various frequencies. Interestingly it peaked at 1,000 Hz, which is the resonant frequency for tire–pavement interaction noise. This attribute can be harnessed for traffic noise reduction. Observing the detriment of fine clogging agents in the upper 2 inches, an experiment injected a portion of the pervious concrete with increasing volumes of clogging agent and measured corresponding sound absorption. The result (Figure 9) showed that with the same volume of clogging agent, clay had the highest decrease in sound absorption. The driveway

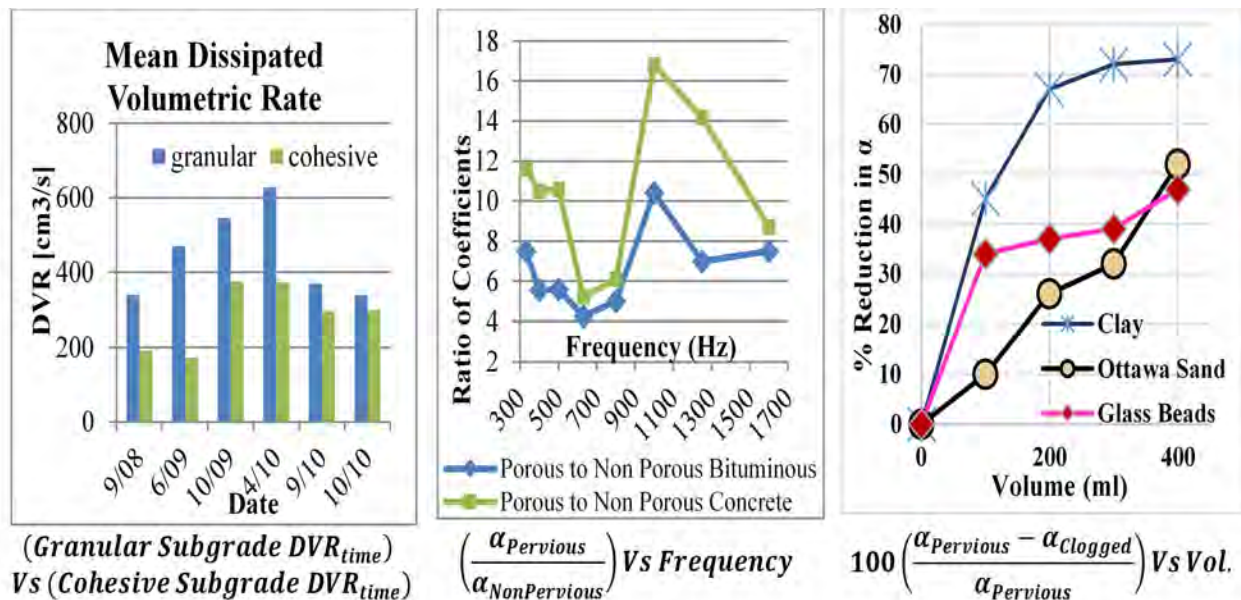


FIGURE 9 Hydraulic acoustic and clogging characteristics.

experienced severe raveling, spalling, and loss of mechanical strength shortly after placement. This likely occurred because the pavement was clogged thus trapping water that became ice lenses that damaged the matrix. Clogging resulted in early raveling as well as loss of mechanical strength properties as observed from Schmidt Hammer spot tests and hydraulic conductivity tests (3, 5). Coagulation of clogging agents over years of not vacuuming had resulted in irredeemable clogging of Cell 64, whereupon the pavement could not respond to regurgitative hydraulic flushing and vacuuming thereafter. Most of the clogging agents were from raveling of the aggregate and associated paste degradation.

Long-Term Distress Cracks from Distress Surveys

A large longitudinal crack propagated down the centerline of the driveway and was likely a result of falling weight deflectometer testing, traffic, and thermal loading. The pervious overlay rapidly increased from 15% debonded at interface in the first month to 30% debonded in the 5th year when it was diamond ground. The diamond grinding vacuum being as efficient as any regurgitative system and tremendously improved hydraulic conductivity and sound absorption. However, a much smoother surface, 70 in. per mile versus 300 in. per mile, reduced the dynamic loads and decelerated the rate of debonding. A 2017 forensic observation revealed only 35% debonding and no longitudinal cracks of the pervious overlay cell, but the wheel track of the traffic lane of the pervious pavement built on clay subgrade (Cell 89) exhibited continuous longitudinal wheel-track cracking. There was spalling at the tooled joints of the cell built on sand subgrade (Cell 85), but the minimal partial-depth longitudinal cracks were not connected. A major crack at the centerline of the driveway in 2006 had indicated that there should have been a longitudinal joint at the driveway centerline. Proper joint establishment cannot be ignored in pervious pavements. Some areas of obvious freeze–thaw damage were repaired in Cell 89, and the sidewalk where cohesive subgrade materials underlay the pavements. Joint spalling was more pronounced in the tooled joints (MnROAD) than in the sawed joints (Shoreview and Detroit Lakes). In spite of the apprehension of autogenous rehealing of the slurry from sawing and the potential for irreversible clogging, the sawed joints performed better than the tooled joints. The former was not associated with slurry-induced clogging. Most of the clogging agents recovered were fines from raveling of the concrete (5).

Hydrologic Evaluation and City Street Benefits

Izevbekhai and Schroeder (10) identified that vacuuming was indispensable to pervious concrete performance and suggested diamond grinding as a long-term improvement at the right time. Comparing the city street to the nonpervious option that included hydraulic structures, they found the long-term pervious option to be more cost-beneficial. The Woodbridge neighborhood had the right granular subgrade and proximity to Owasso Lake. In 7 years, more than 1 acre-ft of runoff had infiltrated the pavement (10).

CONCLUSION

This paper has catalogued the various pervious concrete research and deployment initiatives in Minnesota. Routine vacuuming of pervious concrete must proceed adequate storage detention,

structural, and transmissivity design to minimize susceptibility to freeze–thaw damage. Pervious concrete is better situated over granular subgrades that allow downward flow and provide geothermal balance that in turn reduces the need for snow and ice operations. The early failure of other pervious initiatives, for which Minnesota DOT’s expertise was ignored, buttresses the need for this level of care. Moreover, the worst performing test cells and deployments are those that were built over cohesive subgrade. In this research, pervious concrete did not statistically attain to the compressive strength and flexural strength of conventional concrete, but the reduction in overall project cost arising from the exclusion of hydraulic structures renders them adequate for LVRs with granular subgrade materials. Long-term optimally-timed diamond grinding is also effective in restoring ride comfort and hydraulic conductivity in general and may improve pavement quietness when the pavement was extensively deteriorated before grinding. In local roads where joints are indispensable, it is preferable to perform joint-sawing instead of tooling as the latter was associated with significant spalling at the tooled joints.

ACKNOWLEDGMENT

The author is indebted to Glenn Engstrom and Jeffrey Brunner of Minnesota DOT. The Minnesota Local Roads Research Board funded the major pervious concrete research in which the author was the principal investigator and Mark Maloney (City of Shoreview Public Works Director) was the technical liaison. Minnesota DOT’s Eddie Johnson provided invaluable internal review.

AUTHOR PARTICIPATION

Dr. Bernard Izevbekhai (author) confirms contribution to the paper as follows: study conception and design: B. Izevbekhai; data collection: B. Izevbekhai; analysis and interpretation of results: B. Izevbekhai; draft manuscript preparation: B. Izevbekhai, who also reviewed the results and approved the final version of the manuscript.

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ROAD SURFACINGS 2

Concrete Pavement Options for Resilient Low-Volume Roads in Pacific Island Countries

SAM W. JOHNSON

World Bank

ASIF FAIZ

Faiz and Associates, LLC

ALEX T. VISSER

University of Pretoria

INTRODUCTION

In pursuit of their economic and social development objectives, Pacific Island countries (PICs) desire to upgrade unpaved low-volume roads (LVRs) for the improvements in connectivity and quality of life associated with all-weather access. While the benefits are clear, the capital cost of conventional pavement technology and the recurrent cost of maintenance make it hard to justify the required investment in upgrading LVRs. Typical low-volume paved roads are surfaced with a bituminous chip seal or a thin asphalt concrete layer on processed aggregate base and subbase courses. Constructing such pavements in PICs is expensive, given the scarcity of aggregate of requisite quality; limited domestic road construction capacity; and scale diseconomies in the use of equipment, plant, and materials. Moreover, vulnerability to natural disasters and climate change necessitates consideration of more resilient paving alternatives. This extended abstract reports on a World Bank study (1) aimed to stimulate the road engineering community in PICs to consider whether concrete pavements are a viable pavement type for LVRs, and if so, how to design, construct, and maintain such pavements. The study suggests that there is substantial potential for concrete pavements to be used for LVRs (fewer than 400 vehicles a day). Learning from pilot applications in Kiribati, Vanuatu, and Tuvalu (its Public Works Department recently built over 400 m of geocell roads with own account labor despite limited prior experience in road construction), it was shown that concrete pavements can be built in PICs with basic equipment and semiskilled staff (1, 2). Four different types of concrete pavement were assessed; the assessments included the strengths, weaknesses, and operations and maintenance implications of each pavement type. Although prepared primarily for PICs, the study provides valuable insights and technical guidance for the application of concrete pavements for LVRs in other regions outside of the Pacific Islands. The World Bank study is expected to be published in 2019.

METHODOLOGY

This study (in the form of a concise technical guide) is a synthesis of international practice in the use of four concrete pavement types for LVRs in both industrialized and developing countries,

notably Australia, the United States, India, Chile, Nicaragua, and South Africa. It provides guidance on design, construction and maintenance requirements, with a specific focus on potential application in PICs. The four concrete pavement types assessed in the synthesis included plain jointed concrete pavement (JCP), concrete block pavement, geocell concrete pavement, and roller compacted concrete pavement.

Following a brief discussion on when is the right time to upgrade an unpaved LVR, the study focus narrows down to considering the suitability of the four concrete pavement types for possible application in PICs, including a technical comparison of the pavement types so that the reader can comprehensively understand the characteristics of the pavements and assess their suitability for the local country context. There is not one concrete pavement type that is optimal for all situations—each pavement type has advantages and limitations that lend them to some contexts, but not others. The technical comparison of the four concrete pavement types includes the following:

- Key strengths,
- Key weaknesses,
- Traffic volume and loads,
- Area types where pavement likely suitable,
- Area types where pavement likely not suitable,
- Expected lifespan,
- Construction costs,
- Equipment required for construction,
- When the road can be opened to traffic, and
- Suitability of labor-based construction.

The body of the technical guide includes the following: (1) key features comprising short summaries and comparison of the pavement types; (2) material requirements, mainly pertaining to characteristics and availability of aggregates; and (3) design, construction, and maintenance considerations, covering the steps shown in Figure 1.

The technical guide includes a list of technical resources for detailed, well-tested guidance for each pavement type: JCP (3–16); concrete block pavers (17–22); geocell concrete (2, 23–26); and roller compacted concrete (RCC) (26–28). Most of the resources are freely available via the hyperlinks or an internet search. The audience for this study includes managers, engineers, and technicians of road entities; road sector consultants and contractors in PICs; and development agencies. The dissemination plan includes a series of workshops and executive seminars at international conferences such as the 12th LVR and in-country training events.

FINDINGS

The critical design consideration for LVRs in PICs is not traffic, but the climatic and terrain conditions. Frequent, intense rainfall and poor drainage pose the biggest challenge to road durability. Following are the main findings reported in the study:

- The life-cycle costs of concrete pavements for LVRs are competitive with or cheaper than flexible surface dressings.

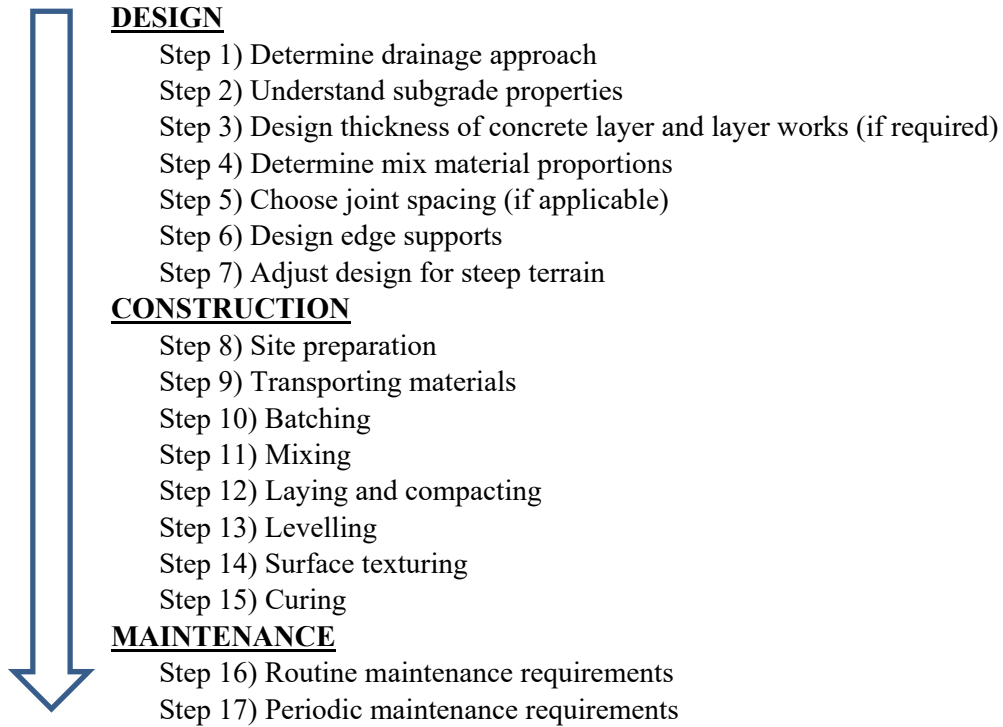


FIGURE 1 Design, construction, and maintenance considerations for concrete pavements.

- Concrete pavements require much less maintenance than flexible surface dressings, a significant advantage given that maintenance is typically not done well in PICs.
- Concrete pavements can be constructed using labor-based technologies and small/community-based contractors.
- Concrete pavements have technical advantages over flexible surface dressings, including resilience to flooding, steep gradients, and surface flow; they are a promising technical option for climate change adaptation.

While concrete pavements are currently uncommon in PICs, they show substantial promise as a suitable pavement type. The tropical environment in most PICs offers pronounced technical advantages for the use of concrete pavements. There is little variation in diurnal and annual temperatures so that the provision for thermal strains in the concrete slabs is not of paramount importance and joints may be more widely spaced. The availability of sandy or coralline aggregate subgrades, especially in the atoll islands, offers a stable (and often a strong foundation) for the concrete slabs and blocks. The Cement and Concrete Institute of South Africa offers the following advantages and benefits of JCP for LVRs, but the benefits are the same for the other concrete pavements evaluated in this study (13):

- Durable;
- Lower whole-life cost for comparable flexible surfacing pavement design;
- Very low maintenance costs;

- Skills acquired are not limited to road construction, but are transferable to the wider building construction industry;
- Ideal for upgrading existing deteriorated unpaved roads by overlaying; and
- Can reduce separate drainage infrastructure needed if drainage integrated into road.

JCP is by far the most common type of portland cement concrete pavement. JCP for LVRs is cast in slabs, with a typical longitudinal length of 4 to 5 m between joints and a width of 3 to 4 m. The slabs are sized to dimensions such that uncontrolled cracking from thermal, traffic, and moisture stresses should not occur. In Chile, a JCP variant is used with the slab size (1.4 to 2.5 m square) configured to one-half the typical dimensions such that any pavement slab is loaded by only one wheel or a set of wheels at the same time, thereby minimizing the critical top tensile stress. With the reduced tensile stress, slab thickness can be significantly decreased (as low as 8 cm) for LVRs. With fiber-reinforced concrete the performance of these short slabs is further enhanced (29, 30). Concrete block paving is a system of individual blocks arranged to form a continuous hard-wearing surface pavement. The block can be made in different shapes and sizes, with grooved interlocking pavers being the most durable shape for vehicle traffic. Geocell concrete pavement is constructed by filling a thin-walled (0.2 mm) dimpled honeycombed plastic geocell formwork with concrete. The geocell is a sacrificial formwork and results in cast in-situ concrete blocks that have positive interlock. RCC is a premix produced at a central batching plant that is laid using a purpose built mixing and paving machine and compacted by heavy vibratory steel drum and rubber-tired rollers. No dowels, reinforcing, or critically, formwork, are required. The key strengths and weaknesses of the four concrete pavement types are summarized in Table 1.

TABLE 1 Key Strengths and Weaknesses of Concrete Pavement Types for LVRs

JCP	Block Paving	Geocell	RCC
Key Strengths: Well suited to unskilled labor; knowledge of plain concrete or block paving construction useful skill outside of road construction; low maintenance needs.			
<ul style="list-style-type: none"> • Well-understood, well-tested method • Does not require expensive equipment 	<ul style="list-style-type: none"> • Very simple to construct and repair relative to other concrete pavements • Can generate social and economic benefits for communities through local block manufacture • Does not require expensive equipment 	<ul style="list-style-type: none"> • Waterproof • Can cast the drainage and road in one go • No joints, virtually no maintenance • Does not require expensive equipment 	<ul style="list-style-type: none"> • Can be driven on immediately after construction • Can be a more rapid method than other pavements if sophisticated equipment available
Key Weaknesses: Requires imported cement.			
<ul style="list-style-type: none"> • Joints can let water in if not maintained, pumping of fines • Shoulders must be well maintained against erosion • If slab cracks badly may require mechanical equipment to rehabilitate 	<ul style="list-style-type: none"> • Requires block manufacturing industry • Joints can let water in if not maintained, pumping of fines • Riding surface may not be as even as JCP • When blocks come loose, they need to be fixed promptly, else further blocks will come loose 	<ul style="list-style-type: none"> • Plastic geocells need to be shipped from single manufacturer in South Africa • Riding surface may not be as even as JCP • Requires good site quality control 	<ul style="list-style-type: none"> • Requires heavy roller and paver/motor grader • Must be mixed at a central facility and then transported • Finish is rough

CONCLUSIONS

The concrete pavement options explored in this study offer good prospects of application not only in PICs, but more broadly in tropical countries vulnerable to natural disasters triggered by extreme weather events and the longer-term impact of climate change. In considering the design–build maintenance cycle of the four concrete pavement types included in this study, the first step is an assessment of the approach to drainage. All other considerations follow from this premise. This study makes a cogent case for the use of concrete pavements for LVRs from the following various perspectives:

- From a rural livelihood’s perspective, concrete pavements can be constructed using labor-based technologies and small/community-based contractors; three of the four concrete pavement technologies assessed in this study are labor intensive and have the potential to create substantial rural off- farm employment, both skilled and unskilled.
- From an operations perspective, concrete pavements provide a more robust alternative for easier and more effective road asset maintenance and traffic overloading control.
- From a climate resilience perspective, concrete pavements are more resistant to erosion, surface flow, and flooding, and under high ambient temperatures, they do not soften, rut, or become brittle (as bituminous pavements do); they are a promising technical option for climate change adaptation. Lighter in color and texture, they are intrinsically cooler pavements: they reflect incoming solar radiation and lessen the impact of the urban heat island effect.
- From a cost-effectiveness perspective, after accounting for their longer service life (30–40 years compared to 15–20 years for bituminous pavements), reduced maintenance costs, and climate and safety benefits, concrete pavements are more cost effective than bituminous paving options for LVRs. With discount rates in the 5%–9% range and applying appropriate conversion factors for factor inputs, taxes, and subsidies, concrete pavements are estimated to be more economical than bituminous alternatives on a life-cycle cost basis.

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Recycled Building Materials for Asphalt Pavement in Low-Volume Agriculture Roads

HESHAM MAHGOUB

Engineering and Research International

AHMED MOHAMADY

MOHAMMED E. ABDEL-MOTALEB

Zagazig University, Egypt

The deposition of demolished concrete from old building structures in landfills is currently an unacceptable strategy because of the increasing cost of disposal, the declining availability of disposal space, and other environmental concerns. Recycling of old building structures as construction materials is a viable alternative.

The objectives of this research study are to investigate the feasibility of using recycled building materials in asphalt pavement mixes in the low-volume agriculture roads. Two types of recycled building materials, recycled bricks (RB) and recycled concrete (RC), were used as coarse aggregates in hot-mix asphalt (HMA). A total of nine hot asphalt mixtures were formed by replacing a certain percentage of the coarse virgin aggregates by crushed bricks and concrete. The prepared mixtures were tested in Marshall, immersion, indirect tensile, and creep tests to investigate the effect of recycled materials on mixture properties and identify the optimum percentage of recycled materials.

The findings in this project support the hypothesis that both RC and RB can be used effectively as a coarse aggregate in HMA in the low-volume agriculture roads when appropriate quality-control techniques are used. Based on the information obtained in this study, the recycled building materials could be used as partial replacement of coarse virgin aggregates in HMA without significant reduction in its performance.

INTRODUCTION AND BACKGROUND

The technology of recycling material started at the end of the 1970s of the last century as a step toward keeping the environment clean and to reserve the natural aggregates. The experience of recycling materials was limited to reusing building rubbles in cement concrete (1, 2) and reusing the removed pavement materials in highway construction (3, 4). Also, pavements became favorable structures for the recycling of a wide range of waste materials because of the large quantities of materials required for their construction (5, 6).

The amount of building demolition debris rapidly increases every year as a result of building demolition and reconstruction processes. So, the recycling of demolished building rubble as coarse or fine aggregate in new HMA represents an important strategy at a time when the cost of dumping such materials is on the increase. In addition, the available resources of

conventional aggregates for pavement construction in most countries are on the decrease, added to an increasing cost of the new construction materials.

So, considerable research was directed in Egypt to investigate the availability of reusing the demolition of building debris in highway construction, from which a limited number of studies focused on the reuse of building recycling materials in HMA.

Abd Allah and El-Saied (7) investigated Marshal Mix Design properties of mixes produced by full and partial replacement of the natural aggregates of crushed limestone and natural sand used in Egypt by recycled crushed concrete and bricks as a trial to increase the used percent of the building waste materials in HMA. The study concluded that the use of recycled aggregates may be accepted up to 60% of the natural aggregate mass. Khalaf (8) investigated the properties of two HMAs; one was produced using crushed brick as a coarse aggregate, and the other was produced using crushed brick as a fine aggregate. The two mixes were compared with the traditional mixes that were produced using the crushed granite and siliceous sand, respectively. The author concluded that the high tendency of brick to absorb bitumen improves the mix strength through better particle interlocking. In addition, the study indicated that the light weight of such mixes reduces its density and increase its stability.

El-Saeid (9) investigated the influence of using recycled sands on the properties of the asphalt concrete mixtures. The author concluded that using the recycled sand decreases the voids in the mineral aggregates (VMA), which may not offer enough space for the effective bitumen, and the required air voids. Therefore, this study restricted the percent of recycled sand by 21% as a maximum value to be used in HMA. Abeyasinghe and Khalaf (10) evaluated the performance of HMA produced with RC coarse and fine aggregates as a replacement to the granite and natural sand, respectively. The results showed that the absorbed bitumen is increased because of the high porosity of the RC, along with enhancing the mix stiffness.

The performance of HMA, which contains recycled building materials as aggregates, is not yet sufficiently investigated. Research is needed to evaluate RC and RB materials and establish application procedures for pavement design. The results obtained from this research study can establish guidelines that will enable RC products to be used with confidence.

EXPERIMENTAL DESIGN

The objective of this study was to perform a laboratory study to determine the applicability of using RB and RC in HMA applications, including typical paving operations and environmental conditions commonly found in the Egyptian low-volume roads. Figure 1 shows the experimental design for the laboratory evaluation. The following sections describe the individual tests that are included in the experimental design.

As shown in Figure 1, nine hot asphalt mixtures were subdivided into two main groups to obtain the suitable applications for each aggregate of the mixtures. Each group was made by replacing a part of the virgin coarse aggregates by different percentages (0%, 25%, 50%, 75%, and 100%) of crushed RB and RC, respectively. The investigated gradation of the used aggregates was obtained by blending 60% coarse aggregate, 33% siliceous sand with 7% stone filler. All mixes gradation lies within the limits of the standard aggregate gradation Number 4-C according to the *Egyptian Standard Specifications (11)*, as shown in Table 1.

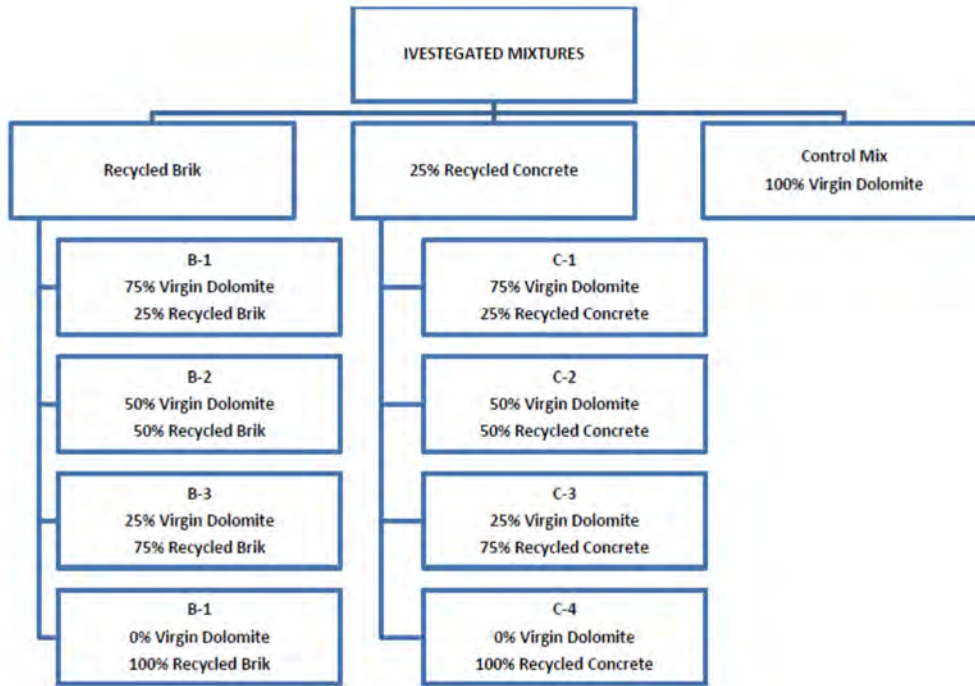


FIGURE 1 Structure and flow of the experimental design.

TABLE 1 Design Gradation (4-C) and Specification Limits (11)

Sieve Size	Designed Gradation	Specification Limits
1 in.	100	100
3/4 in.	88.3	80–100
1/2 in.	78.6	—
3/8 in.	69.5	60–80
No. 4	49.2	48–65
No. 8	36.8	35–50
No. 16	33	—
No. 30	27.3	19–30
No. 50	17.1	13–23
No. 100	12.5	7–15
No. 200	5.7	3–8

Additionally, the experimental design was concerned with selecting the appropriate tests most related to evaluating the paving mix performance. The Marshall test was performed to obtain the optimum asphalt content (OAC) and mix characteristics. The Marshall Immersion test was performed to assess the index of retained strength (IRS) as an indicator for moisture damage resistance. The indirect tensile test was selected to assess the tensile strength for cracking resistance, while the creep test was selected to evaluate the rutting tendency of the investigated mixtures.

MATERIALS AND SAMPLING

In order to adequately examine the use of recycled materials in asphalt mixes and its effects on the properties of such pavements, the study team selected materials more readily available in eastern Egypt. RB and RC obtained from the recycling plant in the City of Zagazig, Egypt. The old concrete made of siliceous gravel aggregates and sand. The virgin coarse aggregate is crushed dolomite obtained from the Ataka quarry. The fine aggregate is siliceous sand and stone filler. The asphalt binder used has a penetration of 60/70 and 1.022 specific gravity. The physical properties of asphalt binder are shown in Table 2.

Durability and acceptance laboratory tests were performed on recycled materials and coarse and fine aggregates. The laboratory procedures included sieve analysis, apparent specific gravity, bulk specific gravity, absorption, LA abrasion, and stripping. All laboratory test methods were based on the ASTM testing specification as shown in Table 3.

LABORATORY TESTING AND RESULTS

The HMA laboratory tests conducted in this project included the Marshall Mix Design procedure, the indirect tensile test, static creep, and the Marshall Immersion test. All laboratory test methods were based on the ASTM and Egyptian Standards Testing Specifications.

TABLE 2 Properties of Asphalt Binder

Test No.	Test	AASHTO Designation	Results	Specification Limits
1	Penetration (at 25°C), 0.1 mm	T-49	64	60–70
2	Softening point, °C	T-53	53	45–55
3	Flash point, °C	T-48	+270	≥ 250
4	Kinematic viscosity, (at 135°C),	T-72	355	≥ 320

TABLE 3 Properties of the Coarse Aggregate Materials

Test No.	Test	AASHTO Designation	Results			Specification Limits
			Crushed Dolomite	Crushed Concrete	Crushed Brick	
1	Specific gravity (SG)	T-85	—	—	—	—
1	Bulk SG	—	2.512	2.196	1.947	—
1	Saturated surface-dry SG	—	2.539	2.324	2.036	—
1	Apparent SG	—	2.659	2.519	2.201	—
2	Water absorption (%)	T-85	2.6	5.76	12.12	≤ 5
3	Disintegration (%)	T-112	0.63	0.58	1.4	≤ 1
4	Los Angeles abrasion	T-96	—	—	—	—
4	After 100 rev. (%)	—	5.6	9.3	14.8	≤ 10
4	After 500 rev. (%)	—	28	26	45	≤ 40
5	Stripping (%)	T-182	> 95	—	—	≥ 95

Marshall Test

Marshall Mix Design procedure was performed to obtain OAC and the corresponding mix properties for the investigated mixtures. The selected test criteria were the 75-blow Marshall Compaction procedure according to ASTM D1559 and AASHTO T-245. In addition, Marshall stiffness (S_M) was calculated for each HMA as follows:

$$S_M, \text{ psi} = \text{Stability}/(\text{Flow} \times \text{Specimen Height}) \quad (1)$$

Indirect Tensile Test

Standard test samples of 2.5-in. height by 4-in. diameter were used for the indirect tensile test. The tests were performed at room temperature of about 30°C by loading test specimens with compressive vertical load that act parallel to and along the vertical diameter plan until failure, at a constant loading rate of 1 mm/min. A steel loading strip 0.5 in. wide with a curved loading surface was used to distribute the load uniformly and to maintain a constant loading area all over the loading period. The simplified relation for calculating the tensile strength (σ_t) for the 4-in.-diameter specimen with 0.5-in.-wide curved loading strip are as follows (11):

$$\sigma_t, \text{ psi} = 0.156 (P_f/H) \quad (2)$$

where

P_f = the total load at failure (lb) and
 H = height of the specimen (inches).

Creep Test

The creep test was performed by applying a constant vertical stress ($\sigma = 0.1$ Mpa) in the axial direction of the tested sample in a very short time and then keeping it constant until the test was completed, using the consolidation-testing machine at a constant temperature of 40°C, applying the same technique used by Howeedy et al. (13). A dial gauge of accuracy 0.01 mm was used to measure the deformations. The vertical deformations and the corresponding loading times were recorded during the test period (1 h). The cumulative creep strain ($\epsilon_c = \Delta H/H$; in which ΔH is the measured vertical deformation and H is the initial height of the tested specimen) was then determined at the end of the test.

Marshall Immersion Test

The Marshall Immersion test, which is a simplified version of the AASHTO-T165, was used to evaluate the resistance of HMA to moisture damage. This test is intended to assess the loss of stability resulting from the action of water on the compacted mixtures by comparing the stability of dry specimens to those immersed in water bath at 60°C. Specimens were made at their OAC and immersed in the water bath for 24 h at 60°C, and some other specimens were immersed in the water bath for only 30 min at 60°C also. IRS was then calculated using the following equation:

$$IRS = S_1 / S_o \times 100 \quad (3)$$

where S_1 and S_o are the stability for specimens immersed in water bath at 60°C for 24 h and 30 min, respectively.

The results of all laboratory tests are presented in Table 4.

ANALYSIS AND RESULTS

Consequently, in comparing performance across mixes, it was found useful to develop trend lines by leveraging collectively all the data assembled for each of the mechanical properties investigated. The process adopted for this purpose was as follows:

Marshall Test Results

Results of the Marshall tests of the investigated hot asphalt mixtures with different percentages of RB and RC as coarse aggregate are presented in Figure 1. The figure indicates that increasing RB generally increases OAC, decreases unit weight and flow, with no significant effect on air voids (AV) and VMA.

On the other hand, the stability of the mix increases up to a mixing ratio of RB = 50%, then it decreases. The stability value of the mix with RB = 50% is about 115% of the traditional mix with an increase in OAC by 13%. However, the reduction in stability with RB = 100% is 22% with increasing OAC by 42%, and the stability is still in the acceptable limit (higher than 1,800 lb). In addition, it is noticed from the figure that increasing RC increases the flow, AV, and VMA, with no effect on OAC, and slightly decreases the mix unit weight, while the stability decreases with an increasing rate. The reduction in the mix stability value with RC = 50%, and 100% is about 10% and 34%, respectively, while the reduction for RC = 75% is 21%, and the stability is still in the acceptable range.

Examining Marshall test results along with the specifications limits of hot asphalt mixes, it can be stated that the chosen recycled materials can be used with replacement percentages up to 100% and 75% for RB and RC, respectively, without critically affecting the strength and mechanical properties of the produced mix.

TABLE 4 Measured Characteristics of the Investigated Mixtures

Mix No.	OAC (%)	AV (%)	VMA (%)	Unit weight (gm/cm ³)	Stability (lb)	Flow (0.01 in.)	S_M (psi)	Tensile Strength σ_t (psi)	Creep Strain (ϵ_c) (%)	IRS (%)
CM	5.40	4.1	15.4	2.352	2418	12.50	7738	80.5	9.10	89.5
B1	5.46	4.24	15.86	2.345	2355	12.75	7388	73.1	9.91	81.7
B2	5.45	4.51	16.14	2.330	2176	13.5	6391	62.5	11.10	71.2
B3	5.55	4.95	16.62	2.321	1909	14.50	5266	57.3	12.40	64.6
B4	5.50	5.41	17.10	2.301	1602	15.50	4134	47.8	14.20	62.0
C1	5.80	4.04	15.30	2.311	2724	12.00	9080	79.9	8.04	83.4
C2	6.1	4.01	15.22	2.280	2778	11.75	9457	81.3	7.28	77.8
C3	6.7	3.98	15.25	2.229	2427	11.25	8629	79.1	7.75	69.5
C4	7.65	3.9	15.32	2.192	1891	10.75	7036	78.2	9.64	65.1

The values of Marshall stiffness (S_M) of the same mixes are presented in Figure 2 and Figure 3. The figure indicates that S_M increases up to RB = 50% then it decreases for any further increase in the recycled brick content. The S_M value of the mix with RB = 50% is 122% of the traditional mix, while the reduction in S_M with RB = 100% is only 9%. However, increasing the RC percent decreases S_M with an increasing rate. The reduction in S_M value of the mix with RC = 25% and 50% is 5% and 17%, respectively; while that for RC = 100% is 47%. Based on resulting S_M , which is considered as an indicator for deformation resistance of the mixes, it can be suggested that recycled material can be used with replacement percent up to 100% for RB, and should not be higher than 50% for RC.

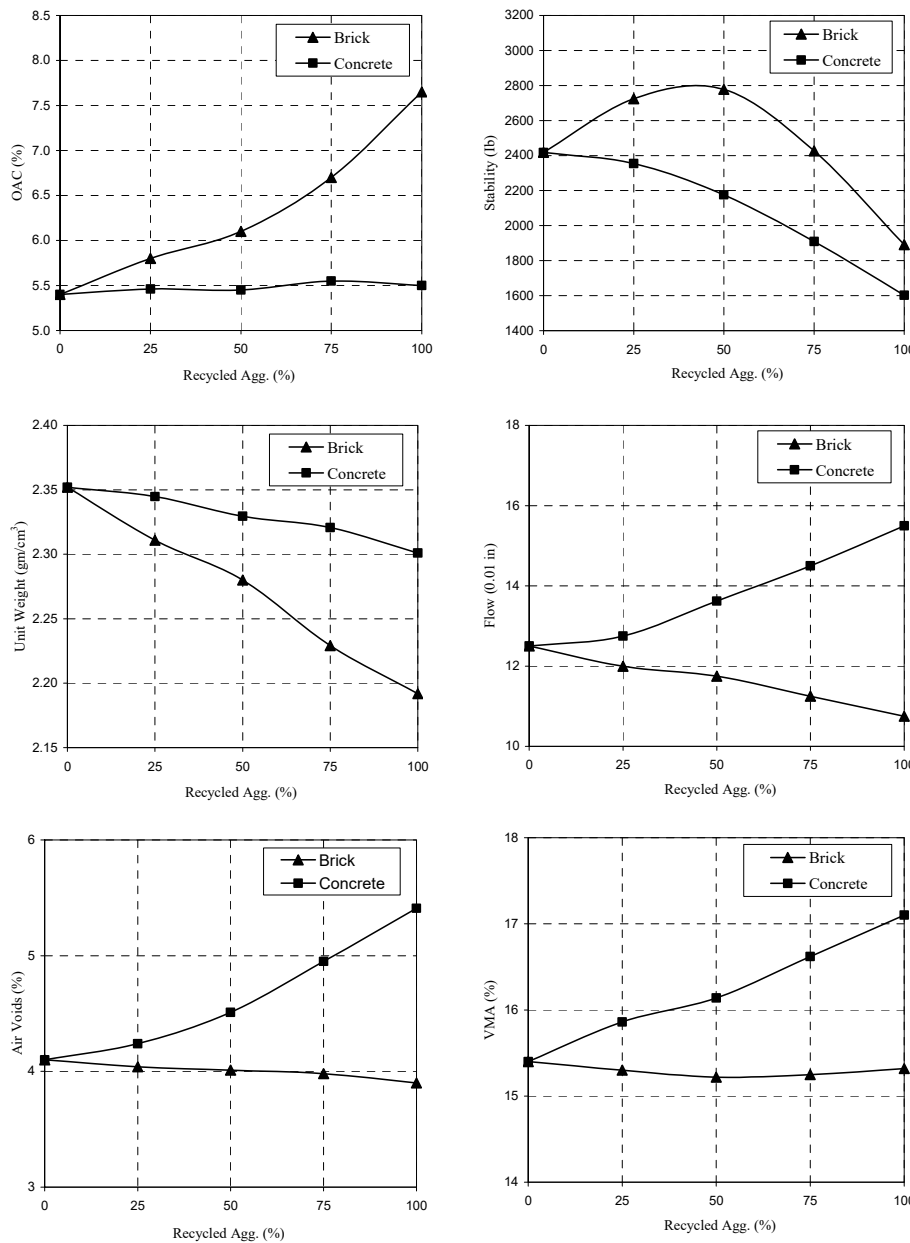


FIGURE 2 Marshall properties versus percent of recycled aggregates.

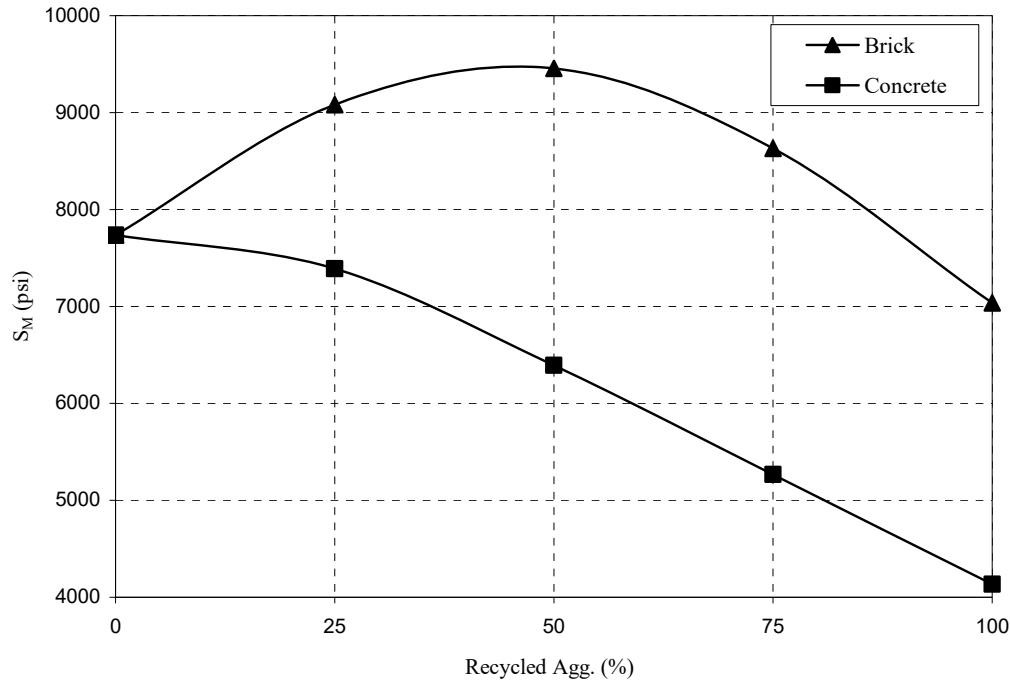


FIGURE 3 Marshall stiffness (S_M) versus percent of recycled aggregates.

Indirect Tensile Test Results

Figure 4 presents the results of indirect tensile test in terms of tensile strength (σ_t), as an indicator for cracking resistance of the investigated mixtures with different percent of RB and RC. It can be noticed from the figure that increasing RB is not significantly affecting σ_t , while increasing RC content decreases σ_t with an increasing rate. The reduction in σ_t value of the mix with RC = 25%, 50%, 75%, and 100% is 9%, 22%, 29%, and 41%, respectively. It may be concluded from the results that RB can be used with percent up to 100% with no reduction in cracking resistance. However, for HMA mixes with RC, it is preferred that replacement percent shouldn't exceed 25% to avoid significant reduction in the cracking resistance.

Creep Test Results

Figure 5 represents the results of creep test in terms of creep strain (ϵ_c) as an indicator for the potential tendency for rutting of the investigated hot asphalt mixes with different percent of RB and RC. The figure indicates that ϵ_c decreases when the RB is increased up to 50%, then it began to increase for any further increase in RB percent. The ϵ_c value of the mix with RB = 50% is 80% of the traditional virgin aggregate mix, while the increase in ϵ_c when RB percent reaches 100% is only 9%. Figure 5 clearly shows that ϵ_c increases with increasing in RC percent.

The increase in ϵ_c value of the mix with RC = 25% and 50% is 9% and 22%, respectively, while that for RC = 100% is as high as 56%. Based on ϵ_c , which is considered as indicator for rutting susceptibility of the hot asphalt mixes, it can be concluded that recycled materials can be used with replacement percent as high as 100% for RB, with an optimum replacement ratio of 50%, while the mixing ratio for RC should not exceed 25%.

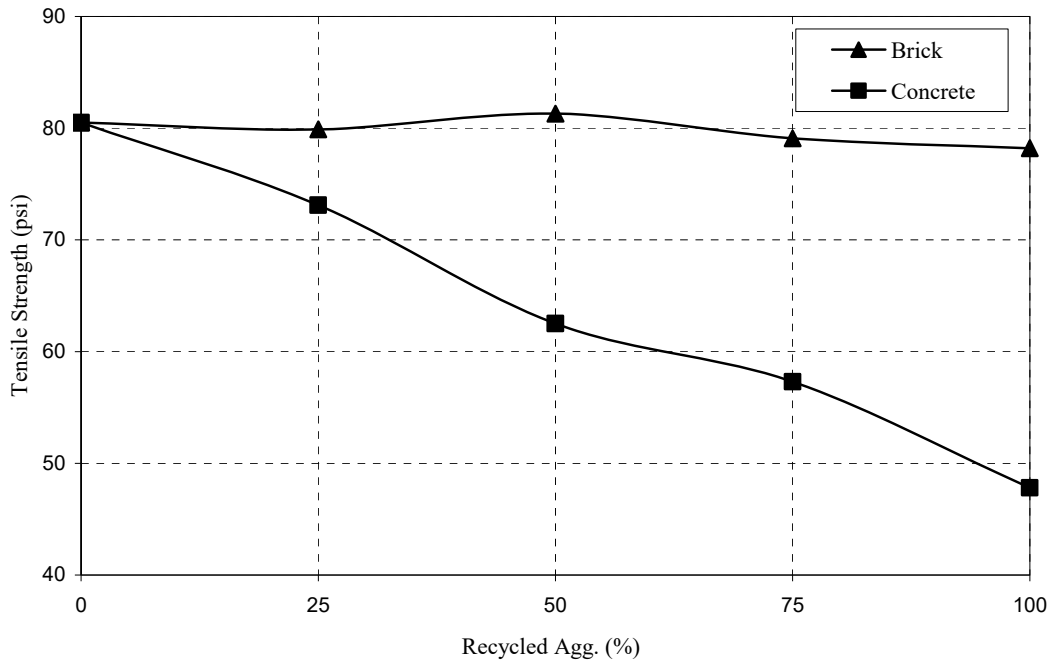


FIGURE 4 Tensile strength versus percent of recycled aggregates.

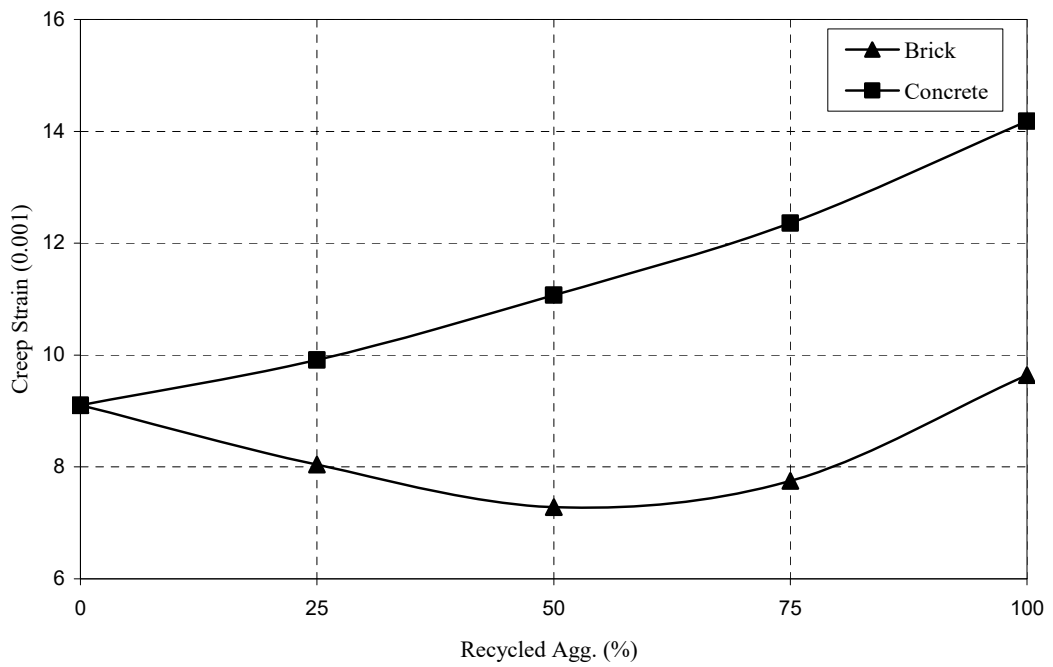


FIGURE 5 Creep strain versus percent of recycled aggregates.

Marshall Immersion Test Results

The results of the Marshall Immersion test in terms of IRS of the investigated mixtures with different percent of RB and RC are presented in Figure 6. The figure shows that the values of IRS generally decrease with increasing either RB or RC content. The IRS values at 50% replacement are 77.8% and 71.2%, while that at 100% replacement are 65.1% and 62.0% for RB and RC, respectively. On the other hand, at a replacement ratio of 25% both the RB and RC achieve IRS values of about 83% and 82%, respectively.

Based on the requirement of hot asphalt mixes for moisture damage resistance (IRS is greater than 75%), it can be concluded that recycled material can't be used with replacement percent more than 50% and 25% for RB and RC, respectively.

Considering all the investigated mix properties together (Marshall properties, cracking resistance, rutting resistance, and moisture resistance) regarding RB and RC, it is noticed that RB specimens gave better performance and higher strength properties than RC specimens. These results may be attributed to the surface roughness of the crushed bricks giving higher particle internal friction and surface interlock.

In addition, the crushed bricks have a higher ability to absorb more asphalt material, allowing for more homogeneous mix with other mix constituents. On the other hand, crushed concrete particles with siliceous gravel and sand are having a smooth surface texture as well as its imperviousness, which produce mix with low angle of internal friction and consequently lower particle interlock. Therefore, the optimum replacement percentages of RB and RC that achieve all the strength and stability requirements of the hot asphalt mixes are 50% and 25%, respectively.

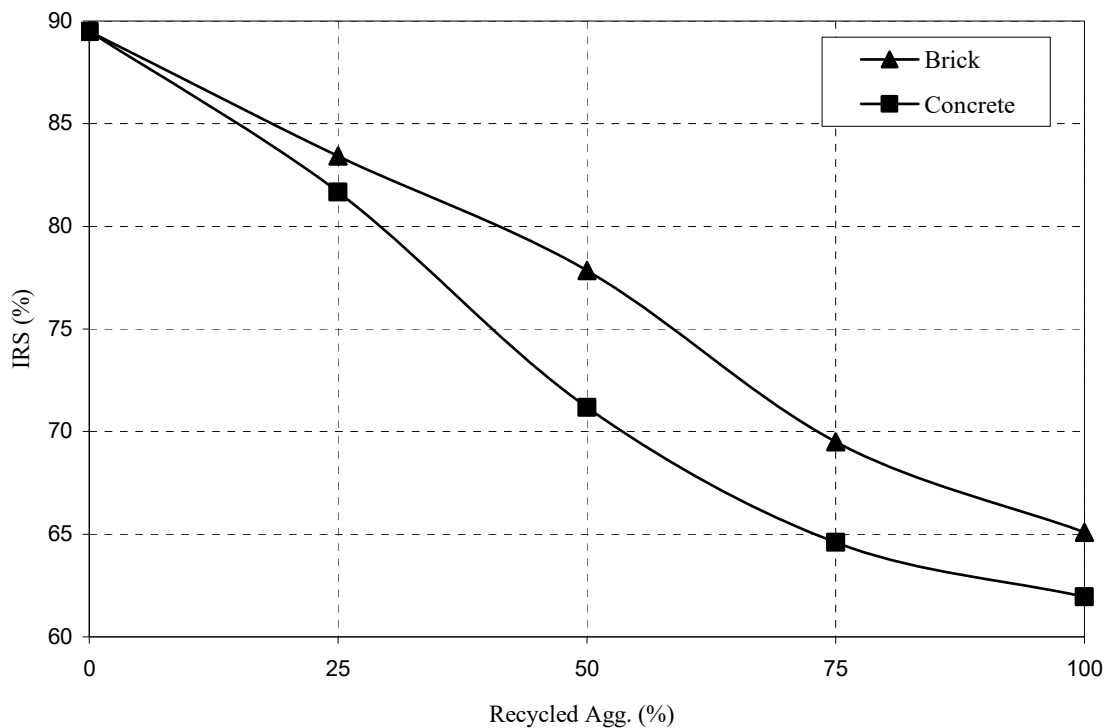


FIGURE 6 IRS versus percent of recycled aggregates.

SUMMARY AND CONCLUSIONS

In order to successfully use the RC materials in flexible pavement construction, series of laboratory tests had been conducted. The tests included the Marshall Mix Design procedure, the indirect tensile test, the static creep test, and the Marshall Immersion test. The paper provided analysis and discussion of results obtained for the replacement of parts of the coarse aggregates in the hot asphalt mixes with certain percent of RB and RC.

In conclusion, the results of this study show that RB can be used in HMA and replace up to 50% of virgin aggregate without scarifying the properties and the quality of the mixes. The RC can replace up to 25% with the same properties.

Test-track pavement sections are recommended to verify the actual performance of HMA made with recycled building materials as well as an economical study to determine the feasibility of using such materials asphalt mixtures.

It is essential for the road agencies to develop practical and reliable guidelines and specifications for the use of recycled building materials in HMA especially in the low-volume agriculture roads. Research is needed to evaluate recycled materials and establish application procedures for pavement design.

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Rural Truck Paths on Low-Volume Roads in Austria

WOLFGANG HASLEHNER

State Government of Burgenland

The rural road network in Austria consists of all roads that are neither federal nor provincial roads and serve the purpose of enabling access to the rural area. This low-volume road network includes all municipal roads, farm roads, and forest roads. The total length of these roads amounts to approximately 160,000 km or 80% of the total Austrian road network. The responsibility for construction and maintenance of this rural road network in Austria is split between private persons and public authorities. Within these special circumstances a new technical design guideline for rural track paths has been elaborated in Austria. During this elaboration the experiences and know-how from Germany and Switzerland have been analyzed and taken into consideration. The main part of the paper deals with this new design guideline and shows an innovative way to handle activities in construction and maintenance of low-volume roads realized as single-lane rural track paths. These track paths consist of two load-carrying tracks constructed of asphalt, concrete, surface treatment, or block pavers. The obvious advantages of this tracked paving approach are to reduce the impact of impervious surface types and the impact on the environment. What makes this guideline unique is the fact that it is the first of its kind in Austria to encompass all aspects of planning, design, practical construction, and implementation of rural track paths on low-volume roads.

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Planning and Economics

Topological Connectivity Criteria for Low-Volume Road Network Design and Improvement

WUBESHET WOLDEMARIAM

Purdue University Northwest

SAMUEL LABI

Lyles School of Civil Engineering,

ASIF FAIZ

Faiz and Associates, LLC

Transport connectivity can be operationalized in investment appraisal when it is used as one of the criteria in multicriteria investment analysis and network link prioritization. The traditional approach to evaluating the network impacts of transport projects (which involves link capacity and congestion delay considerations under a traffic assignment user equilibrium framework), is appropriate for congested urban networks. On the other hand, for sparse networks (such as rural and low-volume roads), the lack of topological connectivity is a more pressing challenge compared to congestion. The paper argues that for certain kinds of networks such as sparse, rural, multivehicle type, and disconnected networks, assessing the network performance based on topological connectivity is a more appropriate compared to link travel times. This paper then identifies five contexts where agencies routinely make investment decisions that require consideration of network topological connectivity. The paper acknowledges the importance of network topological connectivity as an important project evaluation criterion in any of these contexts, and discusses several existing metrics for network topological connectivity. The paper proposes a method for developing a comprehensive index of network topological connectivity. The method incorporates the preferences of the network users regarding specific network features (nodes and links) and individual measures of topological performance. The paper then uses a case study to demonstrate how agencies can apply the framework to assess the overall connectivity of their networks at the current time or in response to prospective link-development projects, in any of the five contexts mentioned above.

INTRODUCTION

Transport decision-makers often consider a wide array of evaluation criteria when they make investment decisions for proposed projects or when they carry out ex poste evaluation of past projects: travel time, vehicle operating cost, safety, and economic efficiency (1–9). Other traditional criteria are related to the impacts on land use, the social and biological environments, economic development, asset resilience (6, 10), aesthetics, air quality, water resources, and noise (11–18). In addition, a number of transport literature have presented or implemented

methodologies that analyze the impacts of investment alternatives based on some combination of these evaluation criteria (19–25).

The above evaluation methodologies for investment decisions do not consider directly the impacts of projects on the topological performance of a network. On one hand, it may be argued that there is no need for considering topological connectivity as an independent evaluation criterion, because transport mobility (which is directly related to network connectivity) is already considered in most evaluations using mobility-related metrics such as travel time. In other words, where travel time is a key criterion in evaluation, the issue of topological connectivity is moot.

The opposing school of thought contends that mobility is not always an adequate surrogate for topological connectivity, and that having both travel time and connectivity does not always constitute double counting; for that reason, transport evaluation may consider topological connectivity in parallel with mobility (travel time). For example, certain corridors and networks are such that the travel time metric does not adequately reflect the user benefits. This is the case at sparse networks, networks with disconnected nodes (seasonally or perennially), low-volume roads or networks, and networks whose links have traffic volumes far below capacity), and networks with multiple modes of transport each with different travel times (such as automobile, bicycle, and walking). In such cases, vehicular travel time is not enough, and topological connectivity is more important.

There exist several decision contexts with respect to network connectivity, in other words, situations where a low-volume road agency makes a decision with due cognizance of the investment impacts that include the changes in network connectivity. These include

1. Designing a new network, where no network currently exists, to connect existing or anticipated nodes most efficiently;
2. Assessing the impact of a proposed or newly added link on the connectivity of the overall network, and prioritize multiple proposed links based on such impacts;
3. Deciding whether and where to add a proposed link to an existing network;
4. Prioritizing existing links by measuring and ranking their topological connectivity contributions to the network; and
5. Carrying out multicriteria evaluation of a highway project where one of the evaluation criteria is network connectivity.

To address any given appraisal problem in any of these contexts, a need exists to develop a measure of network connectivity that is consistent with the mission or goals of the transport agency. The case study we present in this paper addresses some of these decision contexts. The main objective of the paper is to present alternative ways by which the measures of network connectivity could be used as a criterion for transport investment decision making. In reaching this goal, the paper carries out a literature review of network connectivity in the context of investment evaluation and decision-making. The paper then presents existing traditional indicators of network connectivity that transport planners and decision makers could use in their appraisal of projects, policies, and programs. To overcome the limitations of existing indicators, the paper proposes a method for developing a comprehensive index of network topological connectivity. The method incorporates the preferences of the network users regarding specific network features (nodes and links) and individual measures of topological performance. The paper then uses a case study to demonstrate how agencies can apply the framework to assess the

overall connectivity of their networks at the current time or in response to prospective link-development projects, in any of the five contexts mentioned above.

REVIEW OF THE LITERATURE

Motivation for Considering Transport Connectivity in Project Evaluation

In transportation literature, there are several instances where researchers and practitioners have highlighted the importance of considering connectivity in the analysis of the impacts of transportation systems of various modes. The 2015 Paris Process for Mobility and Climate asserted that investments that increase connectivity yield benefits including broader-based economic growth, and easier access to economic opportunities for the economically disadvantaged segments of the population (26). Picot et al. presented the concept, roles, benefits, and challenges of transport connectivity in Asian countries, and discussed how increased connectivity can facilitate inclusive and sustainable economic growth (27).

In Nepal, recognizing that efficient transport and improved connectivity to rural areas is key to poverty reduction, the government established the Transport Connectivity Sector Project with the overall goal of improving feeder road connectivity from Strategic Road Network (SRN) to rural areas (28). In Eastern Europe, Schade et al. analyzed transport connectivity in the triangle region between Poland, the Czech Republic, and Slovakia, and identified rail and road links that could be provided to enhance overall connectivity in that region (29). In Sri Lanka, the World Bank's ultimate objective for the Transport Connectivity and Asset Management Project included the improvement of road service delivery on the Ja-Ela to Chilaw section of the National Highway A003 (30).

The U.S. DOT maintains that transport connectivity, within and among the different transport modes, facilitate movements by shippers and travelers and enhances the livability of communities by offering multiple transportation options to residents (31). Sinha and Labi recommended that transport project decisions should include, among other criteria, the mobility of system users and the connectivity of the transport network but did not provide a detailed framework for measuring network topological performance (20). In the next section of this section, we examine some indicators that researchers have established in the literature to measure transport connectivity.

Criteria for Connectivity Measurement

The Montana DOT, recognizing that increased connectivity reduces the amount of circuitous travel required and often encourages shorter vehicle trips and the use of alternative modes, measures connectivity as a "connectivity ratio" (the number of links divided by the number of nodes) for sketch planning purposes (32). Bell considered the cost of traversing a link in the network as a measure of network performance (33). In a study in United Kingdom, Woollett et al. quantified transport connectivity by combining the economic importance of places and connections with transport network (34). Sandra and Laurie (2004) described general project selection criteria that could be implemented by different districts to prioritize rural transport infrastructure projects in the state of Montana. Lane closure was one of the criteria; this is assumed as a surrogate but not a criterion for directly measuring network connectivity: greater

numbers of lane-closures during construction was used to measure the reduction in connectivity during those periods (32). To help improve highway network connectivity in Sub-Saharan Africa and other developing countries, the World Bank uses a rural accessibility index (RAI) that measures the proportion of rural communities that live within 2 km (which translates into 20 to 25 min of walking) from an all-season road (35). The index could be expanded to cover other investment contexts, urban and rural, and for different modes.

In most literature, the travel time on the network links has been used as an indicator of the extent to which the network is connected. Cambridge Systematics (25) and Sinha and Labi (20) discussed the average O-D travel time and average trip length as connectivity performance measures (PMs) for passenger and freight travel. Travel time was mentioned as a measure of level of satisfaction in OECD (20, 22). Forkenbrock and Weisbrod provided detailed steps for network-level and local level connectivity measurement (6). To identify critical road segments and measuring systemwide robustness in transportation networks with isolating links, Sullivan et al. developed an index called network robustness index and used network-wide travel time as a performance measure (36). Derrible used the concept of network centrality to determine key transfer stations in public transportation systems (37), and Derrible and Kennedy analyzed the connectivity of subway systems at various locations worldwide using enhanced graph theory (38). Erath et al. applied graph theory to analyze the Swiss road and railway networks at different points in time (39).

A number of researchers have developed connectivity measures that consisted of both travel time as well as other criteria. Alstadt defined “connectivity” in the context of transportation planning as the ease, time or cost of traveling between different transportation route systems or modal systems (40). Scott et al. evaluated the impact of a highway section to the change in network level travel-time using the network robustness index (41). For non-sparse networks such as those typically associated with urban streets, travel time is considered a more important (even if indirect) measure of network topological performance. On the other hand, for sparse networks (such as rural roads in many developing countries as an example), the lack of topological connectivity is a more pressing challenge compared to congestion. There exist in older, non-transport literature, network connectivity evaluation criteria that are purely based on the network topology. These were developed and applied in the context of graph theory, communications, and other applications other than transport.

Topological Performance Measures for a Low-Volume Road Network

A number of measures are used in order to characterize the extent to which the network, i.e., nodes and links, are connected. To illustrate the computations for each measure, node-to-node cost (distance) data from a simple network shown in Figure 1 is used. The calculation results are given in Table 1, and the definitions of the indices and terms are provided in Table 2.

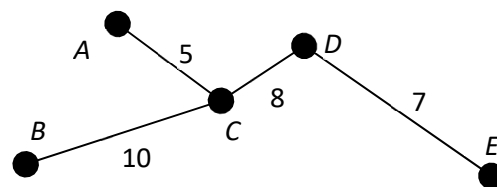


FIGURE 1 Example network of low-volume roads, for illustrating connectivity calculations.

TABLE 1 Calculated Values of Network Connectivity Using the Sample Network in Figure 1

Category	Measure	Calculated Index
Connectivity Measures	Degree of a node	$C_B = 1; C_C = 3; C_A = 1; C_D = 2; C_E = 1$
	Cyclomatic number (μ)	$\mu = 0$
	Diameter	$\delta(G) = 25$ mi
	Alpha index	$\alpha = 0$
	Beta index	$\beta = 0.8$
	Gamma index	$\gamma = 0.67$
	Eta index	$\eta = 7.5$ mi per link
	Pi index	$\pi = 1.2$
	Theta index	$\theta = 6$ mi/node
	Iota index	$\iota = 2.31$ mi per weighted node
	Degree of connectivity	d.c. = 2.5
	The Shimmel distance (D-Matrix)	Node 1=8; Node 2 = 5; Node 3= 8; Node 4 = 6; Node 5 = 9
Node Accessibility Measures	Node accessibility index	$A(B,N) = 68; A(C,N) = 38; A(A,N) = 53; A(D,N) = 46; A(E,N) = 67$
	Dispersion	$D(N) = 272$
	Degree of circuitry	DC = 0

For the sample network, using node-accessibility index performance measure, it can be shown that from a topological perspective, Node C has the greatest node accessibility because it costs the least (in terms of distance traveled) to go from Node C to all other nodes in the network, followed by Nodes D, A, E and B, respectively. The cyclomatic number of this network is zero; this suggests that there are no circuits (closed paths) in the network. A transportation network with higher number of independent closed paths, as measured by the cyclomatic number, serves network users better than a network with lower number of independent closed paths in unwanted situations such as natural and man-made disasters (such as flood and earthquake) that cause network disruption, by providing alternative routes in the network. The diameter of the sample network, that is, the length of the longest path between an origin and destination (O-D) pair, is 25 mi. The measures of network connectivity are typically represented in terms of topological concepts such as number of nodes and the number of links. Table 1 presents the calculated values of network connectivity using the sample network in Figure 1.

The alpha index of the network is 0%, that is, the network attains 0% of the maximum possible connectivity; the beta index, which is an indication of network complexity, is 0 because for trees and all disconnected graphs, the beta index is not greater than 0; and the gamma index is a ratio between the actual number of links in the network and the maximum possible number of links (assuming all nodes pairs were connected by a link); thus the gamma index indicates the extent of relative connectivity of the network (42). For the sample network, the value of the

TABLE 2 General Measures of Network Connectivity

Category	Measure	Description	Equation	Source
Connectivity Measures	Degree of a node	The number of nodes directly attached to a node in a network	$C_i = \sum_j^n C_{ij}$	(43, 52)
	Cyclomatic number (μ)	Maximum number of independent cycles of the graph. It measures the extent to which a network is developed, compared to other networks	$\mu = e - v + p$	(44-46)
	Diameter	The length of the longest path between an origin and destination pair	$\delta(G) = \max_x d(x, x)$	(44, 45)
	Alpha index	The ratio between the actual number of circuits in the network and the maximum number of circuits	$\alpha = \mu / (2v - 5)$ (for planar graphs)	
	Beta index	The ratio between number of links and number of nodes in the network	$\beta = e/v$	
	Gamma index	The ratio between actual number of links and the maximum number of links in the network	$\gamma = e / [3(v - 2)]$ (for planar graphs)	(36, 44-46, 53)
	Eta index	A ratio of sum of all links and all nodes to the observed number of links of the network	$\eta = M/e$	(44)
	Pi index	Measures the relation between the entire transportation network and individual links of the network	$\pi = c/d$	(43, 44)
	Theta index	A ratio between the entire network length and its nodes and expresses function of the average index	$\theta = M/V$	(44)
	Iota index	Represented by the ratio between the entire network and its weighted nodes	$\iota = M/W$	
	Degree of connectivity	Compares the relative position of a network's connectivity between the minimum and maximum connectivity ratios	$d.c. = (1/e)[v(v - 1)/2]$	(53)

Continued on next page.

TABLE 2 (continued) General Measures of Network Connectivity

Category	Measure	Description	Equation	Source
Node Accessibility Measures	The Shimmel distance (D-Matrix)	The sum of the number of links in the shortest path between a node and all other nodes in the transportation network	Involves matrix multiplication and manipulation of the resulting matrices	(53)
	Node accessibility index	A measure of the spatial relation between node i and all other nodes in the network	$A(i, N) = \sum_{i=1}^n d(i, j)$	(44)
	Dispersion	A measure of the overall property in terms of node- accessibility of a network	$D(N) = \frac{\sum_{i=1}^n \sum_{j=1}^n d(i, j)}{v}$	(44, 54)
	Degree of circuitry	A measure of the relative location of nodes of a network	$DC = \frac{\sum_{i=1}^n (E-D)^2}{v}$	(44)

NOTES:

C_i = degree of node i ;
 C_{ij} = Connectivity between node i and node j (either 1 or 0);
 n = number of nodes;
 μ = cyclomatic number;
 e = the number of links;
 v = the number of nodes;
 p = number of graphs;
 $\delta(G)$ = Diameter of graph G ;
 α = alpha index for planar graphs;
 β = the beta index;
 γ = gamma index;
 η = eta index;
 M = total network mileage;
 π = pi index;

c = the total length or mileage of the entire transportation network;
 d = the total length or mileage of the network's diameter;
 θ = theta index;
 ι = iota index;
 w = the sum of network's nodes weighted by their function;
 $d.c.$ = degree of connectivity;
 $A(i, N)$ = node-accessibility index;
 $\sum^n d(i, j)$ = summation of distances between node i and all node j 's in the network;
 $D(N)$ = dispersion of network N ;
 $i = 1$
 $\sum^n \sum^n d(i, j)$ = the sum on a sum of distances between node i and all other nodes in the network;
 E, D = real $i = 1 j = 1$ straight line distances, respectively between nodes;
 DC = degree of circuitry.

gamma index is 67%. The eta index of the network is 7.5 mi/link (note that the eta index decreases with increasing number of nodes). Therefore, a lower eta index is indicative of a more developed network. The Pi index of the network is 1.2 (a higher pi index reflects a more developed network). The sample network has a theta index of 6 mi per node which explains the average length per node in the network. On the other hand, the iota index, which takes into consideration the importance of nodes, is 2.31 mi. In the sample network, the end points and the interior (intersection) nodes were taken as having two and eight practical functions, respectively. The degree of connectivity of the network is 2.5 which shows the relative position of the network's connectivity between the maximum connectivity (which is 1) and the minimum connectivity.

The D-matrix or Shimmel distance is a tabular display of the number of links associated with each origin and destination node pair in the network. For the sample network provided, using Node E as the start node, Node C is the most accessible network as it takes the least number of links to travel from that node to every other node in the network; Node E is the least accessible node. Node B has the highest node accessibility index (68 mi) and Node C has the least (38 mi). The network dispersion measures the total length (in units of distance) associated with traversing a node to every other node in a network. The network dispersion is 272 mi. The degree of circuitry of the network is 0 which implies that the real distance between any two nodes in the network is the straight line (the shortest) distance between the nodes. This is obvious for the network example because straight line connection between nodes are assumed; however, in certain application contexts including rolling terrain, the real distances between nodes may be different from the straight line distances.

The degree of a node (DN) is a representation of the node's importance relative to others in a network. From a topological viewpoint, the level of nodal importance is directly proportional to the number of incident links to the node. For example, hub nodes are generally considered to be more important compared to terminal nodes (43).

The diameter of a transportation network can be used to represent the network extent which, in topological terms, refers to the number of links in the network. The drawback of this connectivity measure is that two different networks may have the same diameter due to their difference in degree of connectivity. Conversely, two networks with the same extent may have different diameter; a network with higher degree of connectivity is generally more likely to have a lower diameter (44).

The closeness centrality (CC) of a node depends on its geographical location in the considered network under consideration. The link betweenness centrality (LBC) is defined as the number of shortest paths in a network that pass through a link (39). Different indices have been developed by researchers to quantify the topological performance of networks (43–47).

Transportation infrastructure projects can and do change the topological characteristics of the transportation network, and therefore affect how stakeholders carry out routing operations on the network in order to minimize the cost of doing businesses. Therefore, in order to improve network performance by implementing projects, it is vital to know the different types of trips that typically could be made by the stakeholders. The most common trip types include shortest paths and Chinese postman path (CPP).

Shortest paths are often preferred when sending goods, services or information from an origin to a destination in a network. This is because the shortest path between any two nodes in the network is the optimal route in terms of the cost, or convenience associated with traversing the path (48). A number of network topology performance measures incorporate, directly or

indirectly, the concept of shortest paths. For example, betweenness centrality of a node is a measure of the percentage of all shortest paths in the network that pass through that node in the network. Other performance measures associated with shortest path length and used in this study include the BC of a link and network diameter. For a highly-connected network, it is reasonable to expect that the shortest-path length between any two nodes in the network is generally likely to be small.

The Chinese postman problem (CPP) is one of combinatorial optimization problems that are widely studied and useful problems to solve (49). It has been applied to solve problems such as analysis of DNA, routing robots, routing snow removal in winter season or planning road maintenance activities (50). If G is a connected network containing N vertices and links, then the CPP is about finding a closed path in the network, that contains all links of G and the total cost of the closed path is the minimum (49).

Summary of the Literature Review

The network connectivity measures that exist in the literature are not comprehensive, i.e., they deal with a single attribute of network topological performance (37, 38, 45, 46). Also, the existing measures do not incorporate weights that reflect PM preferences to specific measures of network topology based on their appropriateness with respect to the policy or operations of transportation agency or service organization (36). That is, there is no widely used comprehensive network PM that enables decision makers to incorporate special consideration to specific routes (links or nodes in the network) or specific measures of network performance. Table 2 presents the general measures of network connectivity.

STUDY METHODOLOGY

The framework developed in this study allows the selection of network PMs of interest to be included in the model as well as to provide individual weights for the measures and their sub-criteria. The framework also can be applied both to proposed or existing networks for quantifying the overall topological performance of a network or the performance of individual nodes and links. The steps in the developed framework are described below.

- Consider planned or existing network. The methodology can be used to quantify the network connectivity and node-accessibility performance of proposed transportation networks, proposed improvements to an existing network, or existing network without improvement.
- Select network PMs. Based on the context of the evaluation, the network PMs should be selected appropriately. For example, if the objective is to quantify the percentage of shortest paths that pass through links, a link BC PM is considered in the evaluation.
- Provide weights for PMs. At this stage, the decision maker assigns weights to the individual measures of network topological performance which reflect the relative importance of the PMs compared to each other, and are generally derived by the decision maker on the basis of the inputs of multiple stakeholders.
- Scale PMs (whenever necessary). PMs may have different units of measurement, which in some cases may be unit-less. In order to be able to quantify network connectivity, these PMs are converted into the same scale of measurement.

- Determine values of topological PMs. The values of the topological measures are determined by applying suitable formulas and algorithms. The topological performance values may be associated with nodes, links, or the entire network, as described in the next section.
- Analyze node, link, and network importance. In this step, the PM values obtained in the previous step are analyzed and the nodes or links are prioritized by their importance. The entire network performance is also determined.

General Form of the Index

The proposed model was conceptualized in order to quantify network topological performance as a single composite quantity that incorporates multiple PMs. The general proposed model is given in Equation 1.

$$NCI = \frac{\sum_{k=1}^l \sum_{i=1}^n W^k \alpha^k \gamma_i^k w_i^k}{\sum_{t=1}^s \sum_{j=1}^m W^t \alpha^t \beta_j^t w_j^t} \quad (1)$$

where

- NCI = network connectivity index;
- k or t = PM;
- l or s = number of trip types or criteria;
- n or m = number of nodes, routes, or links;
- i or j = node, link, or criteria;
- W^k = weight given to a PM k ;
- α^k = normalization factor for network PM k ;
- γ_i^k = value of network PM k for a node, link, or criteria i ;
- w_i^k = weight for network PM k for node, link, or criteria i ;
- W^t = weight given to a PM t ;
- α^t = normalization factor for network PM t ;
- β_j^t = value of a network PM t for a node, link, or criteria j ; and
- w_j^t = weight for network PM t for a node, link, or criteria j .

The normalization factor is an adjustment that seeks to cancel out bias due to the effects of certain network features or properties. For example, in comparing the diameter of two or more network topologies which differ in their number of links, it is essential to cancel out the effect of the number of links because those network topologies with a higher number of links are likely to have a smaller diameter due to the possibility of more route options which could reduce the diameter value.

In some situations, a network PM may have sub criteria. In these cases, individual weights could be assigned to the sub criteria. Equation 2 specifies that the sum of the weights of the sub criteria of a network PM should be equal to the weight given to that network PM:

$$\sum_{i=1}^n w_i^k = W^k \text{ and } \sum_{j=1}^m w_j^t = W^t \quad (2)$$

where the definitions of symbols are the same as given in Equation 1.

The model places the network PMs which the decision maker seeks to maximize in the numerator, and those to be minimized in the denominator. For example, if the objective involves maximization of the average nodal degree and the minimization of the network diameter, the former appears in the numerator and the latter in the denominator.

Generalized Form of the Index

To demonstrate the application of the general network connectivity model described in Equation 1, the following PMs were considered: node betweenness centrality, link betweenness centrality, nodal degree, closeness centrality, shortest path length, Chinese postman cost, and network diameter. Equation 3 provides the network connectivity index (NCI) model which incorporates these PMs and was used in a case study that is described in the subsequent sections of this paper.

$$NCI = \frac{\frac{W^{BC}}{n} \sum_{i=1}^n \gamma_i^{BC} w_i^{BC} + \frac{W^{LBC}}{l} \sum_{m=1}^l \gamma_m^{LBC} w_m^{LBC} + \frac{W^\theta}{n} \sum_{i=1}^n \gamma_i^\theta w_i^\theta + \frac{W^{CC}}{n} \sum_{i=1}^n \gamma_i^{CC} w_i^{CC}}{\frac{W^{SPL}}{n(n-1)/2} \sum_{i=1, j=1, i \neq j}^n \beta_{i,j}^{SPL} w_{i,j}^{SPL} + \frac{1}{n} \beta^{MST} W^{MST} + \frac{W^{CPP}}{n} \sum_{i=1}^n \beta_i^{CPP} w_i^{CPP} + \frac{1}{n} \beta^D W^D} \quad (3)$$

where

- NCI = network connectivity index;
- BC = betweenness centrality;
- LBC = link BC;
- θ = degree of a node;
- CC = closeness centrality;
- SPL = shortest path length;
- MST = minimum spanning tree;
- CPP = Chinese postman cost;
- D = network diameter;
- n = number of nodes;
- l = number of links;
- m = name of link;
- i, j = name of node;
- $\frac{1}{n!}, \frac{2}{n(n-1)}$ = normalization factors;
- γ_i^{BC} = BC value for node i ;
- γ_m^{LBC} = LBC value for link m ;
- γ_i^θ = value for DN i ;
- γ_i^{CC} = closeness centrality value for node i ;
- $\beta_{i,j}^{SPL}$ = shortest path length between nodes i and j ;
- β^{MST} = Length of minimum spanning tree;
- β_i^{CPP} = Chinese postman cost that starts and ends at node i ;
- β^D = network diameter value;
- W^{BC} = weight for BC PM;
- W^{LBC} = weight for LBC PM;

- W^θ = weight for nodal degree PM;
- W^{CC} = weight for closeness centrality PM;
- W^{SPL} = weight for shortest path length PM;
- W^{MST} = weight for minimum spanning tree PM;
- W^{CPP} = weight for Chinese postman cost PM;
- W^D = weight for network diameter PM;
- w_i^{BC} = weight for node i w.r.t BC PM;
- w_m^{LBC} = weight for link m w.r.t LBC PM;
- w_i^θ = weight for node i w.r.t nodal degree PM;
- w_i^{CC} = weight for node i w.r.t closeness centrality PM; and
- $w_{i,j}^{SPL}$ = weight for SPL PM for shortest path between nodes i and j .

Equation 4 satisfies the condition that the sum of the weights of sub-criteria must equal the weight given to the network PM to which the sub-criteria belongs.

$$\begin{aligned} \sum_{i=1}^n w_i^{BC} = W^{BC}; \quad \sum_{m=1}^l w_m^{LBC} = W^{LBC}; \quad \sum_{i=1}^n w_i^\theta = w^\theta; \quad \sum_{i=1}^n w_i^{CC} = W^{CC} \\ \sum_{i=1, j=1, i \neq j}^n w_{i,j}^{SPL} = W^{SPL}; \quad \sum_{m=1}^l w_m^{CPP} = W^{CPP} \end{aligned} \quad (4)$$

The definitions of symbols are the same as given in Equation 3.

Weighting and Scaling

Transportation network stakeholders typically have different network performance preferences with regard to certain PMs based on their importance in the stakeholder's day-to-day operations. Therefore, in the framework for NCI development in this study, the stakeholders were requested to assign weights to the PMs as well as the individual elements of the network. The weights for the PMs and their sub-criteria were determined through a questionnaire survey. Scaling (also referred to as metricization) of PMs is common in multiple criteria evaluation, particularly when there is a need to combine the PMs to yield a single combined value of overall performance for each alternative (20). The PMs used in this study to quantify network connectivity have different units of measurement. There is a need, therefore, to scale the units so that they can be represented in the same scale of measurement. That way, the PMs can be used to characterize the network connectivity level described by Equation 3. The PM values were scaled to the same scale of measurement using Equation 5.

$$PM_{scaled} = \left(1 - \frac{PM_{max} - PM}{PM_{max} - PM_{min}} \right) * 100 \quad (5)$$

where

- PM_{scaled} = scaled PM value;
- PM = actual PM value;
- PM_{max} = maximum computed PM value; and

PM_{\min} = minimum computed PM value.

For example, if the actual BC of a node in a network is 20 percent and the minimum and maximum BCs of the nodes in the network are 10 and 40 percent, respectively, using Equation 5, their scaled value, PM_{scaled} is 33.33%.

CASE STUDY

A case study involving road infrastructure network was conducted to demonstrate the developed network connectivity framework. The infrastructure case study network, shown in Figure 2, consists of 17 nodes and 24 links. The numbers along each link represent the impedance of the link. Impedance can be measured as a distance (miles) or travel time (minutes).

Stakeholders

There are a number of transportation stakeholders in the case study area, such as post office, farmers, agricultural companies, school bus systems, and so on. Due to the limitations of time and difficulty in collecting data for the entire population of stakeholders, only representative stakeholders were solicited for data: these are coded as follows: Stakeholders A, B, C, and D.

Survey Results

The survey questions that measured various aspects of the network performance were prepared by considering all network PMs incorporated in Equation 3. With regard to the weights assigned by the stakeholders to the network features, the average weight provided by stakeholders to the nodes of the network with respect to each PM was recorded. In the case study, all nodes were provided weights of 5 or greater by all stakeholders; the only exceptions are Nodes 10 and 11 that received weights slightly lower than 5 for the case of the CPP and the LBC PMs. Also, node 12 received a weight less than 5 for the CPP PM. Each node was assigned a different weight based on the PM under consideration, showing that the importance of a node relative to other nodes depends on the type of PM under consideration.

With regard to the weights assigned by the stakeholders to the topological performance measures, the average weights were also recorded. For the network diameter PM, Stakeholder A assigned the highest importance (with an average weight of 8.2) to this PM followed by Stakeholder B (weight equals about 7.9). Stakeholder C assigned a weight of 4. The lowest weight for this PM was assigned by Stakeholder D. A higher weight was assigned by Stakeholder B, most likely due to their need to be able to reach all corners of the network. A lower diameter means that the Stakeholder B can quickly arrive at the incident location and provide services anywhere in the network when needed. On the other hand, Stakeholder C and Stakeholder D have routine routes in the network and therefore may not be highly concerned with increasing or decreasing the network diameter because changes in the network diameter may not affect their routine business.

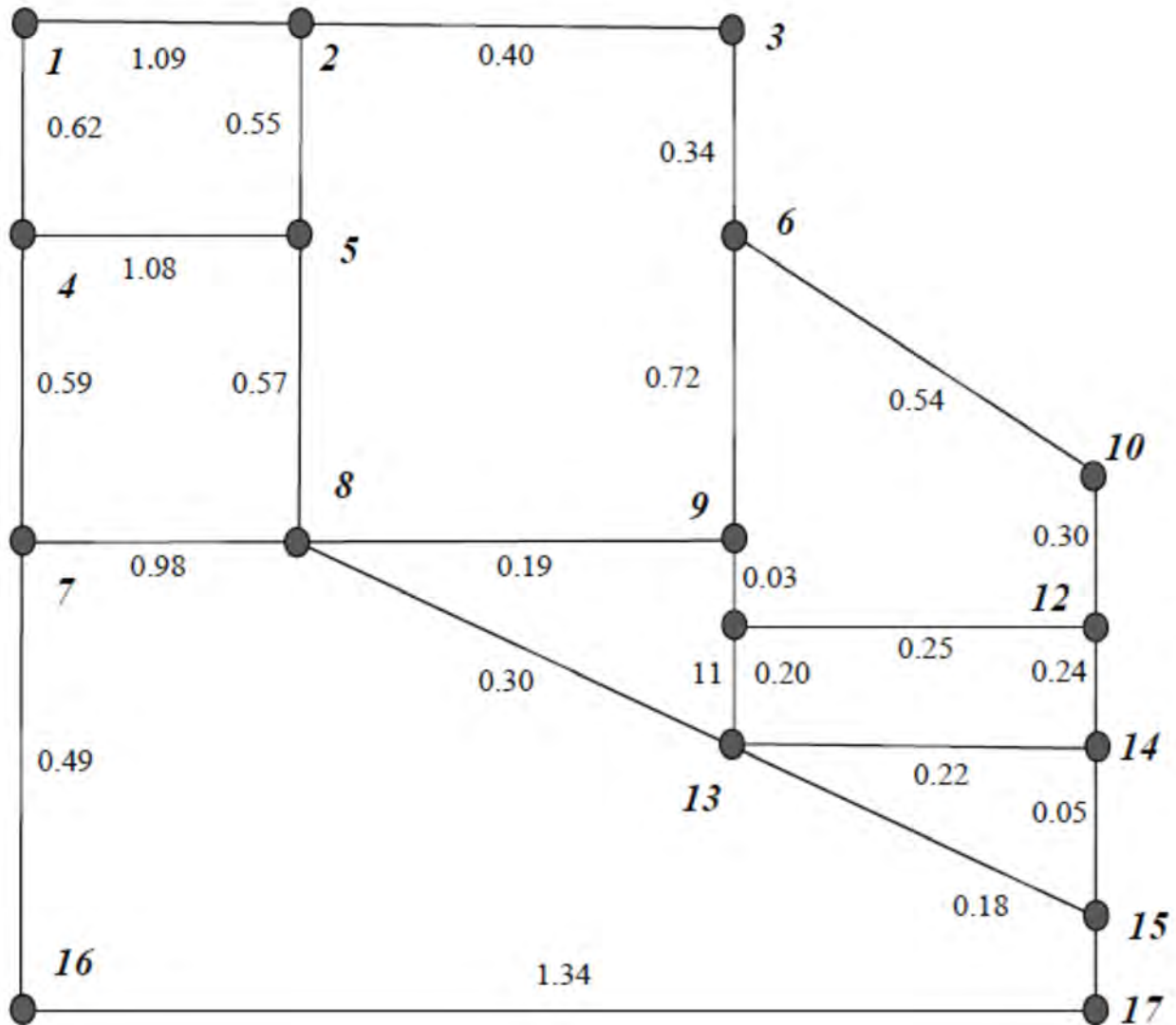


FIGURE 2 Case study network of low-volume roads.

Categories of Stakeholders’ Preferences for Nodes and Links

Survey results showed that the stakeholders considered in this study provided variable degree of importance to different nodes, links, and PMs. Some of the results are described below.

It was found out that for all the PMs, no node had the same degree of preferences as per the preference criteria. For example, Node 8 was assigned a preference of 8 or greater for all PMs, except for BC in the case of Stakeholder D and Stakeholder B, and for CPP in the case of Stakeholder D. A given stakeholder had different preferences for a given node for different PMs. For example, Stakeholder C assigned the highest preference for nodes 8 and 13 w.r.t nodal degree (ND) PM; Node 13 for BC; Nodes 1, 2, 3, 4, 8, 13, and 17 in the case of CC; and Nodes 1, 2, 8, 13, 16, and 17 for the case of CPP. On the other hand, Stakeholder C assigned its lower preferences to Nodes 10, 11, 12, 13, and 14 for the network diameter PM; Nodes 9 and 11 for BC; Nodes 4, 7, 9, 10, 11, 12, 14, 15, and 16 for CC; and Nodes 6, 10, 11, and 12 for CPP PM.

The above results generally show that transportation decision-making could incorporate the preferences of stakeholders for network elements (nodes and links) in a given transportation network with respect to the PMs of interest. This approach is useful for minimizing the negative impact that an infrastructure project may have on some stakeholders and an unfairly large advantage for other stakeholders.

Network Connectivity Index

In determining the NCI for the case study network given in Figure 2, values of all PMs were first computed using python code and Graph Magics® software for computing CPP (51). Then, the PM values were scaled into the same unit of measurement in order to apply the NCI model. The NCI was determined using average weights provided by stakeholders to each network element corresponding to each PM. For network-level PMs (such as network diameter), a single weight was given; there was no need to consider the network elements. Equation 6 shows a simplified NCI model for the case network after inserting constant values into the NCI equation.

$$NCI = \frac{\frac{8.00}{17} \sum_{i=1}^n \gamma_i^{BC} w_i^{BC} + \frac{7.75}{24} \sum_{i=1}^l \gamma_m^{LBC} w_m^{LBC} + \frac{8.75}{17} \sum_{i=1}^n \gamma_i^{\theta} w_i^{\theta} + \frac{9.25}{17} \sum_{i=1}^n \gamma_i^{CC} w_i^{CC}}{\frac{8.25}{17(17-1)/2} \sum_{i=1, j=1, i \neq j}^n \beta_{i,j}^{SPL} w_{i,j}^{SPL} + \frac{9.5}{17} \sum_{i=1}^n \beta_i^{CPP} w_i^{CPP} + \frac{8}{17} \beta^D} \quad (6)$$

The constant values are the average weights of PMs given by all stakeholders as shown in the list below, and the number of nodes and links in the network, which are 17 and 24, respectively. The list below provides scaled and weighted sum of PM values. These were calculated by multiplying the PM value corresponding to each node or link in the case network by the average weight given to the same node or link by the stakeholders. Due to the scaling process, these values are dimensionless.

ND: 265.44
 Betweenness Centrality: 232.49
 LBC: 289.18
 CC: 271.99
 Shortest Path Length: 2031.82
 CPP: 651.47
 Diameter: 55.88

If the weighted sum values of the PMs are inserted into Equation 1, the NCI is determined as 1.04. When different projects on the LVR system are considered for implementation, the corresponding NCI values can be determined. LVR projects which provide higher NCI values are preferable to those that yield lower NCI values. A thorough investigation of the impact of the projects on the NCI should be conducted when two or more LVR projects are considered for simultaneous implementation; the maximum possible number of combinations of projects should be evaluated in order to select projects which relatively maximize the NCI value. For example, if there are 10 LVR candidate projects and only two projects are to be implemented due to other influencing factors (such as budget constraint), $\binom{10}{2} = \frac{10!}{(10-2)!2!} = 45$ combinations of projects should be considered and the corresponding 45 NCI values should be

computed and ranked, and the projects that provide the highest NCI values should be selected for implementation based on the network connectivity evaluation criteria.

SUMMARY AND CONCLUSIONS

In this paper, a general network connectivity model was introduced. A case study network of low volume roads was used to demonstrate the proposed overall network connectivity framework. The survey results on stakeholders' preferences regarding individual measures of network topological performance and regarding specific features of the network (nodes and links) were presented. The network connectivity computation and scaling of performance measures was also demonstrated. Finally, the network connectivity index (NCI) of the case study network was determined using the developed framework. The results show that stakeholders tend to give different levels of importance to nodes and links in the network from the perspective of their operations. The framework can be utilized to determine more than one NCIs when multiple projects are considered for implementation; a project associated with the highest NCI could be selected. Future work could include this measure of performance in multi- criteria evaluation that considers other evaluation factors such as safety, noise, emissions, in addition of the network connectivity impacts of proposed projects.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: Study Conception and Design: Wubeshet Woldemariam, Asif Faiz, and Samuel Labi; Literature Review: Wubeshet Woldemariam and Samuel Labi; Data Collection: Wubeshet Woldemariam; Manuscript Preparation: Wubeshet Woldemariam, Samuel Labi, and Asif Faiz. All authors reviewed the results and approved the final version of the manuscript.

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Connectivity, Accessibility, and Mobility Relationships in the Context of Low-Volume Road Networks

SAMUEL LABI
BORTIORKOR NII TSUI ALABI
TARIQ U. SAEED
Purdue University

ASIF FAIZ
Faiz and Associates, LLC

WUBESHET WOLDEMARIAM
Purdue University Northwest

The concepts of connectivity, accessibility, and mobility (CAM) are key measures of transport network performance that have been discussed extensively in the literature. However, there has been little work that discussed the relationships among these concepts. A clear discourse on these concepts and their interrelationships can help agencies carry out more objective evaluations of projects that seek to improve at least one of these measures of transportation performance. This paper presents three alternative perspectives (models) of the CAM relationship: the nested, the snowman, and three-way overlapping models. The paper also presents, for project appraisal purposes, two alternative ways of classifying the three CAM concepts. The first is based on the three concepts in their basic forms. The second considers some variation of these concepts in addition to aspects of the network topology, operational performance, road condition, and socioeconomic characteristics of the project's area of influence. The conceptual framework outlined in this paper contributes towards a holistic approach to the appraisal of low-volume road projects, programs, or existing networks based on their impact on overall CAM.

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Review of Local Roads' Model Inventory of Roadway Elements Fundamental Data Elements

SERI PARK
LESLIE MCCARTHY
Villanova University

ANTHONY R. GIANCOLA
Transportation and Local Government Consultant

According to the Moving Ahead for Progress in the 21st Century Act (MAP-21), all states are required to collect roadway data based on the Model Inventory of Roadway Elements (MIRE) as well as to use the linear referencing system. Furthermore, Fixing America's Surface Transportation (FAST) Act emphasizes high-quality data that will aid practitioners to reach informed decision-making. The above facts underline the need for a thorough review of how states are engaging local agencies to meet the data collection requirement, especially for the local roadway system that encompasses low-volume local roads and unpaved roads. More specifically, identification of successful practices in integrating and maintaining roadway data from various sources (e.g., different local agencies' data sources) will help states achieve the data collection requirements. In this paper, a nationwide survey summary pertaining to MIRE Fundamental Data Elements (FDE) collection and identified sample successful practices are described.

INTRODUCTION

The MIRE is a recommended list of roadway inventory and traffic elements related to safety management. A subset of MIRE, established as part of the Highway Safety Improvement Program Final Rule changes, is referred to as FDEs. FDEs contain 37 roadway and traffic elements that are categorized by roadway functional classification and surface type—including the low-volume local roads as well as unpaved roads (Table 1). It should be noted that according to the FHWA (1), unpaved roads account for approximately 35% of the roadway system in the United States.

In MAP-21 (2), the significance of MIRE FDE is well noted as the requirement for the collection of roadway data based on the MIRE. The recent FAST Act (3) further underlines the importance of high-quality safety data to support comprehensive transportation decision-making. To meet the requirement to collect data on all public roadways based on the MIRE, state agencies are putting effort to coordinate with local agencies to gather data that are available at the local level. Considering that MIRE FDE also encompasses local paved roads and unpaved roads (Code 7 per FHWA Highway Performance Monitoring System), it is imperative for state agencies to understand the current status of MIRE FDE collection of local roads, degree of MIRE FDE integration into the state database system, identified barriers and successful practices in data integration. This paper synthesizes the aforementioned elements based on the survey of state transportation agencies and interview with selected agencies, which is based on the authors' previous research (4).

TABLE 1 MIRE FDE for Local Paved Roads and Unpaved Roads

Local Paved Roads (Roadway Segment)	Unpaved Roads (Roadway Segment)
Segment identifier	Segment identifier
Functional class ^a	Functional class ^a
Surface type ^a	Type of governmental ownership ^a
Type of governmental ownership ^a	Begin point segment descriptor ^a
Number of through lanes ^a	End point segment descriptor ^a
Average annual daily traffic ^a	
Begin point segment descriptor ^a	
End point segment descriptor ^a	
Rural–urban designation ^a	

^a = Represents the required Highway Performance Monitoring System items.

METHODOLOGY

In this paper, a total of three approaches are presented for capturing the various efforts that have been made by states in working with local public agencies to integrate roadway safety data from local and state sources. These approaches are: (1) a comprehensive literature review; (2) a survey distributed to the state departments of transportation (DOTs); and (3) a series of interviews with local public agencies (LPAs) and FHWA division personnel. Based on the state DOT survey results and the subsequent in-depth interviews with LPAs, the noteworthy practices associated with the partnership between state and local agency are summarized.

FINDINGS

State Survey Results

Survey results gathered from 45 states showed that the advancement in technologies and data process–visualization [e.g., GPS, geographic information system (GIS), lidar] helped state and local agencies in streamlining roadway data collection and integration process. Consistent data format between state and local sources and adequate state resources were elements identified to improve the integration of roadway data MIRE FDE from state and local sources. There was a total of 11 states that reported to have 100% completed MIRE FDE collection efforts for non-state–owned roadways including the unpaved local roadways.¹ The survey also showed that eight states rated their roadway safety MIRE FDE integration into the state system effort as an effective one.² Some of those states will be further described in the next section.

With the MIRE FDE from all public roads, many states have indicated benefits as a more integrated decision-making, improved project identification and priority setting, and enhanced roadway safety level using the data-driven approach. The limited state and local resources, technical expertise of the agency, and inconsistent funding levels were identified as barriers in implementing the integration of local roadway safety data.

In-Depth Interview

To further obtain details of the successful practices in roadway data integration, interview results are described in four topic areas:

1. New technology–software application;
2. A healthy communicative and collaborative relationship between state and local agencies;
3. Engagement with academic partners; and
4. Development of data dictionary, standards, and manual.

Technology Application

For the past decade, the Ohio DOT, the Ohio Department of Public Safety (ODPS), and county engineers have collaborated to gather and maintain a common file of roadway inventory data that includes the MIRE FDE needed for assessing safety. Referred as the Location-Based Referencing System (LBRS) and funded through various sources (e.g., ODPS 408 grants, Ohio DOT Highway Safety Improvement Program funds), the corresponding project applied GIS and helped Ohio DOT enhance crash location rate. As a state–county partnership product, the LBRS enables to gather accurate locational information on all public roads and addresses. Prior to the LBRS project, Ohio DOT located for about 30% of all crashes within a given county due to the inaccurate or incomplete local (non-state–owned) roadway databases. With the roadway inventory file through LBRS, approximately 90% of all crashes are now locatable regardless of jurisdiction. Figure 1 presents a screenshot of Ohio Geographically Referenced Information Program’s LBRS (5).

Collaborative Relationship

Interview with various local agencies showed a close relationship among state and local staff is an essential factor in facilitating a successful and coordinated data collection, integration and maintenance effort. A good example is the County Road Administration Board (CRAB) and Washington State DOT (6). CRAB acts as a clearinghouse by confirming compliance, quality work, training, and maintaining the data for each of the counties. In addition, CRAB collaborates with the Washington State DOT to assist identify gaps and overlaps in the road inventory system, to increase the success of accurately locating jurisdictional boundaries, to explore multimodal opportunities, and to foster data management sharing.

Engagement of Academic Partner

In Michigan, Michigan DOT will lead the efforts the MIRE FDE collection while the Michigan Technological University (MTU) will conduct support services. Table 2 describes the MIRE FDE data collection plan in partnership with MTU.

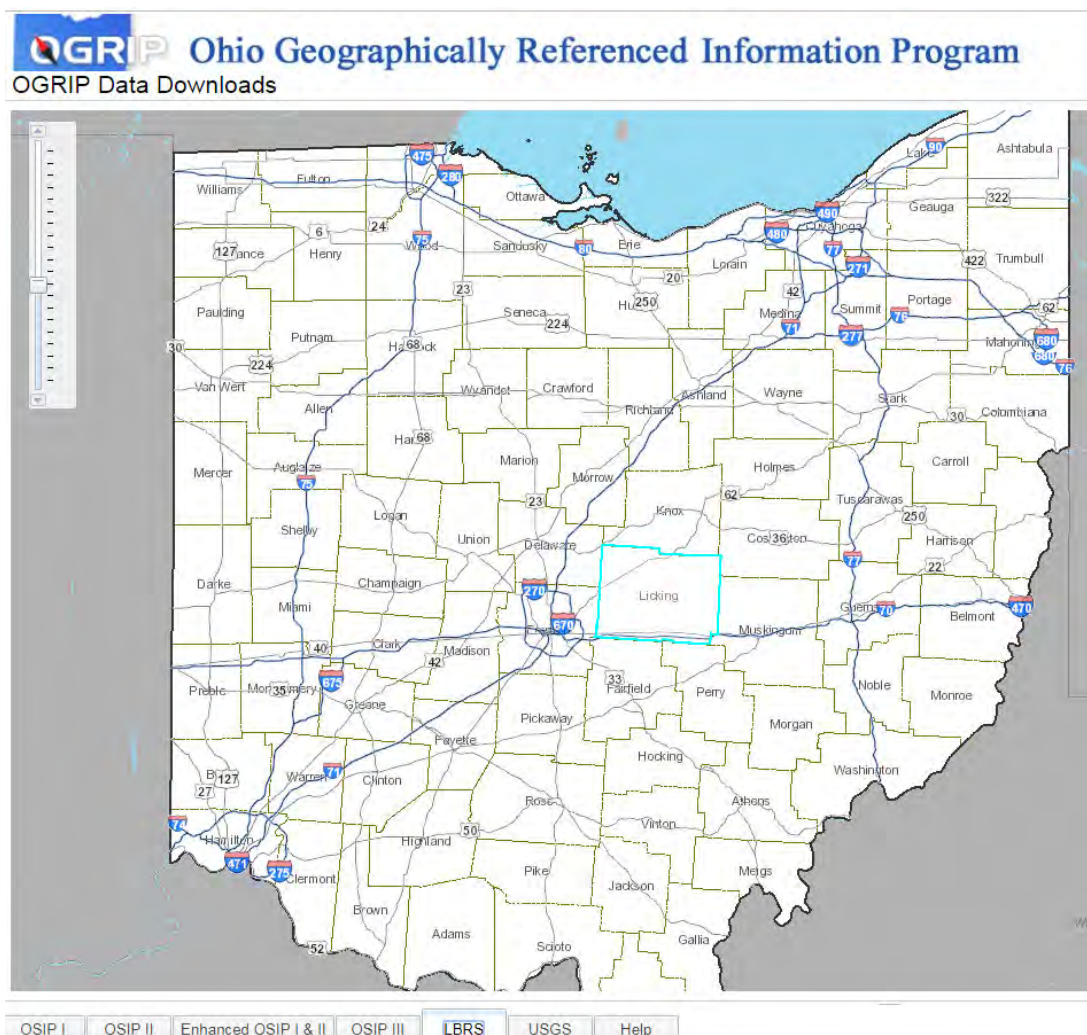


FIGURE 1 LBRS in Ohio, Ohio Geographically Referenced Information Program (5).

Another example is the state of Iowa and Institute for Transportation (InTrans) at Iowa State University. The Center for Transportation Research and Education at InTrans has been responsible for generating a statewide intersection database and filtering—complementing a roadway horizontal curvature database that are significantly related to the MIRE FDE components. Iowa State University’s InTrans supports the Iowa DOT with monitoring the pavement condition data collection, quality assurance, and data distribution to the locals.

Development of Data Dictionary, Standards, and Manual

In the state of Kentucky, the Kentucky Transportation Cabinet (Kentucky DOT) contracts with Area Development Districts (similar to a regional planning organization) to work with local agencies to collect non-state-road roadway data. During this process, the Kentucky DOT requires the data collection to confirm the DOT’s system based on the well-established data dictionary, the *Local Road Update Standards* (7). Kentucky DOT also developed *Standards for Road Data Collection Using Global Positioning System Techniques* which is used for the

TABLE 2 Michigan DOT MIRE FDE Collection Plan Schedule

Fiscal Year	Plan Description
2017-2018	Michigan DOT and MTU will conduct a survey focusing on identification of 1) collected MIRE FDE and 2) its data format.
2018	Michigan DOT will build a state-level based MIRE FDE repository using GIT that includes server configuration, a MIRE FDE model, web mapping application, two-way web services, and a reporting tool. The Unified Work Program (UWP) asks for the participation of metropolitan planning organizations and regional planning commissions to participate in the state's outreach efforts to refine the MIRE FDE collection plan. A main goal of the outreach is to determine the time and cost required.
2019	Michigan DOT will establish a pilot with a few select partners using the new tools and the data repository to collect and exchange MIRE FDE. The goal is for Michigan DOT to enhance the MIRE FDE model, web mapping app, two-way web services, and reporting tools as determined appropriate from the pilot. Michigan DOT will work with MTU to develop training and outreach materials. Continued partner participation is the goal in the FY 2019 UWP, details to be determined.
2020-2026	The first year of MIRE FDE collection occurs in 2020. Roadsoft MIRE FDE webinars and materials are offered to partners and Michigan DOT. Michigan DOT will engage partners to determine the best methods and frequency to collect MIRE FDE to ensure that the state is compliant and can report all the required data by September 30, 2026. Continued partner participation is the goal in the FY 2020–2026 UWP, details to be determined.

effective data collection with GPS (8). Currently, the Kentucky DOT is planning to develop average annual daily traffic estimates on local roads where no actual values are being collected.

CONCLUSIONS

MAP-21 requirement of roadway data collection of all public roads based on MIRE FDE denotes the importance of effective data collection and integration from various sources. State survey and interview with local agencies indicate the technology advancement, coordination and partnership among agencies, and the establishment of data format standards and manual are essential for successful data collection and integration. Identified knowledge gaps include further investigation of the cost of developing various aspects of the roadway safety MIRE FDE program, limited state and local resources, and lack of MIRE FDEs for tribal nation roads.

NOTE

1. Colorado, Illinois, Kentucky, Louisiana, Montana, Nebraska, New Hampshire, Nevada, Rhode Island, Tennessee, and Wisconsin.
2. Kentucky, Illinois, Iowa, Louisiana, Massachusetts, New Hampshire, South Dakota, and Vermont.

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Using Travel Demand Model to Estimate Traffic Volumes on Low-Volume Roads in Wyoming

ER YUE

KHALED KSAIBATI

University of Wyoming

Low-volume roads connect between rural areas and markets, and they are crucial parts of transportation systems. However, traffic volume estimation on low-volume roads is usually ignored compared to that on high-volume roads. This study developed a four-step travel demand model (TDM) to estimate average daily traffic (ADT) on low-volume roads in Wyoming. Four types of trips, including person trips, crop production freight trips, oil production freight trips, and tourism trips, were evaluated in the model to estimate ADT. The model had a reliable prediction accuracy, with an R-square greater than 0.8 compared to the actual traffic volumes. The model outputs indicated that person trips and tourism trips are two main traffic generators on low-volume roads in Wyoming. Tourism trips play a significant role in Northwest Wyoming, where Yellowstone National Park is located. Crop production freight trips had impacts on low-volume roads near farms. The model developed in this study was capable to capture traffic flows on low-volume roads. The model is recommended for use by government agencies in other states or regions for traffic prediction and transportation planning on low-volume roads.

INTRODUCTION

Low-volume roads, defined as roads having fewer than 400 vehicles per day, are crucial parts of transportation systems. They serve as important links between rural areas and markets (1). Low-volume roads in rural areas have been largely ignored by transportation planning and maintenance compared to high-volume roads. Traffic volume estimation on high-volume roads has received much attention from transportation engineers and researchers. However, in recent years there is a need for reliable traffic volume estimation on low-volume roads focusing on road maintenance and safety issues (2). A well-designed and maintained low-volume road network is essential for regional development and resource management. A variety of methodologies have been used to estimate traffic volumes. Of all the methodologies, a four-step TDM, including trip generation, trip distribution, mode choice, and trip assignment, has been widely used for transportation planning. A TDM is a computer-based model used to estimate travel demand and pattern in future based on a number of factors (3). TDMs are useful tools for transportation planning purposes. The outputs of TDMs can help decision makers make appropriate transportation planning decisions.

The applications of TDMs began in the United States in the 1950s. The models consist of surveys and the development of a computer package for travel forecasting by 1969 (4). They provide estimates of a number of parameters that can be used in transportation forecasting and planning. With the introduction of desktop computers, transportation agencies had access to computing power for developing more sophisticated models. TDMs have been largely used in

statewide models for estimating traffic demand along regional corridors and for intercity corridor studies, bypass studies, and statewide system planning applications such as air quality conformity analysis, traffic impact studies, freight planning, and economic development studies (5). Most state departments of transportation (DOTs) have developed and implemented four-step TDMs for large metropolitan areas. However, some of those advanced models are not applicable for most county or rural roads, which carry a low ADT. Therefore, a TDM that is designed for low-volume roads is needed for local agencies (6).

The State of Wyoming is facing an energy industry development and a tourism industry development in rural areas. In the energy industry, the oil and gas production has increased in recent years. Crude oil production in Wyoming ranks eighth in United States, and in 2016 Wyoming produced 72.6 million barrels of crude oil. Natural gas production in Wyoming ranks sixth in United States, and in 2016 Wyoming produced 1.80 billion MCF (MCF = 1,000 cubic feet) of natural gas (7). In the tourism industry, Yellowstone and Grand Teton National Parks in Northwest Wyoming are among the top 10 most-visited national parks in United States in 2016. Both Yellowstone and Grand Teton National Parks have experienced increasing visitors in recent years. Annual visitors to Yellowstone National Park have increased from 3,000,000 before 2000 to more than 4,000,000 in 2017, and annual visitors to Grand Teton National Park have increased from 2,500,000 before 2000 to more than 3,300,000 in 2017 (8). A transportation management plan is needed to accommodate the rural roads with the higher traffic volumes (9). To provide a better estimation of traffic volumes on low-volume roads, the Wyoming Department of Transportation (WYDOT) funded this study to develop a cost-effective traffic volume estimation model. A four-step TDM was developed for estimating ADT on low-volume roads in Wyoming. Person trips, crop production freight trips, oil production freight trips, and tourism trips were included in the model for estimating ADT. The objectives of this study include (1) identifying the locations of low-volume roads and estimating traffic volumes on low-volume roads and (2) evaluating the impacts of different types of trips on traffic volumes on low-volume roads.

LITERATURE REVIEW

Many studies have focused on the estimation of traffic volumes and the various factors affecting the values. Zhan et al. (10) developed a hybrid framework to estimate citywide traffic volumes. Their results indicated the effectiveness of the proposed framework in traffic volume estimation (10). Kwon et al. (11) proposed an algorithm to estimate real-time truck traffic volumes from single loop detectors on Interstate 710 near Long Beach, California. The algorithm was able to capture the daily patterns of truck traffic volumes and mean effective vehicle length with only 5.7% error (11). In addition to urban areas and Interstate highways, the issues of traffic volumes on low-volume roads have also been addressed in some research. Sharma et al. applied artificial neural networks to estimate average annual daily traffic (AADT) on low-volume rural roads in Alberta, Canada. They found a number of advantages of the neural network approach in AADT estimation compared to the traditional approach (12). Karlaftis and Golias developed a statistical methodology to assess the relationship between rural road geometric characteristics and traffic volumes on rural roadway accident rates. The methodology they developed allowed for the explicit prediction of accident rates on rural roads (13). Raja et al. developed a linear regression model to estimate AADT on low-volume roads for 12 counties in Alabama. Their research concluded that the linear regression model can be used to estimate AADT on low-volume roads

for future application (14). Several states in United States have conducted studies on rural roads. The Virginia Department of Transportation (VDOT) used a trip generation method to estimate traffic volumes on secondary local roadways. This method allowed VDOT's staff to identify eligible traffic links and estimate traffic volumes. There were several benefits, such time and cost savings, of this method, and VDOT will continue to implement the trip generation method for estimating traffic volumes on local roads (15). The Arizona DOT developed a Low-Volume State Routes Study on 22 low-volume state routes to identify opportunities and limitations for each route. The ultimate goal of this study was to determine the potential for reducing costs to maintain these routes. This study proposed a guideline for state route management that can be used by state agencies (16).

A number of factors can affect traffic volumes. Demographic data, including population, household, and employment, are usually used in TDM (17). To improve the model prediction accuracy, some other factors were also included in the previously developed models. In addition to demographic data, Saha and Fricker used some economic factors, such as gasoline price, Consumer Price Index, and Gross National Product to develop models for rural traffic forecasting (18). Tourism trips occupy a major part of traffic volumes in Wyoming, especially near Yellowstone National Park. The use of tourism-related travel data in TDM is one of the ways for transportation planners to incorporate tourism issues into their forecasting, planning, and designing processes. More focus has recently been given to the transportation systems in and near national parks because of the levels of visitor demand exceeding the transportation infrastructure within many parks (19). Several studies have assessed the cost-effectiveness and practicality of alternative transportation solutions, including roads, parking, bus services, and other forms of transit facilities (20). Overall, 42% of the state DOTs and 54% of the other agencies reported that they regularly make use of tourism travel forecasts. Among the state DOTs that do make use of tourism forecasts, the dominant use is for transportation planning (20). In tourism planning, evaluation in new and expanded transportation facilities can serve to support the operation and development of attractions (such as national parks) and identify needs for future maintenance.

There is a growing need for developing TDMs for low-volume roads. First, TDMs are helpful cost-effective tools for evaluating existing low-volume road system, where funding is limited (12). Second, low-volume roads connect between rural areas to the highway system, and they are crucial to both local and national economies (21). Third, although low-volume roads carry less than half of nation's traffic, they account for more than half of the nation's vehicle fatalities, so it is significant to estimate traffic volumes on low-volume roads to address planning and safety issues (22).

DATA COLLECTION

The TDM inputs usually consist of road networks, demographic data, socioeconomic data, and land use types (17). The TDM developed in this study included four trip types: person trips, crop production freight trips, oil production freight trips, and tourism trips. This section will explain the data collection procedures for each of the four trip types.

Person Trips

Population, household, and employment data were obtained from the U.S. 2010 Census. The U.S. census provides the best source for basic demographic data. Three major employment categories (retail, service, and other) were selected based on the standard industrial classification system.

Crop Production Freight Trips

For crop location data, a raster image file of crop cover in Wyoming was downloaded from the U.S. Department of Agriculture (USDA) National Agricultural Statistics Service's website. ArcGIS software was used to convert the raster file into a vector file in order to determine the proportion of areal crop coverage for each crop type. The major crops identified in Wyoming were hay, alfalfa, winter wheat, corn, barley, beans, sugar beets, and oats. Forsythe et al. (23) described the transportation of hay and alfalfa to be too complicated to enable their modeling with the four-step TDM since they can either be stockpiled, distributed with trucks, or even transported out of the State by trucks. Wheat, corn, barley, beans, sugar beets, and oats can be modeled easily based on their direct movements from the field to an elevator. The elevators serve as an interface between the highway and the rail system that transports the products out of the State. As a result, the areas of wheat, corn, barley, beans, sugar beets, and oats were input into the model to estimate crop production freight trips. The crops produced are assumed to have a harvesting season window from July 22 through September 13 (a period of 52 days).

Oil Production Freight Trips

For oil production, the activities generating freight traffic include transportation of water from freshwater wells to oil rigs, transportation of crude oil from the rig to elevators, and transportation of waste water to disposal sites. The data needs for estimating the trips generated by oil and gas activities are (1) oil rig location data including information on monthly oil and water production in crude oil barrels, (2) location of oil elevators, (3) freshwater well locations and production, and (4) injection wells. These datasets were downloaded from the Wyoming Oil and Gas Conservation Commission's website. Aggregated oil productions in tons, freshwater demands, and wastewater production were computed. Oil production tons per day were converted to trucks using a conversion factor of 220 barrels per truck. Freshwater demand in tons was also converted to trucks and served as the attraction for freshwater wells. The amount of wastewater produced as a byproduct of oil production was also converted to trucks and transported to injection wells for disposal.

Tourism Trips

For tourism trip estimation, ADT at national/state park entrances, park areas, and number of campsites in park were used as model inputs. Figure 1 shows the spatial distribution of national and state parks as well as road network in the study area. The year of 2014 park visitation data were used in this study since the actual traffic count data used for model validation were mostly obtained in the summer of 2014. The National Park Service (NPS) collects visitation data for all

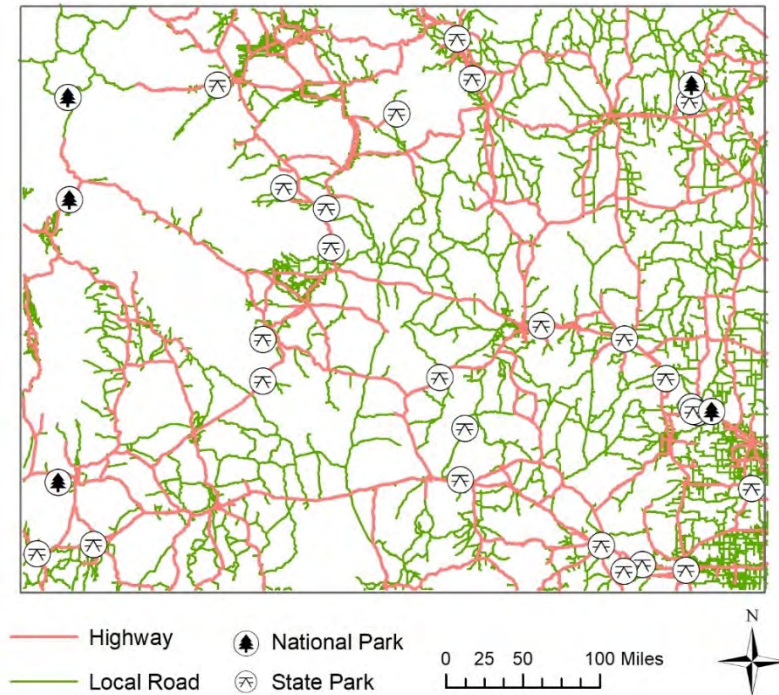


FIGURE 1 Parks and road network included in this study.

national parks, and the Wyoming State Parks and Historic Sites (WSPHST) collects visitation data for 25 state parks and historic sites. Parks without visitation data are either due to no public access or limited management by the WSPHST. Visitation data obtained from the parks were then converted to ADT values as model inputs.

MODEL DEVELOPMENT

Figure 2 shows the overall model development procedure for estimating traffic volumes. In addition to input parameters for four trip types, road network data is also required to represent the transportation system. An ArcGIS shapefile of all roads within Wyoming was downloaded from the WYDOT website. With the help of aerial photos, the shapefile was reviewed to correct connectivity issues and to digitize parts of the network that were missing. Additional data on the roads were obtained and added as attributes of the shapefile. The additional data included the direction of traffic (one- or two-way roadway), number of lanes, speed, and road functional class. A four-step TDM, including trip generation, trip distribution, mode choice, and trip assignment, was developed by Citilabs' Cube software in this study. Esri's ArcGIS software was also used to perform some spatial and network analysis. Three ArcGIS shapefiles: road network, transportation analysis zones (TAZs), and actual traffic counts were used in the procedure of model development and model validation. Many parameters need to be considered for model development, primarily including socioeconomic data, land use types, and roadway networks. Socioeconomic and land use data are usually arranged into geographic units to design

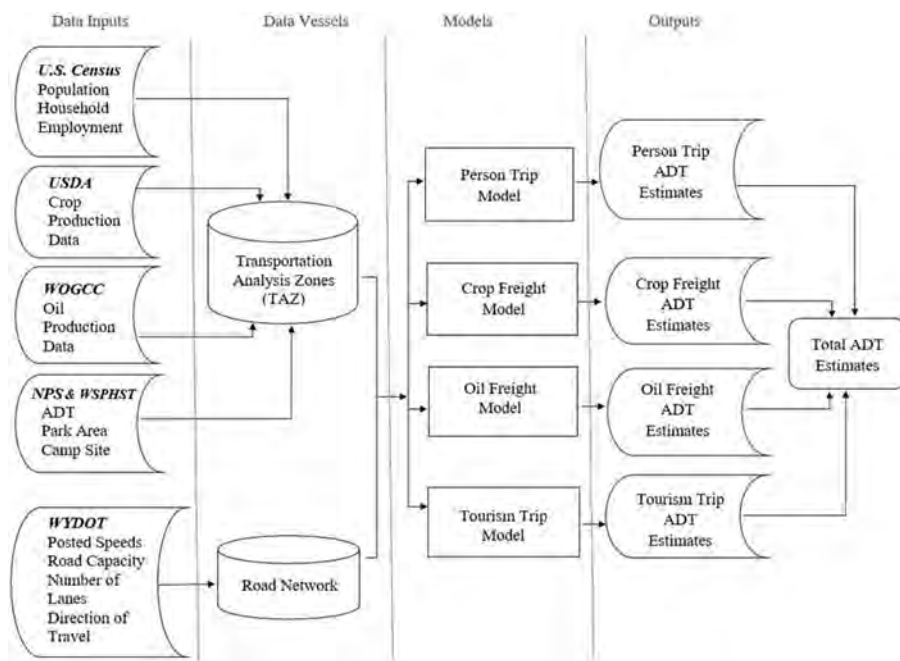


FIGURE 2 Overall model development.

TAZs. TAZs are used to create trip generation equations. The design of TAZs in this study was completed with GIS technology. Figure 3 shows the TAZs and road network in Laramie County.

Trip Generation

The trip generation procedure defines the number of total daily trips at the TAZ level. This procedure splits each trip into a production and an attraction. Initial trip production and attraction for person trips were computed from the TAZ attribute data and the trip rates obtained from the NCHRP Report 365 (24). Trip rates by purpose for each TAZ were calculated for three trip purposes – home-based work (HBW), home-based other (HBO), and non-home-based other (NHBO) trips. HBW trips are trips that have one end at home and the other at work. HBO trips are all trips with one end at home and the other end at any other place other than work. NHBO trips are those trips that start and end away from home (24). The trip generation equations for the model are presented in Table 1. A weighted average trip production per household (across all income levels) of 9.2 was selected from the household trip production rates recommended by the *NCHRP Report 365: Travel Estimation Techniques for Urban Planning* (24). NCHRP Report 365 recommended rates indicating the proportion of average daily person trips by purpose (HBW, HBO, and NHBO) of 16%, 60% and 24% for the three respective purposes. The equations for trip attractions were also derived from values recommended by the NCHRP 365 Report. An adjustment factor of 1.3 was determined for Wyoming by comparing traffic volumes estimated using the NCHRP national parameters to actual traffic volumes in Wyoming.

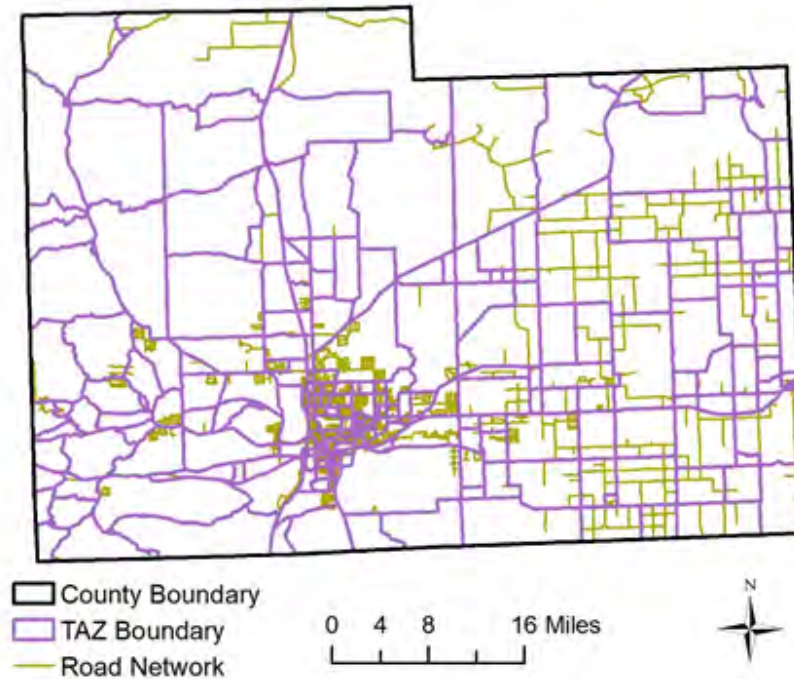


FIGURE 3 TAZs and road network in Laramie County.

The trip equations for crop production–related activities were derived using the crop yield per acre divided by the number of harvest days in a year. The result represents a factor for determining the tonnage of the crop produced in a day when applied to crop cover area in acres. This was then converted to trucks by dividing by the tonnage that can be hauled by a truck to obtain the combined factor for each crop. The combined factor for each crop is shown in the equations for crop trips indicated in Table 1. The capacities of the crop elevators serve as the attraction for crop productions.

Oil production and attractions were grouped by the products transported to or from the oil wells. The trip rate factor converts the amount in barrels per day to trucks per day. The factor is applied to the data on daily oil and waste water production, and the daily freshwater requirement data.

Three tourism-related parameters, including ADT at park entrances, park area, and number of campsites in park, were selected to determine trip rates. The selection of these parameters was based on their significance, sensitivity, and forecastability, which are recommended by the *Trip Generation Manual* (25). The constants (2, 0.2, and 0.27) before each parameter were obtained from the *Trip Generation Manual* (25). The constants of 0.22 and 0.78 indicate that 22% of total tourism trips were generated by visitors from Wyoming, and 78% of total tourism trips were generated by visitors from other states or countries. Those ratios were obtained from the NPS visitor survey (8).

External trips are trips with one or both ends outside the study area. These trips are determined for most models to enable balancing the production and attraction trips for the trip assignment step as well as accounting for trips that leave the study area from within a production zone or those that end up in an attraction zone from outside the study area. The NCHRP Report 365 provides an extensive description of how to identify external zones and calculate

TABLE 1 Trip Generation Equations

Activity	Trip Type		Equation
Person trips	Production	HBW	8.125*9.2*0.16*Households
		HBO	1.3*9.2*0.6*Households
		NHBO	1.3*9.2*0.24*Households
	Attraction	HBW	1.3*1.45*Employment
		HBO	9.0*Retail+1.7*Service+0.5*Other+ 0.9*Households
		NHBO	4.1*Retail+1.2*Service+0.5*Other+ 0.5*Households
Crop production trips	Production	Crops	0.9375*Oats+1.15*Beans+2.484*Corn+1.484* Barley+0.5*Wheat
	Attraction	Elevator	1*Elevator Capacities
Oil production trips	Production	Oil production	0.01*Oil Production
		Waste water production	0.01*Waste Water Production
		Freshwater production	0.01*Freshwater Production
	Attractions	Oil elevators	0.01*Rail Elevator
		Waste water production	0.01*Water Disposal
		Freshwater demand	0.01*Freshwater Consumption
Tourism trips	Attractions	HBO	0.22*(2*ADT at park entrance +0.2*Park area+0.27*Number of camp sites)
		NHBO	0.78*(2*ADT at park entrance +0.2*Park area+0.27*Number of camp sites)

external trips; however, the procedure is not applicable in this study. In the vast rural study area of this study, traffic on the rural low-volume roads is generated locally, and most external trips are external-external through trips that have little impact on the roads of interest to the study. Thus, external travel was deemed to have no significant impact on the local roadways and was not considered in developing the model.

Trip Distribution

In the procedure of trip distribution, the trips generated in each zone in trip generation procedure are then distributed to all other zones based on the choice of destination. This procedure creates a trip matrix that lists the number of trips going from each origin to each destination. Two main methods are usually used to distribute trips among destinations: gravity model and growth factor model. This study applied the gravity model method, derived from Newton's fundamental law of attraction, to distribute trips from each origin into distinct destinations. The gravity model is expressed as follows:

$$T_{ij} = T_i \frac{A_j f(C_{ij}) K_{ij}}{\sum_{j=1}^n A_j f(C_{ij}) K_{ij}} \quad (1)$$

where

- T_{ij} = trips from i to j ;
- T_i = trips produced in zone i ;
- A_j = trips attracted to zone j ;

$f(C_{ij})$ = friction factor related to travel time; and
 K_{ij} = optional trip distribution calibration factor.

The friction factor is expressed as follows:

$$f(C_{ij}) = a \times t_{ij}^b \times e^{c \times t_{ij}} \quad (2)$$

where

a, b, c = constants listed in Table 2 and
 t_{ij} = travel time between zone i and j .

The constants of a , b , and c were obtained from the *Trip Generation Manual (25)*. Travel time was calculated by dividing total distance between zone i and j by posted speed limit. For unpaved roads, a speed of 10 mph was assigned in the model.

Mode Choice

The procedure of mode choice determines the means of travelling. Only the mode of personal vehicle was considered for personal trips and tourism trips since other travel modes, such as public transportation, are not significantly represented in the study area. For crop production freight trips and oil production freight trips, only the mode of truck was considered. In this procedure, average vehicle occupancies were used to create the trip matrix of personal and tourism trips. Table 3 lists auto-occupancy rates for personal and tourism trips.

TABLE 2 Coefficients Used in Equation 2

Activity	Trip Type	a	b	c
Person trips	HBW	28,507	0.02	0.123
	HBO	139,173	1.285	0.094
	NHBO	219,113	1.332	0.100
Tourism trips	HBO	24,108	0.032	0.158
	NHBO	26,544	0.156	0.188

TABLE 3 Auto-Occupancy Rates

Activity	Trip Type	Auto-Occupancy Rates (Persons/Vehicles)
Person trips	HBW	1.11
	HBO	1.59
	NHBO	1.43
Tourism trips	HBO and NHBO	3.04

Trip Assignment

The trip assignment procedure allocates a given set of trip interchanges to the specified transportation network. In this procedure, the traffic volumes on each road segment will be estimated, and the travel patterns will be analyzed. This procedure calculates the shortest path from each zone to all the other zones. The assigned number of trips are compared to the capacity of the road to see if it is congested. If a road is congested, the trip routes are changed to result in a longer travel time on that road. The whole process is repeated several times until there is an equilibrium between travel demand and travel supply.

RESULTS

Model Outputs

Figure 4 shows the spatial distribution of low-volume roads ($ADT \leq 400$) and non-low-volume roads within the study area. Only low-volume roads were future analyzed. Figure 5 shows traffic volumes on low-volume roads. Most low-volume roads in Wyoming have an ADT smaller than 200. Some low-volume roads near metropolitan areas and tourism destinations have relatively high ADT.

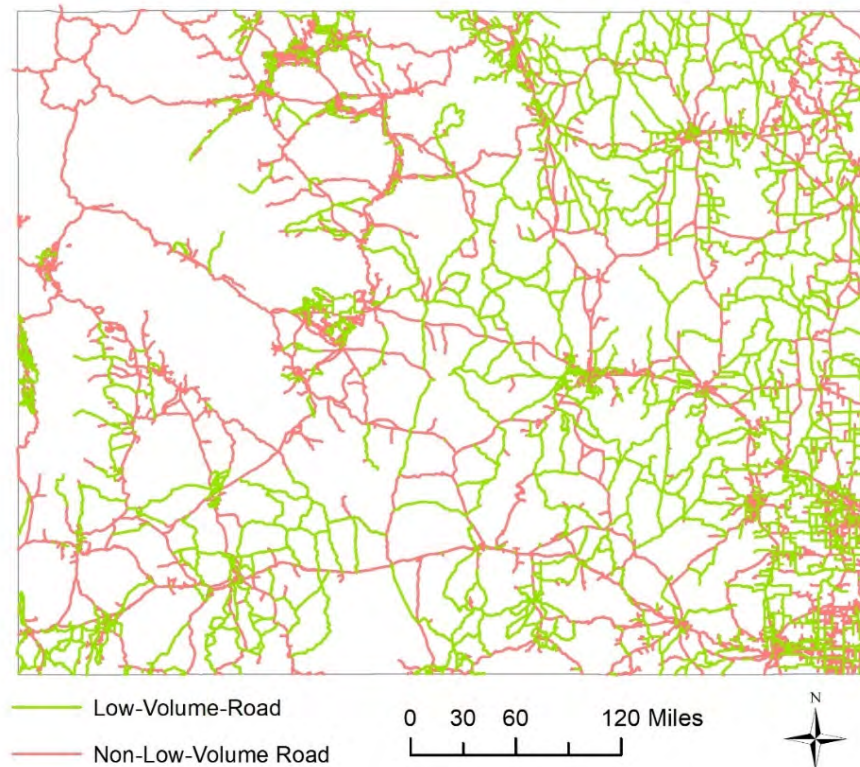


FIGURE 4 Low-volume roads versus non-low-volume roads.

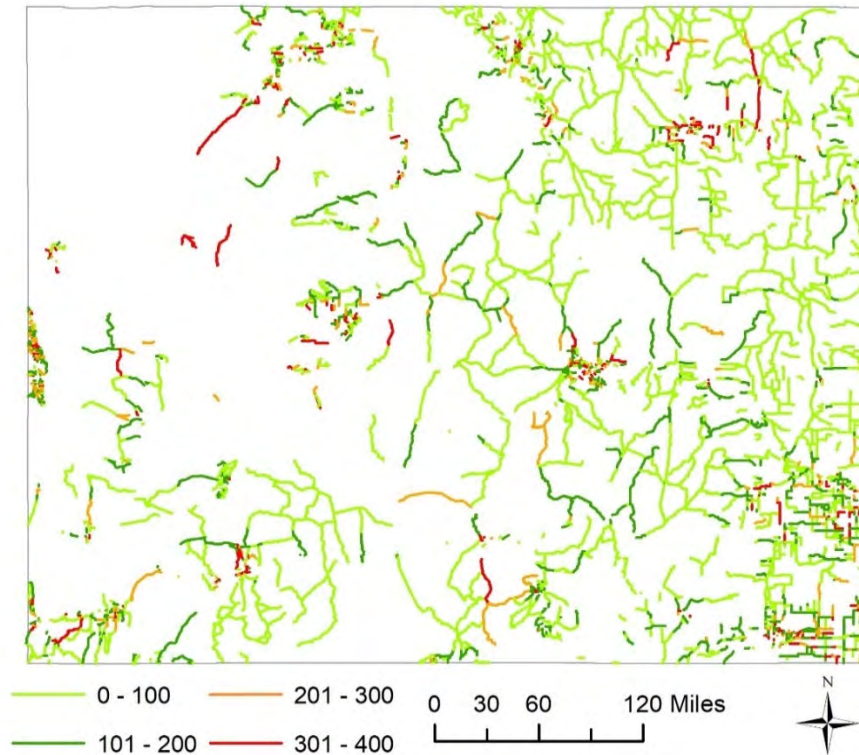


FIGURE 5 ADT for low-volume roads.

Figure 6 presents ADT from four trip types included in the model. It can be seen that person trips and tourism trips are two dominant trip types on low-volume roads in Wyoming. Crop freight-related ADT values are high in Southeast Wyoming and Northwest Wyoming. According to the USDA Annual Statistical Bulletin (26), Southeast Wyoming is the main area for cultivating corns and wheat, and Northwest Wyoming is the main area for cultivating barley and beans. Almost all low-volume roads in Wyoming have an ADT smaller than 50 from oil freight-related trips. Tourism-related ADT are higher near Yellowstone National Park, Devils Tower National Monument, and some popular state parks in Southeast Wyoming.

Model Validation

Once the four steps are completed, the model provides planners with a picture of existing travel patterns. The results are then given a reality check to make sure the traffic volume estimations make sense. In this study, traffic volume data generated by the model were compiled and compared to actual traffic volumes on 542 selected roads in Wyoming. Figure 7 shows the spatial distribution of actual traffic counters and their ADT values. The majority of the actual traffic counts were obtained during the summer of 2014. The duration of actual traffic counts ranged from 5 to 10 days. The actual traffic counts were then converted to the ADT values for validation purposes.

An R^2 value that indicates the percentage of the variation in the actual traffic volumes that is explained by the model was used to validate the model. The overall R^2 of the model is

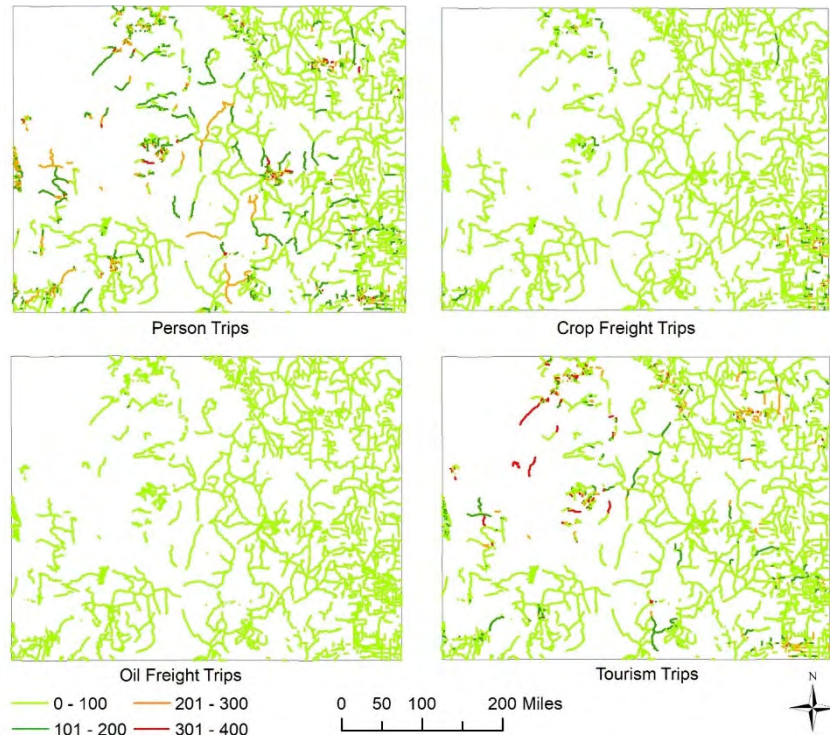


FIGURE 6 ADT from different trip types.

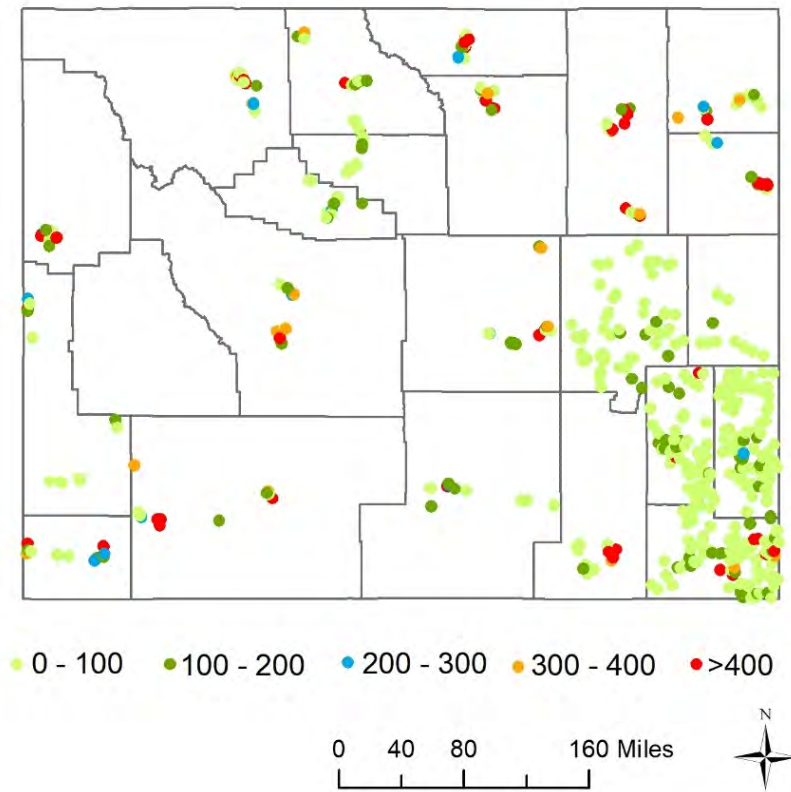


FIGURE 7 Actual ADT values for model validation.

0.86, which indicates that the model is capable to predict traffic volumes on low-volume roads. Table 4 listed R^2 values for each region. The TDM performed best in Region 2 and did not perform as well as other regions in Region 1, which consisted of many actual traffic counts on unpaved roads.

Although the TDM well predicted ADT, there are some factors that should be considered in using the model and its results:

1. The model estimates are not applicable to urban roads including low-volume roads that leads directly to or within an urban area.
2. The model estimates are not applicable to interstates and state highways.
3. The model estimates are for the higher summer traffic volumes only, since park visitation data from peak season (May to October) was input into the model.

CONCLUSIONS

Traffic volume estimation is essential to a reliable transportation planning program. Not all roads can be installed traffic counters to record traffic volumes. As a result, it may be desirable and useful to establish a method to determine traffic volumes on low-volume roads. A four-step TDM for estimating ADT on low-volume roads was developed in this study by using a variety of variables. Four types of trips, person trips, crop production freight trips, oil production freight trips, and tourism trips were evaluated to make use of available data and to provide reliable traffic volume estimates. Detailed TAZs were developed to enable traffic distribution to smaller local roads. Urban trip rates published in the NCHRP Report 365 were utilized in estimating the person trip productions and attractions from population and employment data. For freight trips related to agricultural crop production activities, crop cover data were obtained from the USDA to compute the tons of crops produced in each TAZ with crop elevators serving as the attractions for the crops produced. For freight trips associated with oil production activities, data on location of oil wells, freshwater wells, injection wells, and oil elevators were used to estimate trips production and attraction at each TAZ. For tourism trips, ADT at park entrances, park area, and number of campsites in each park were used to estimate tourism-related trips. The combined trips from all four activities were distributed among the TAZs and assigned to the routes between pairs of TAZs to generate traffic volume estimates on the road network.

This study shows the significance of predicting traffic volumes on low-volume roads for transportation planning and maintenance. The model can be easily incorporated into the existing statewide TDM. This study also adds to the existing knowledge on the estimation of traffic volumes by TDM in rural areas. Previous studies mainly focused on estimating traffic volumes

TABLE 4 Model Validation Results.

Region	R^2	Number of Actual Traffic Counters	
		On Paved Roads	On Unpaved Roads
1	0.81	57	270
2	0.93	36	35
3	0.88	43	33
4	0.84	42	26

in urban areas and Interstate highways. The model developed in this study can be used to estimate ADT in the rural areas where not enough traffic counters are installed. The model is recommended for upgrade after each census by the U.S. Census Bureau. Updating the model every ten years will ensure that the model is not using outdated census data and its predictions match closely with the actual traffic volumes on the low-volume roads in Wyoming. Spikes in oil and gas activities as well as tourism activities may also require model updates. The model developed in this study is recommended to be applied by government and tourism agencies in other states or regions when certain types of trips are known as major generators of traffic flow on low-volume roads.

ACKNOWLEDGMENTS

The Wyoming DOT and the Mountain-Plains Consortium are acknowledged for funding this study. The Wyoming Technology Transfer Center is also acknowledged for facilitating this study by making available resources and providing relevant training. The authors acknowledge that this work is part of a project funded by the Wyoming DOT. The subject matter, all figures, tables, and equations not previously copyrighted by outside sources are copyrighted by Wyoming DOT, the State of Wyoming, and the University of Wyoming. All rights reserved copy righted in 2016.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: Study Conception and Design: Er Yue and Khaled Ksaibati; Data Collection: Er Yue and Khaled Ksaibati; Model Development: Er Yue and Khaled Ksaibati; Analysis and Interpretation of Results: Er Yue and Khaled Ksaibati; Draft Manuscript Preparation: Er Yue and Khaled Ksaibati. All authors reviewed the results and approved the final version of the manuscript.

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Estimating Annual Average Daily Traffic for Low-Volume Roadways *A Case Study in Louisiana*

XIAODUAN SUN

University of Louisiana at Lafayette

SUBASISH DAS

Texas A&M Transportation Institute

Annual average daily traffic (AADT) is a critical input to many key components of transportation activities. Accurate AADT data are vital to the calibration and validation of travel demand models, roadway improvement funding allocations, and safety performance evaluations. Lack of AADT information on low-volume roads (LVR) is a common problem facing public transportation agencies everywhere in the United States. Traffic volumes on these roads are generally fairly low, and vehicle miles traveled on these roads are much less compared with that on other roadways. Thus, regularly conducting traffic counts is not economically feasible for LVR. This paper introduces an AADT estimation methodology with the statistical and pattern recognition methods. By using available traffic counts on selected roadways and four variables (namely population, job, and distance to intersection and to major state highways at block level), a training set to estimate roadway AADT for eight parishes was obtained by the support vector regression method. This pattern recognition method yields reasonable AADT estimates for LVRs. Sensitivity analyses indicates that the parish-specific (county-specific) model works better than an aggregated single model.

INTRODUCTION

Annual average daily traffic (AADT) is the average 24-h traffic volume at a roadway segment over an entire year. AADT is a key input to many activities of the state departments of transportation (DOTs), such as roadway planning, design, traffic operation, pavement maintenance, air quality assessment, revenues from the roadway user fees, and roadway safety evaluations. Accurate AADT data are vital to the calibration and validation of travel demand models. AADT is also used to estimate statewide vehicle miles traveled on all of the roadways and is used by governments and the environmental protection agencies to determine compliance with the Clean Air Act Amendment.

The DOTs and local transportation agencies traditionally use AADT count programs to collect traffic information. The focus of these traffic count programs is on higher classes of roadways, which consists mainly of interstates and arterials. Due to the budgetary and administrative constraints, regular traffic counting is only conducted on state highways network (generally every 3 years). The traffic counting for LVRs is highly selective and irregular. Most of the major cities in Louisiana, such as New Orleans, Baton Rouge, Lafayette, and Lake Charles, have the traffic volume data collected or estimated by their respective metropolitan planning

organizations. Other nonstate roadways in rural and small urban areas do not have an AADT collection or estimation programs. Since approximately three-fourths of all roadway mileage in Louisiana is nonstate maintained by local governments (parishes, i.e., counties and municipalities), estimating AADT on nonstate roadways is important. With increased emphasis on nonstate roadway safety and publication of the *Highway Safety Manual* (HSM), AADT is becoming a “must-have” element in roadway safety evaluation. Lack of AADT information hinders roadway safety assessment and makes it hard to develop cost-effective safety improvement projects.

LITERATURE REVIEW

Due to the strategic importance of AADT, many state DOTs have established methods to estimate AADT for roadways that do not have regular traffic counts. The most noteworthy state practices on the AADT estimation are summarized in Table 1.

The literature review on the state of the modeling techniques reveals that two methods are widely used in estimating AADT: statistical model and pattern recognition. Statistical models were developed through the use of linear regression, parcel-level trip generation, and spatial grids, with the latter two being more related to this study. Although statistical models are

TABLE 1 Summary of Most Noteworthy State Practices

State	Analysis Unit	Scope	Method
Alaska	Densities of built tax parcels	Local urban roadways	Geographic information system-based linear regression model and parcel data analysis
Florida	Block-level data and densities of built tax parcels	Urban and rural locations with a permanent traffic monitoring site	Linear regression analyses to identify possible factors contributing to the seasonal fluctuations in traffic volumes
Kentucky	By functionally classified collector roads and local roads, minimizing the level of effort required to estimate traffic volumes on local roads	Collector roads and local roads	Random sampling procedure
Pennsylvania	Stratification of locally owned roads for traffic data collection	Local roads owned by municipalities	Sampling method
Texas	Selected traffic count sites randomly on local streets, resulting in a statistically valid estimation of local street vehicle miles traveled	Local roads	Random count site selection
New York	Sample-based count program representing geographic distribution, functional classification, and volume group	Locally owned, non-Federal-aid highways	Sampling method

relatively easier to interpret, these models can only be used at an aggregated level. Machine learning or pattern recognition approaches like artificial neural network, decision trees, clustering, support vector machines (SVM), and fuzzy algorithms are also widely used in estimating AADT. A summary of the state of the art of the methods of AADT estimation is listed in Table 2.

To the authors' knowledge, there is little practical application in AADT estimation for LVRs. The current study aims to mitigate the current research gap.

METHODOLOGY

This study introduces the Support Vector Regression (SVR) technique to estimate AADT in nonstate roadways. The majority of LVRs are nonstate roadways. The Louisiana Department of Transportation and Development (LADOTD) maintains traffic count in few count stations for nonstate roadways throughout the state. The available traffic count from the count station is used

TABLE 2 Summary of Important Research on AADT Estimation of Local Roadways

Division	Methods	Advantages	Disadvantages	Reference
Parametric Statistical methods	Regression method	Easy to group, highly aggregated, and easy in application	Ignoring the difference among roadways in the same classification group	2–5
	Travel demand modeling method at tax parcel level	Update of data at least annually which makes possible to update the AADTs in response to lane use changes	Need number of parcels to be preprocessed to improve the model's efficiency	6
	Stratification method	Better results than the traditional sampling methods	Not useful when the population cannot be exhaustively partitioned into disjoint subgroups	7
	Kriging methods	Less error and statistically significant	Ignoring the difference within each roadway classification	8
Pattern Recognition Methods	Clustering and regression trees	Analyzing AADT for roadways with similar characteristics	No specific models	9, 11
	Support vector regression with data dependent parameters	Highly aggregated at county by rural and urban with good results	Ignoring the difference inside a large geographic area or roadway class	10
	Fuzzy theory and neural network	Consideration of uncertainty	Pre-defined aggregated roadway group	12

as parish-specific training data. This method predicts parish-specific U.S. Census block-level AADT based on the distribution of the training data and performs better than conventional statistical methods. Eight mostly rural parishes (a county is called a parish in Louisiana) are selected for this study based on their population, accessibility to interstate and U.S. highways, and the number of traffic count stations as shown in Figure 1.

A preliminary data exploration was first conducted to examine the important factors that may contribute to AADT. All potential variables were obtained from several sources: (1) LADOTD-collected AADT data on nonstate roadways, (2) U.S. Census block-level data (population and household), (3) employment data from Longitudinal Employer-Household Dynamics, and (4) the shortest distance measurement based on the ArcGIS files (13). The database was developed based on the process illustrated in Figure 2.

An SVM is a learning machine that executes the structural risk minimization inductive principle to attain good generalization on a limited number of learning patterns. Vapnik et al. originally developed this theory on a basis of a separable bipartition problem at the AT&T Bell Laboratories in 1992. The basic idea of SVM is to map the data x into a high-dimensional feature space F via a nonlinear mapping and to perform linear regression in this space. The support vector algorithm can also be applied to regression, maintaining all the main features that characterize the maximal margin algorithm: a nonlinear function is learned by a linear learning machine in a kernel-induced feature space, while the capacity of the system is controlled by a parameter that does not depend on the dimensionality of the space. An overview of the basic conception underlying SVR and function estimation has been given in two papers (14, 15).

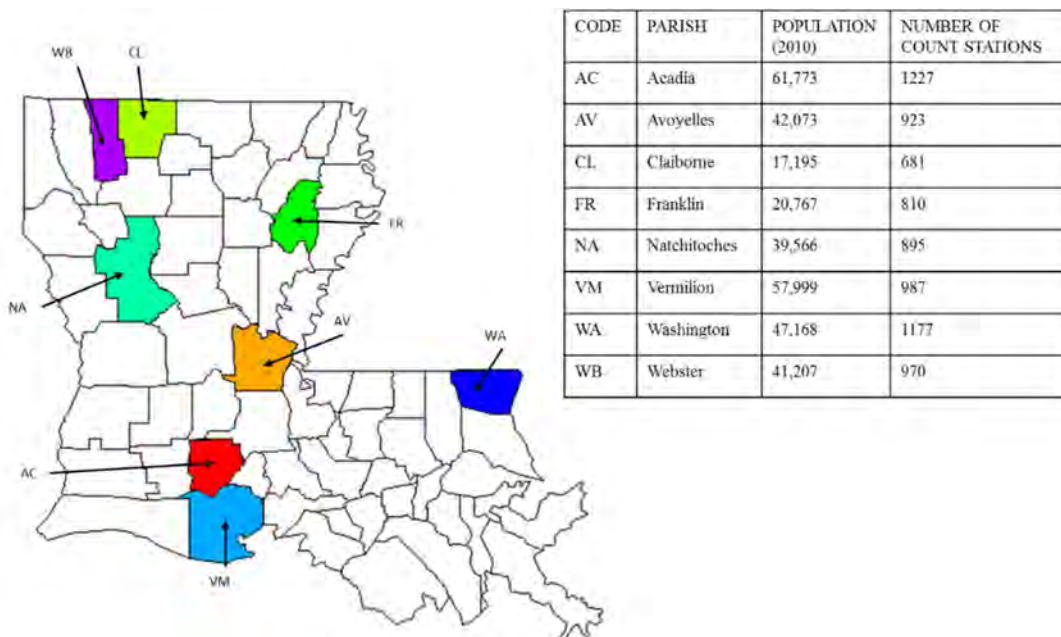


FIGURE 1 Eight selected parishes in Louisiana.

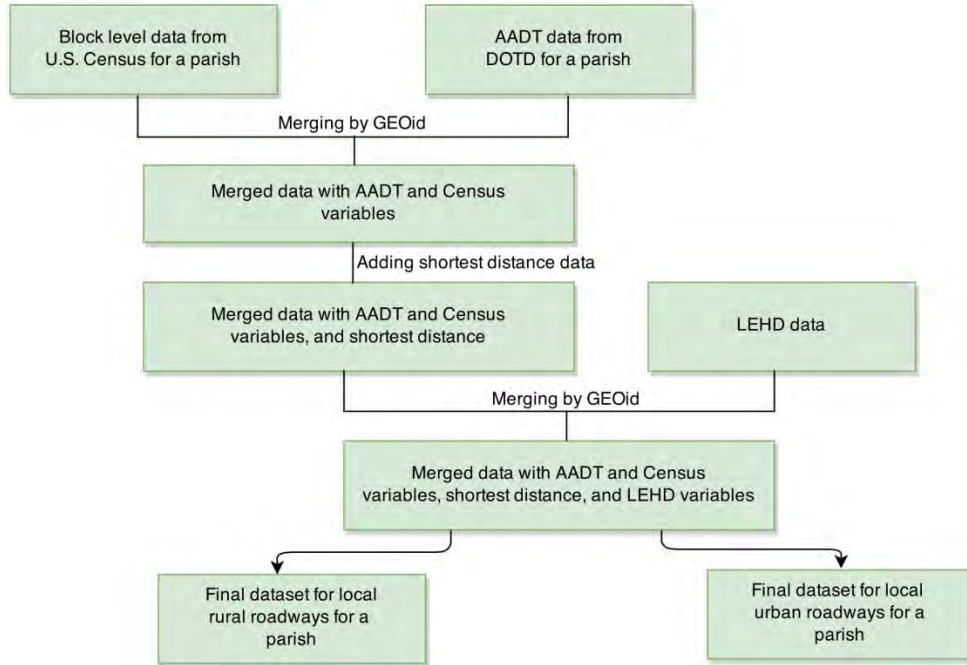


FIGURE 2 Data development process.

SVR is one of the most common application forms of SVMs. First, consider a training dataset $\{(x_1, y_1), \dots, (x_n, y_n) \in \mathfrak{X} \times \mathfrak{R}\}$, where \mathfrak{X} denotes the space of the input patterns (e.g., $\mathfrak{X} = \mathfrak{R}^d$). In ε -SV regression, the target is usually to find a function $f(x)$ that has at most ε deviation from the actually obtained targets y_i for all of the training dataset. The other target is to make it as flat as possible. So, errors less than ε are acceptable, but no deviations larger than this. The linear function $f(x)$ can be described as follows:

$$f(x) = \langle w, x \rangle + b \quad \text{with } w \in \mathfrak{X}, b \in \mathfrak{R} \quad (1)$$

where $\langle \dots \rangle$ denotes the dot product in \mathfrak{X} . Flatness in Equation 1 means smaller w . To obtain this we need to minimize the Euclidean norm $\|w\|^2$. Formally, this can be considered as a convex optimization problem by fulfilling the condition

$$\begin{aligned} &\text{minimize } \frac{1}{2} \|w\|^2 \\ &\text{subject to } y_i - \langle w, x_i \rangle - b \leq \varepsilon \quad \text{and} \quad \langle w, x_i \rangle + b - y_i \leq \varepsilon \end{aligned} \quad (2)$$

The convex optimization in Equation 2 is feasible in cases where f actually exists and approximates all pairs (x_i, y_i) with ε precision. At times, some errors are usually allowed. There is a need to introduce slack variables ξ_i, ξ_i^* to handle otherwise infeasible constraints of the optimization problem in Equation 2, the formulation will be

$$\begin{aligned}
& \text{minimize } \frac{1}{2} \|\omega\|^2 + C \sum_{i=1}^n (\xi_i + \xi_i^*) \\
& \text{subject to } \begin{cases} y_i - \langle w, x_i \rangle - b \leq \varepsilon + \xi_i \\ \langle w, x_i \rangle + b - y_i \leq \varepsilon + \xi_i^* \\ \xi_i, \xi_i^* \geq 0 \end{cases} \quad (3)
\end{aligned}$$

The constant $C > 0$ defines the trade-off between the flatness of f and tolerance of deviations larger than ε . The ε -intensive loss function $|\xi|_\varepsilon$ can be described as

$$|\xi|_\varepsilon = \begin{cases} 0 & \text{if } |\xi| < \varepsilon \\ |\xi| - \varepsilon & \text{otherwise} \end{cases} \quad (4)$$

The dual formulation provides the key for extending SVM to nonlinear functions. The standard dualization method using Lagrange multipliers can be equated as follows:

$$\begin{aligned}
L = & \frac{1}{2} \|\omega\|^2 + C \sum_{i=1}^n (\xi_i + \xi_i^*) - \sum_{i=1}^n (\alpha_i + \xi_i - y_i + \langle \omega, x_i \rangle + b) - \sum_{i=1}^n \alpha_i^* (\varepsilon + \xi_i^* + y_i - \langle \omega, x_i \rangle - b) - \\
& - \sum_{i=1}^n (n_i \xi_i + n_i^* \xi_i^*) \quad (5)
\end{aligned}$$

The dual variables in Equation 5 is needed to satisfy positivity constraints, i.e., $\alpha_i, \alpha_i^*, \eta_i, \eta_i^* \geq 0$. It follows from saddle point condition that the partial derivatives of L with respect to the primal variables $(\omega, b, \xi_i, \xi_i^*)$ have to vanish for optimality condition.

$$\frac{\partial N}{\partial b} = \sum_{i=1}^n (\alpha_i^* - \alpha_i) = 0 \quad (6)$$

$$\frac{\partial N}{\partial \omega} = \omega - \sum_{i=1}^n (\alpha_i^* - \alpha_i) x_i = 0 \quad (7)$$

$$\frac{\partial N}{\partial \xi_i^*} = C - \alpha_i^* - \eta_i^* = 0 \quad (8)$$

The dual optimization problem after using Equations 6, 7, and 8, by maximizing

$$-\frac{1}{2} \sum_{i,j=1}^n (\alpha_i - \alpha_i^*) (\alpha_j - \alpha_j^*) \langle x_i, x_j \rangle - \varepsilon \sum_{i=1}^n (\alpha_i + \alpha_i^*) + \sum_{i=1}^n y_i (\alpha_i - \alpha_i^*) \quad (9)$$

The Equation 9 subjects to $\sum_{i=1}^n (\alpha_i - \alpha_i^*) = 0$, and $\alpha_i, \alpha_i^* \in [0, C]$. Dual variables η_i, η_i^* through condition 8 have been eliminated for deriving 9. Equation 7 can be rewritten as follows:

$$\omega = \sum_{i=1}^n (\alpha_i^* - \alpha_i) x_i \Rightarrow f(x) = \sum_{i=1}^n (\alpha_i - \alpha_i^*) \langle x_i, x \rangle + b \quad (10)$$

This is known as the support vector expansion, i.e., ω can be completely described as a linear combination of the training patterns x_i . The standard SVR to solve approximation problem can be written as:

$$f(x) = \sum_{i=1}^N (\alpha_i^* - \alpha_i) k(x_i, x) + b \quad (11)$$

where, α_i , and α_i^* are Lagrange multipliers. After the initial data exploration, the following variables were selected for the model development:

- Total population: the total population of a census block with traffic count;
- Total jobs: the number of jobs in a census block with traffic count;
- Distance from Interstate: the shortest distance (in miles) between the count location and interstate access point (on-ramp merge with mainlines); and
- Distance from a major (U.S.) highways: the shortest distance (in miles) between the traffic count location and the intersection with a major highway.

Because of the variations in each selected variable and in the characteristics of the parish (e.g., demographic and interstate access), parish-specific models and integrated parish models were developed. The integrated parish models are for all parishes with and without direct interstate access.

The open source R software package “e1071” was used to develop the AADT estimation models with the SVR techniques (17). The “e1071 library” contains implementations for a number of statistical learning methods. A cost argument allows specifying the cost of a violation to the margin. When the cost argument is small, the margins will be wide, and many support vectors will be on the margin or will violate the margin. When the cost argument is large, the margins will be narrow, and there will be few support vectors on the margin or violating the margin. To fit an SVM with a radial kernel, kernel = “radial” was used. To specify a value for the radial basis kernel, gamma is used. In the AADT estimation, the following parameters were used (after performing several trial-and-error runs to get the best prediction):

- SVM-Type = eps-regression,
- SVM-Kernel = radial in this study,
- Cost = 100,
- Gamma = 1, and
- Epsilon = 0.1.

SVR can enhance prediction accuracy and provides an efficient way to compute parameters. The quality and performance of the SVR models depend on the setting of three parameters: kernel type, value of the penalty for excess deviation during training (C), and error-term value for the ϵ -insensitive loss function (ϵ). In addition, the number of support vectors is determined before running the SVR analysis. Once all parameters are determined, R is used to

run the SVR analysis. Also, the values can be graphically summarized to better analyze the results since the initial estimated values are not shown in the script window in R.

RESULTS

Figure 3 and Figure 4 show the estimated AADT versus traffic counts for eight parishes. The points on the diagonal line indicate a perfect match between the predicted and observed. Two lines placed on each side of the diagonal line represent the difference between the predicted and observed in the 100 and 200 ranges. Compared to the models that were explored initially, the SVR model yields much better results. Neither Poisson nor Negative Binomial models tried with the same data produced more than 30% of match with the observed AADT.

It is clear that the SVR model tends to underestimate the AADT at higher observed traffic count values and somewhat overestimate the AADT at lower traffic count values. However the SVR model does capture the majority of traffic counts as shown in Table 3. Within the ± 100 bandwidth, the coverage runs from the minimum 64% to the highest 82%. Within the ± 200 bandwidth, the coverage runs from the lowest 78% to the highest 91%. This close match provides great hope for getting AADT for the intended applications in transportation planning and traffic management, as well as in roadway safety evaluation with the HSM considering AADT is hard to come by on LVRs. All other models explored can only research the 30%–40% match. No previous studies on the AADT estimation revealed such results at the disaggregated level. Again, the majority of the previous studies estimated AADT at a much aggregated level, such as by roadway functional classification.

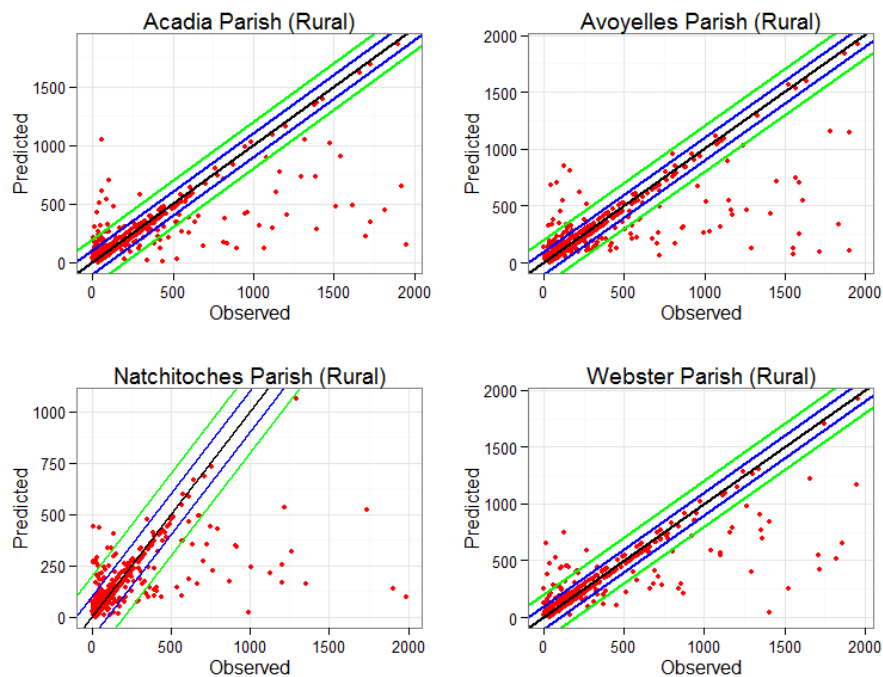


FIGURE 3 Predicted AADT versus observed AADT for parishes with direct interstate access in rural areas.

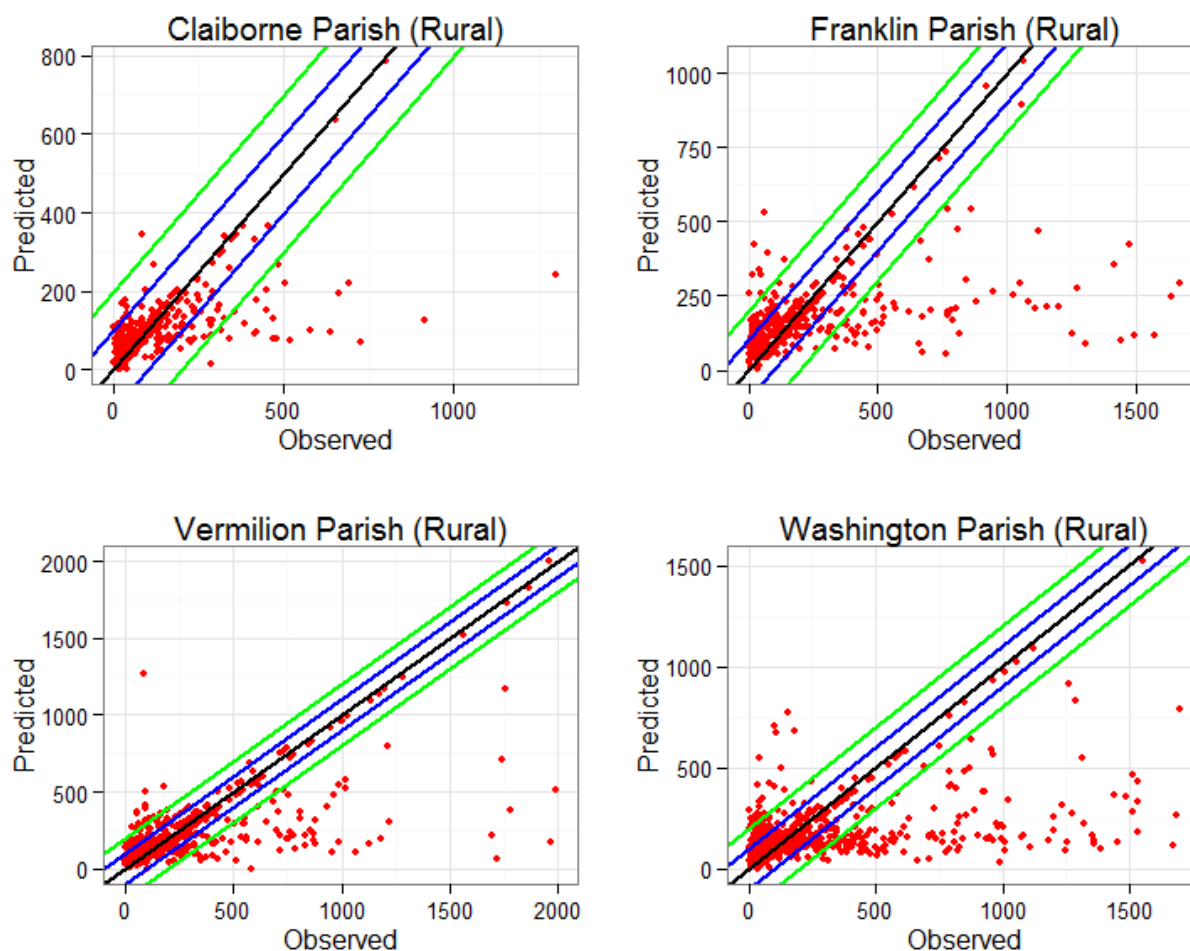


FIGURE 4 Predicted AADT versus observed AADT for parishes without direct interstate access in rural areas.

TABLE 3 Results Evaluation for Nonstate Rural Highways

Parish	Presence of Interstate Roadways	Sample Size	Support Vectors	Within ± 100 vpd Boundary		Within ± 200 vpd Boundary	
				Count	Percent	Count	Percent
Acadia	Yes	464	419	380	82%	413	89%
Avoyelles	Yes	481	422	391	81%	425	88%
Natchitoches	Yes	453	378	373	82%	406	90%
Webster	Yes	380	344	310	82%	333	88%
Claiborne	No	335	295	283	84%	310	93%
Franklin	No	431	376	304	71%	357	83%
Vermilion	No	447	401	298	67%	368	82%
Washington	No	740	634	477	64%	581	79%

Figure 5 and Figure 6 show the estimated AADT versus traffic counts for LVRs in small urban areas. Similarly, the points on the diagonal line indicate a perfect match between the predicted and observed. Two lines placed on each side of the diagonal line represent the difference between the predicted and observed in the 100 and 200 ranges. Compared to the models explored initially such as the Poisson and Negative Binomial models, the SVR model yields better results. However, comparing with the results for the rural areas, the predicted results for the small urban areas are as good as the predicted AADT for the rural roadways. As shown in Table 4, the percentage match between the observed and predicted is given. Within the ± 100 bandwidth, the coverage runs from the minimum 63% to the highest 100%. Within the ± 200 bandwidth, the coverage runs from the lowest 74% to the highest 100%.

CONCLUSIONS

This study applied SVR to estimate traffic volumes on LVRs in rural and small urban areas in eight Louisiana parishes. The findings show that SVR performs better than Poisson and Negative Binomial models in AADT estimation in rural and small urban areas. The estimated AADT are sufficiently accurate for transportation planning and roadway safety evaluation purposes. The developed method tends to underestimate AADT for roadways observed with traffic count higher than 1,500 per day, which is not a concern of LVRs. There are significant differences in the estimated AADT among the parishes, thus parish-specific models should be developed. AADT estimation by nature is highly stochastic. AADT estimation for nonstate roadways in small urban areas also yield satisfactory results.

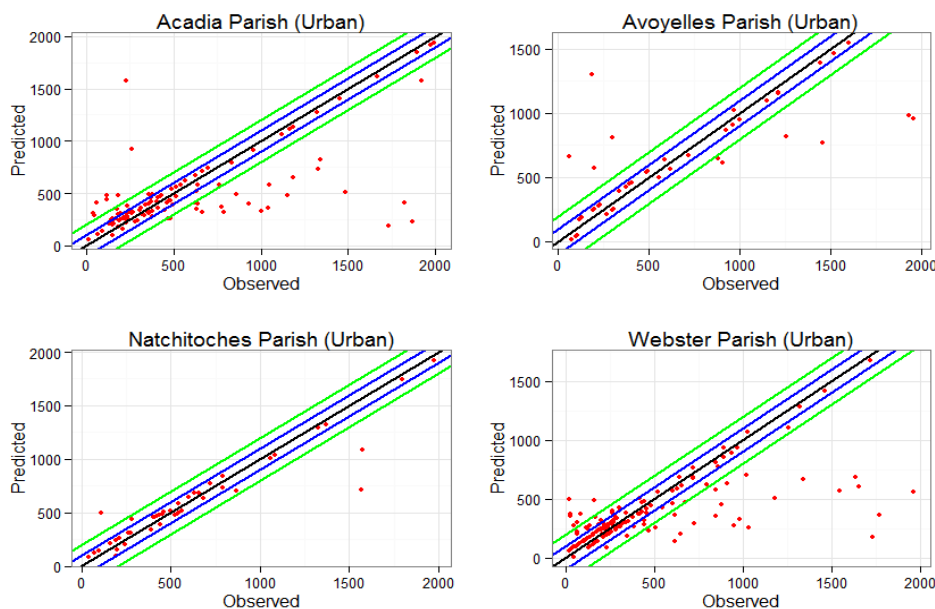


FIGURE 5 Predicted AADT versus observed AADT for parishes with direct interstate access in small urban areas.

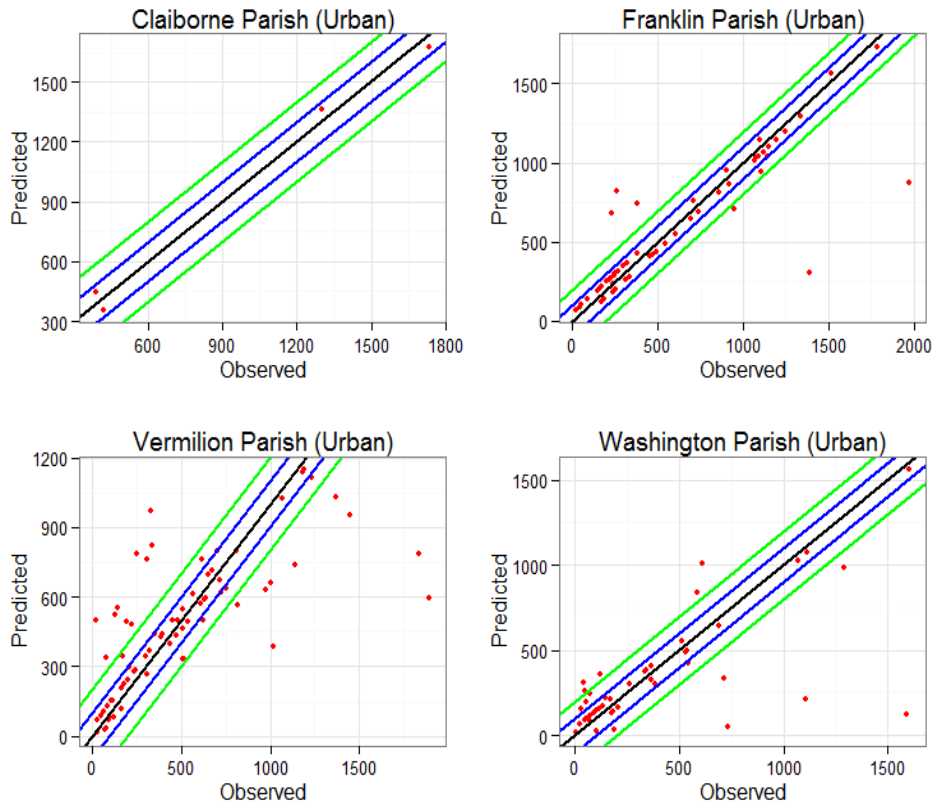


FIGURE 6 Predicted AADT versus observed AADT for parishes without direct interstate access in small urban areas.

TABLE 4 Results Evaluation for Nonstate Roadways in Small Urban Areas

Parish	Presence of Interstate Roadways	Sample Size	Support Vectors	Within ± 100 vpd Boundary		Within ± 200 vpd Boundary	
				Count	Percent	Count	Percent
Acadia	Yes	114	104	72	63 %	84	74%
Avoyelles	Yes	46	46	36	78%	36	78%
Natchitoches	Yes	44	38	39	89%	41	93 %
Webster	Yes	167	153	118	71%	137	82%
Claiborne	No	5	5	5	100 %	5	100%
Franklin	No	55	54	47	85%	49	89%
Vermilion	No	73	69	46	63%	54	74%
Washington	No	49	47	34	69%	39	80%

The method is not without limitations. One of the major drawbacks of the SVR modeling technique is the absence of a structural equation for easy interpretation. An association between response variable and predictors is hard to explain in machine learning models like SVRs. New techniques like partial dependence plots (PDP) can be used in interpreting the results. Future studies can focus on the application of PDPs in SVR interpretation.

ACKNOWLEDGMENT

This study was supported by the Louisiana Transportation Research Center.

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Evidence-Based Investment Planning for Unsealed Roads

THEUNS F. P. HENNING

KATE ROBINSON

ANDREW THOM

The University of Auckland

JULIE MUIR

ANDY BARTLETT

Central Otago District Council. New Zealand

INTRODUCTION

Unsealed roads account to a significant portion of road networks 40% to 60% for developed countries, where developing countries often have unsealed roads in excess of 80%. Sealing these networks is not always economically warranted, nor is it necessary as it remains a sustainable surfacing option for low volume roads (1).

The planning and maintenance regimes of these roads vary vastly depending on the prevailing climatic conditions, geology and institutional resourcing availability. For developing countries with constraint funding, the focus is mainly on providing nearly all-weather access, while maintenance investment is prioritized on a socioeconomic basis. Therefore, roads that could stimulate the economy, such as farming connector roads will have higher priority than local residential access roads (2). With stronger economies the focus shift towards optimizing the resourcing on the basis of the whole-of-life agency costs and sometimes road user cost–benefit analyses (3). In developed economies, there is more emphasis on customer satisfaction, which creates tension between proving a good level-of-service, while preserving resources at a sustainable level (4). The challenge under these circumstances is to have sufficient evidence that the investment is at appropriate levels. This paper proposes a framework that uses customer feedback, material properties, and a thorough understanding of maintenance achievement in order to manage unsealed roads in a sustainable manner. It is recognized that the management of unsealed roads considers a number of factors such as traffic volume, material characteristics, economic returns, and environmental considerations. The scope of this paper only considers the interaction of user satisfaction and maintenance effectiveness. Although targeting developed economies and adequately maintained unsealed road networks, the application is of value for most other networks.

RESEARCH METHODOLOGY

The objectives of the research were to:

- Understand the relationship between customer complaints and maintenance frequency;

- Quantify the exact impact of maintenance activities, in particular, grading activities;
- and
- Establish the overall road performance, given specific material properties.

The research was undertaken in two geographical locations in New Zealand. Network level analyses and customer complaints were undertaken in the Central Otago District Council (CODC), while specific maintenance analyses were undertaken as part of unsealed road trials in Kaipara District Council. The data consisted of

- Customer feedback sourced from the customer complaint help line, maintenance records, and material properties on the CODC data; and
- Maintenance records, material properties, and roughness data recoding using cellphone technology. (Note, this authority does not collect a visual assessment of roads due to the rapid change in road conditions due to changing weather patterns.)

FINDINGS

Analyses outcomes from the analyses on the customer complaint data have revealed the following trends on the total number of complaints.

- Road users often complain when something changes from what they are used to. These changes may be the frequency of grading, the type of grading, the grader operator, or the condition of the road.
- There was a weak correlation to a median roughness level and complaints, while there was a stronger relationship to peak roughness and number of the complaints, suggesting people will often complain about isolated rough areas on the road.
- Road users mostly complain about the occurrence of roughness and potholes.

A further analysis was also completed on the timing of complaints following grading activities. Five-year historical data was used to plot how long it took (number of days) before road users complain about condition following a graded activity (Figure 1). For these analyses, only the top five roads attracting the highest rate of complaints were considered. The time to complaints was also related back to the quality of the material as classified in the Paige-Green Charts (5). On the figure, the outlier (Gimmerburn Waiapata Road) was excluded from the analyses as this road is only a back-country track with very low traffic volume (annual average daily traffic <20). Observations from the figure include the following:

- There is a high percentage of complaints (25th percentile) that are instantaneously following grading activities for most roads.
- Higher-quality material roads (with ideal material properties indicated by the two blue bars) show a high distribution suggesting that most users will take longer to complain about roughness on these roads. This suggests that higher-quality material roads perform better in the long run, thus taking longer for people to complain.

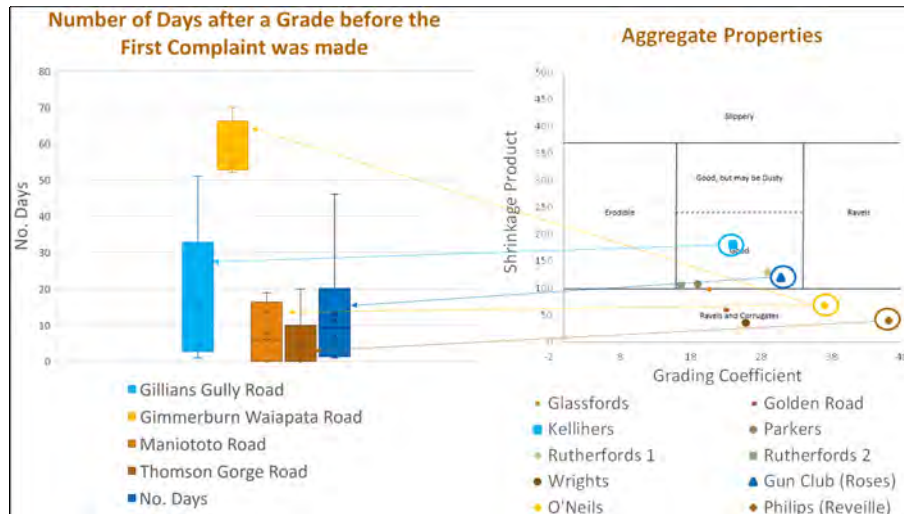


FIGURE 1 Understanding when users complain.

The second part of the research was to investigate the specific impact of grading operations across three trial sections. The outcome from one of the sections is illustrated in (Figure 2). It shows the roughness changes over time as a function of rainfall, maintenance and two types of grading operations: a standard grade (blue dots) and a wet-roll-and-grade (red dots).

- Some grading activities may result in a higher roughness level directly following the activity (see September 2016 to October 2016). This is often the case with shape correction or “deep grades” with the intention of significantly mixing the pavement material. This result may explain some of the observation on networks where road users complain about roughness directly following grades.
- Under the wet condition, the road could become smoother purely as a result of vehicle loading. (This trend obviously depends on the material see January 2017 to April 2017).
- The impact of storm events on the unsealed road performance is significant, especially when the storm intensity causes erosion (see April and June 2017).

Observations from the figure include the following:

- The grading activities do not always result in a reduction of the mean roughness but do narrow the distribution and extreme roughness ends (see October 2016 to November 2016 and April 2017 to June 2017).
- Some grading activities may result in a higher roughness level directly following the activity (see September 2016 to October 2016). This is often the case with shape correction or “deep grades” with the intention of significantly mixing the pavement material. This result may explain some of the observation on networks where road users complain about roughness directly following grades.
- Under the wet condition, the road could become smoother purely as a result of vehicle loading. (This trend obviously depends on the material see January 2017 to April 2017).
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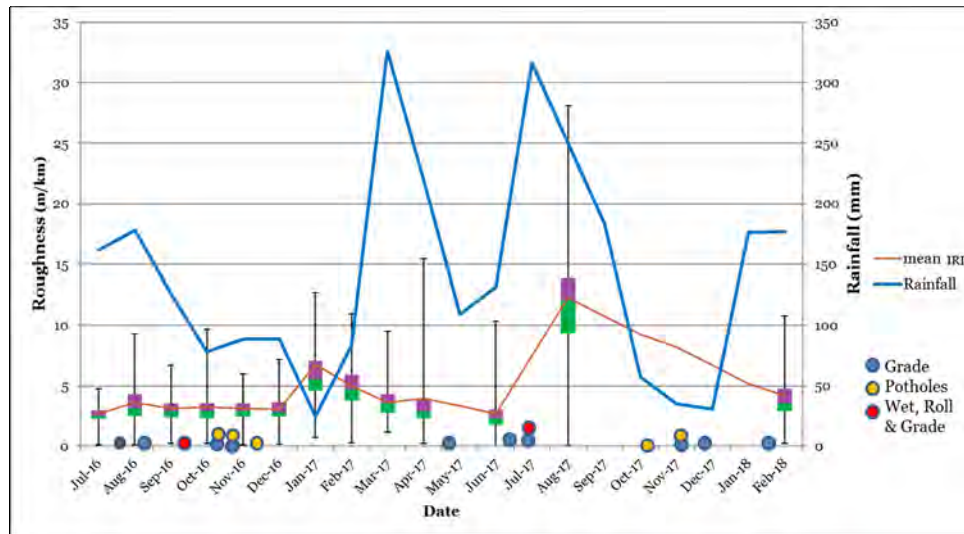


FIGURE 2 Roughness given maintenance and rainfall.

SUMMARY AND CONCLUSIONS

This research investigated the value from customer complaints, material properties, and a detailed understanding of the achievement of maintenance activities as primary evidence into investment decisions for unsealed roads. For example, a grading strategy could be developed to satisfy road user requirements plus adhere to sustainable maintenance intervention to ensure optimal use of gravel sources.

The research has concluded that a good understanding of when customers complain is needed before the feedback could be used directly as an input into decisions. The research further demonstrated that the outcome from grading event could be highly variable. Specific change or reduction in roughness as a result of grading is not always predictable. However, once an understanding of these dynamics is established, it is highly effective to drive maintenance regimes and investment levels. The research also re-confirmed some well-established fundamentals in managing unsealed roads including:

- Appropriate material selection is crucial to yield the best performance of unsealed roads.
- The quality of grading (grader operator skills) has a significant impact on the overall effectiveness of unsealed roads maintenance.
 - The performance of unsealed roads during rainfall events strongly depend on the adequacy of drainage in combination with the material properties.
 - Sealing of unsealed roads is only warranted when traffic volume increases above an economic threshold or where other options do not provide desired outcomes.
 - Choosing the right maintenance strategy at any given point of time will impact on not only the immediate condition outcome but also the medium to the long-term performance of the road section.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: Study Conception and Design: Henning (40%), Muir (30%), Bartlett (30%); Data Collection: Muir (40%), Bartlett (40%), Robertson (10%), Thom (10%); Analysis and Interpretation of Results: Henning (10%), Muir (10%), Bartlett (10%), Robertson (35%), Thom (35%); Draft Manuscript Preparation: Henning (60%), Muir (10%), Bartlett (10%), Robertson (10%), Thom (10%). All authors reviewed the results and approved the final version of the manuscript.

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A Novel Performance-Based Economic Analysis Approach *Case Study of Iowa Low-Volume Roads*

SAJJAD SATVATI
JERAMY ASHLOCK
ALI NAHVI
CHARLES T. JAHREN
BORA CETI
HALIL CEYLAN
Iowa State University

Approximately 110,000 km (68,400 mi) of granular roadways exist in the 183,500-km (114,000-mi) road network in the state of Iowa, and operation and maintenance of these roadways costs roughly US\$270 million annually. The major maintenance costs of these roads are aggregate cost and hauling costs from the quarry to the site. Accordingly, acquiring a cost-effective and high-performance surface material to be utilized in granular roadways can be a challenge. In this study, three conventional granular roadway materials and four coarser aggregate materials from different quarries were used to construct seven test sections to assess their relative performance and costs. The first three test sections were constructed with conventional materials, and the other four sections used optimum mixtures of the four coarse aggregate materials with the local conventional aggregate. The long-term performance and mechanistic behaviors of the different surface materials, including stiffness, changes in ride roughness, and dust production were evaluated for a period of 2 years. Using the resulting data, a mechanistic life-cycle benefit–cost analysis approach was developed to evaluate the use of coarse aggregate materials on granular roadways. The stochastic benefit–cost analysis results for different aggregate materials are presented in the form of probability density functions. Two different scenarios are presented based on the field test results, and the benefits in terms of dust production and surface ride quality are evaluated for each section.

INTRODUCTION

The quality of granular roadway materials (e.g., abrasion resistance, freeze–thaw durability) is very important since common surface deteriorations such as material loss, gradation change, loss of crown, surface erosion, rutting, and potholes can be directly related to the quality of the materials used in these roadways (*1*). Granular material sources such as aggregate quarries produce aggregates with different qualities and different prices. Moreover, depending on the location of the quarry the total cost of obtaining a given aggregate could vary significantly due to the hauling costs. Therefore, finding a cost-effective high-quality aggregate source (with a balance between quality and hauling costs) is a common concern and challenge for construction and maintenance of granular roadways.

Since hauling of high-quality aggregate from greater distances increases construction and maintenance costs significantly, it is important to assess the benefits of using these materials for

construction of granular roadways, and determine if they can sustain performance for longer durations and with less maintenance. More frequent maintenance activities on granular roadways may require road closures which would increase maintenance costs (2). In addition, low-quality aggregates could result in greater vehicle operating costs (e.g., reduced fuel consumption, increased wear, and damage). Moreover, low-quality aggregates will generally abrade faster, thus producing more dust which can penetrate into engines and other vehicle components, resulting in greater wear rates and more frequent maintenance (3, 4).

There is a lack of high-quality aggregate sources in United States, particularly in the Great Plains region. For instance, previous studies reported that aggregate sources northeast Iowa have higher-quality aggregates than other regions such as the west and south portions, requiring the use of only half as much aggregate for comparable roadway performance (5–7).

Stiffness, dust, and surface roughness are three main factors in the evaluation of the serviceability of granular roadways and can serve as quality assurance–quality control (QA/QC) parameters. Evaluating the performance of the granular roadway layers by nondestructive tests is inexpensive due to their availability and low operation costs (1–4). There are several nondestructive test methods for stiffness measurements of granular roads. These include multichannel analysis of surface waves, falling weight deflectometer, automated plate load test, and lightweight deflectometer (LWD) tests. Among these tests, LWD is the most commonly used for measuring the stiffness of unbound granular surface layers due to its relatively low cost in terms of equipment and labor required (5, 6). In order to measure the amount of dust produced in the road, a quantitative dust measuring device has been developed that is commonly used as a repeatable and reproducible method (7–9). Moreover, measuring the International Roughness Index (IRI) by smartphone as a simple and low-cost solution to evaluate the surface roughness is growing rapidly while other traditional methods such as profilometers are expensive and labor-intensive (10, 11).

Approximately 50% of the total road network in the United States is composed of granular roads. Although literature on the importance of maintenance for paved roads is vast, this is not the case for granular roads (7). Because granular roadways provide access for rural areas, mostly for transportation of agricultural products, sustainability of these roads is very important to the economy of the United States (4). Since operation and maintenance of these granular roads costs roughly US\$270 million annually (7), economic analysis is helpful to determine the cost-effectiveness of transporting materials from high-quality sources to replace low-quality local materials in granular road construction and maintenance (18–21).

Benefit–cost analysis (BCA) is an assessment of decisions based on the consequences (benefits and drawbacks) in accordance with them (22). BCA evaluates the benefits of using different alternatives instead of using a base case, by calculating the benefit–cost ratio (BCR) which is defined as the ratio of the present value of benefits over the service life of the project to the present value of the initial and future costs (23, 24). BCA is closely dependent on many important decision-making factors. Therefore, to define the benefits properly requires a deep oversight of the project and exact knowledge of the costs.

While there have been extensive mechanistic-based performance studies on low-volume roads (5, 25–31), and some have studied the economic performance of these roads (7, 21), to the best knowledge of the authors, performance-based economic studies of granular roads have not yet been investigated in detail. This paper presents a performance-based life-cycle benefit–cost analysis (LCBCA) method for comparing economic performance of granular roads constructed in rural road systems. Monetizing resilience benefits of higher-quality aggregate in a LCBCA

framework provides an assessment of broader benefits from such materials and contributes to building more-resilient and sustainable infrastructure. In this study, a granular road was defined as a two-lane local road with a granular surface, with traffic of less than 400 average daily vehicles and providing access for areas with low population density (32). In this study, seven different surface aggregate materials were collected from four different Iowa quarries and different performance measures were tested over a 2-year period. Using the performance measure tests, different possible scenarios for establishing the maintenance frequency of test sections were developed to use in an economic analysis framework. This paper has two major objectives. The first is to identify benefits associated with different types of gravel materials based on their long-term performance. The performance measurement techniques used in this study to identify such benefits were LWD, dustometer, and IRI. The second main objective is to investigate the economic performance of granular roads constructed with the different materials.

SITE DESCRIPTIONS

Surface aggregate materials for this study were collected from quarries featuring four different Iowa bedrock types: Lime Creek formation (LCF), Oneota formation dolomite (OFD), Bethany Falls limestone (BFL), and crushed river gravel (CRG) (Figure 1). The first three quarries provided both conventional (Class A) and coarse clean aggregate materials, while the CRG provided crushed coarse clean gravel materials. The main difference between the Class A and clean materials was their particle sizes, whereby the Class A materials had higher fines contents and lower percentages of coarse aggregates than the clean materials.



FIGURE 1 Locations of the aggregate quarries.

Seven field test sections were built in Decatur County, Iowa. The first three sections consisted of Class A materials—LCF Class A, OFD Class A, and BFL Class A—while the local BFL Class A material was also mixed with clean aggregate materials collected from all four quarries for the final four sections. Therefore, the local BFL Class A material was the only one mixed with the four clean materials to examine the mechanistic performance of such mixtures. To achieve the best performance and durability for the mixture sections, the optimum target particle size distribution (PSD) curves of the mixtures were determined via the gradation optimization method described in Li et al. (5). According to the optimization analyses, it was determined that the mixing ratios by weight for the last four test sections should be as follows: 80% BFL Class A with 20% BFL clean; 70% BFL Class A with 30% OFD clean; 70% BFL Class A with 30% LCF clean; and 70% BFL Class A with 30% CRG clean aggregate. Figure 2 shows the grain size distribution of all seven surface aggregate materials.

Table 1 also summarizes laboratory results of the soil index properties, including sieve analysis, Atterberg limits, and soil classification. The gravel, sand, silt, and clay contents for the surface aggregate materials ranged from 46% to 79%, 13% to 45%, 7.8% to 13.5%, and 0.1% to

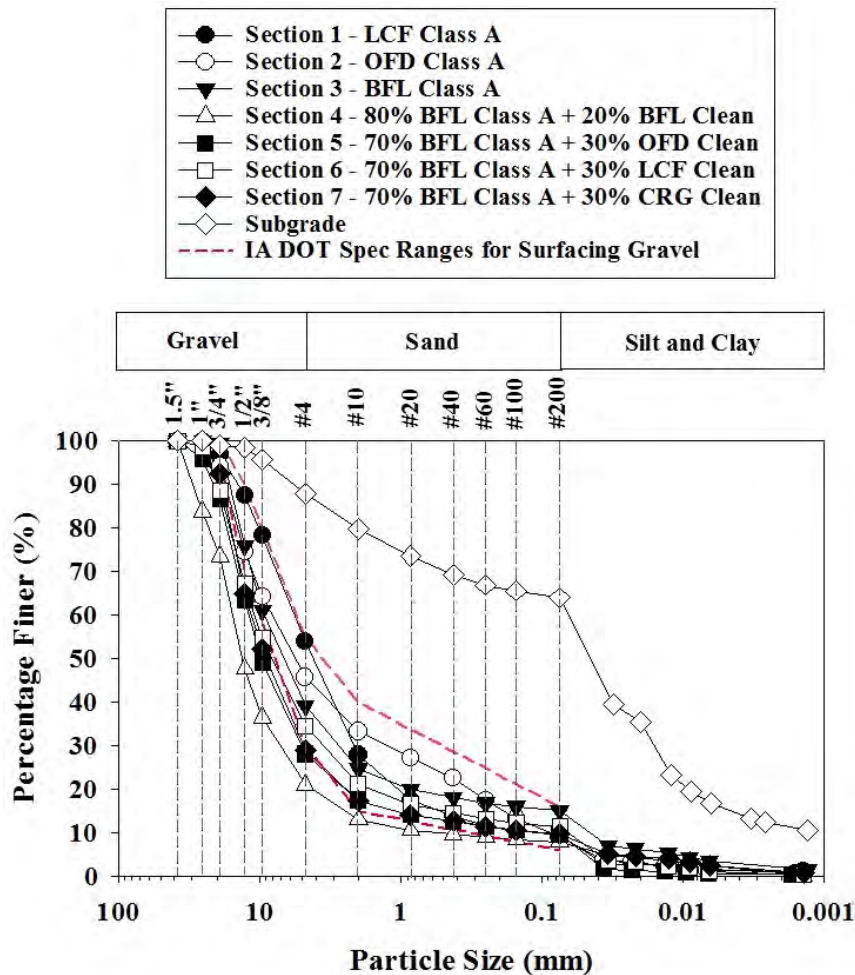


FIGURE 2 PSD curves for the surface aggregate materials used in this study.

TABLE 1 Particle Size Analysis, Atterberg Limits, and Soil Classification Results

Parameter	Section 1 (LCF Class A)	Section 2 (OFD Class A)	Section 3 (BFL Class A)	Section 4 (% BFL Class A + 20% BFL Clean)	Section 5 (%BFL Class A + 30% OFD Clean)	Section 6 (% BFL Class A + 30% LCF Clean)	Section 7 (% BFL + % CRG Clean)	Subgrade
Particle Size Analysis Results (ASTM D422-03)								
D ₁₀ (mm)	0.1	0.1	0.1	0.62	0.1	0.1	0.1	0.1
D ₃₀ (mm)	2.2	1.2	2.7	7.17	5.0	3.6	4.8	0.0
D ₆₀ (mm)	5.7	8.2	9.2	15.45	11.9	10.8	11.3	0.0
Coefficient of uniformity (C _u)	47.5	90.5	185	25.12	110.7	154	103.2	0.0
Coefficient of Curvature (C _c)	6.7	2.1	16.6	5.41	19.4	17.1	19.1	6.6
Atterberg Limits Test Results (ASTM D4318-10e1)								
Liquid limit (%)	14.6	NA	19.5	20	19	17	19	31.1
Plasticity index	1.2	NA	4.2	5.2	3.6	3	5.1	11.7
AASHTO and USCS Soil Classification (ASTM D2487-11 and D3282-09)								
AASHTO	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-6
USCS group symbol	GW	GW	GW	GW	GW	GW	GW	CL

2%, respectively. All the surface aggregate materials were classified either as USCS well-graded gravel (GW) or AASHTO A-1-a, while the subgrades were sandy lean clay (CL) or A-6. Test Section 2 (OFD Class A) exhibited nonplastic behavior, while the liquid limit and plasticity index values for the other sections ranged from 14.6 to 20 and 1.2 to 5.2, respectively.

Figure 3 indicates that the thicknesses and widths of all sections were 10 cm (4 in.) and 9.1 m (30 ft), respectively. The length of each test section was 152.4 m (500 ft), except for Section 2 (OFD Class A), where due to the lack of sufficient material, the length was reduced to 91.4 m (300 ft).

FIELD TESTS AND MAINTENANCE OPERATIONS

In this study, several nondestructive field tests including LWD, dustometer, and IRI were performed to monitor the performance and estimate the required maintenance frequencies for the test sections. These tests were selected because they are inexpensive, can be performed relatively quickly, and the equipment required to conduct these tests are generally available for most agencies and private-sector entities.

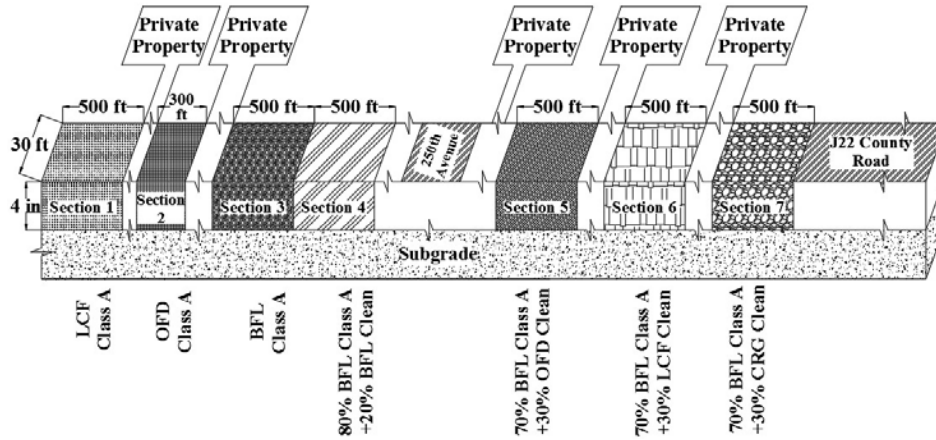


FIGURE 3 Schematic diagram of the test sections.

Test sections were subjected to major maintenance only once. Before the major maintenance operations, the change in thickness of each section was measured and the amount of materials required to recover the material lost for each test section was calculated. The required aggregate materials were then determined according to the proposed mixing ratio based on the optimized Fuller’s model as described by Li et al. (5). Figure 4 shows the change in thickness of each test section and demonstrates that Section 5 (70% BFL Class A + 30% OFD clean) and Section 2 (OFD Class A) experience the largest and smallest changes in their original thickness, respectively.

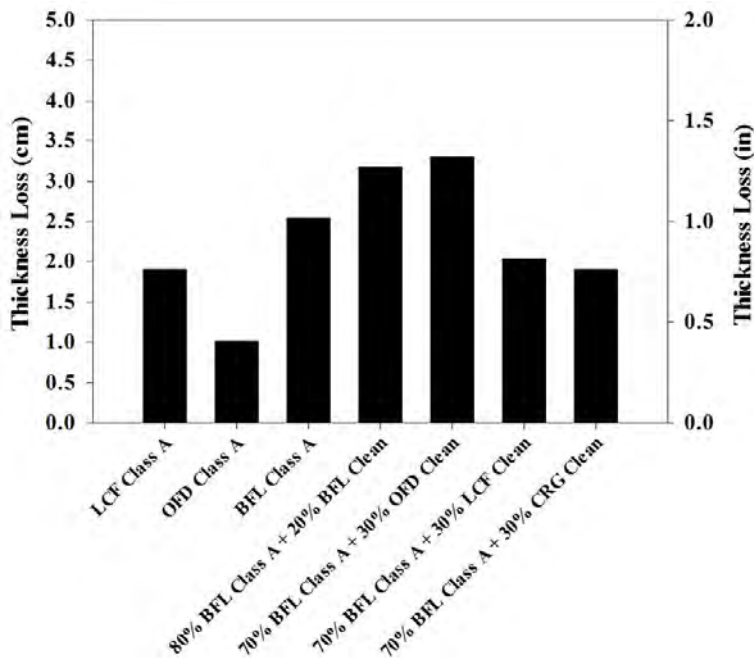


FIGURE 4 Change in thickness of surface layers before maintenance, from September 2016 to May 2017.

The major maintenance procedures included the following steps: (1) scraping the granular surfaces down to the subgrade with a motor grader (scarifier blades were required for the second section due to its stronger surface); (2) spreading new Class A and clean aggregates from piles; (3) blading of both existing and new aggregates to achieve the optimum uniform mixture; (4) shaping the crown of the granular surface with motor graders; and (5) compacting the granular surface with the motor grader (12 to 16 passes).

Lightweight Deflectometer Tests

LWD tests were conducted to determine the maintenance frequency required for the test sections. The tests were performed on five points within each test section to evaluate the in situ composite elastic modulus (E_{Comp}) (stiffness) of the granular surfaces and subgrades, as a measure of road serviceability. This stiffness is a function of several factors, including compaction quality, packing structure of the various particle sizes (33), density of the road layers, water content, and temperature (34). Any changes in these factors can result in severe distresses (e.g., potholes, rutting, etc.), creating a need for road maintenance. Therefore, along with the E_{Comp} data for each test section, the surface layer temperature and water content are presented. The ambient temperature of the surface course was measured using a thermocouple installed in the middle of the first section and the same ambient temperature was assumed for all the sections. The water content values were measured from samples collected during field testing. The LWD device used for testing in this study features a 10 kg (22 lb) hammer with a drop height of 0.5 m (19.69 7 in.), and a base plate diameter of 30 cm (11.81 in.). The in situ elastic modulus then was calculated based on the average vertical deflection as it is shown in Equation 1.

$$E_{LWD} = \left(\frac{1-v^2}{d_0} \right) \sigma_0 A f \quad (1)$$

where

E_{LWD} = elastic modulus, as the result of LWD test;

σ_0 = vertical stress applied on top of the plate;

v = Poisson's ratio (assumed as 0.4);

d_0 = applied stress;

A = plate radius; and

f = shape factor (assumed for a uniform stress distribution) (35).

Dustometer Tests

The dustometer test is another road performance measure used in this study to estimate the appropriate granular road maintenance frequency. To evaluate the dust production of each test section in relation to the different aggregate sources utilized in the surface layers, dustometer tests were performed several times over the length of the project. Figure 5 shows the setup of the dustometer device, attached to the bumper of a 1-ton truck by a steel bracket. It has a 30.5 × 30.5 cm (12 × 12 in.) steel mesh with a 200 μm (0.0079 in.) mesh size sieve to prevent large particles from damaging the tightly held filter paper. A 1/3-hp suction pump is connected to the mounted dustometer with a 2-in. diameter flexible hose to collect dust behind the rear wheel

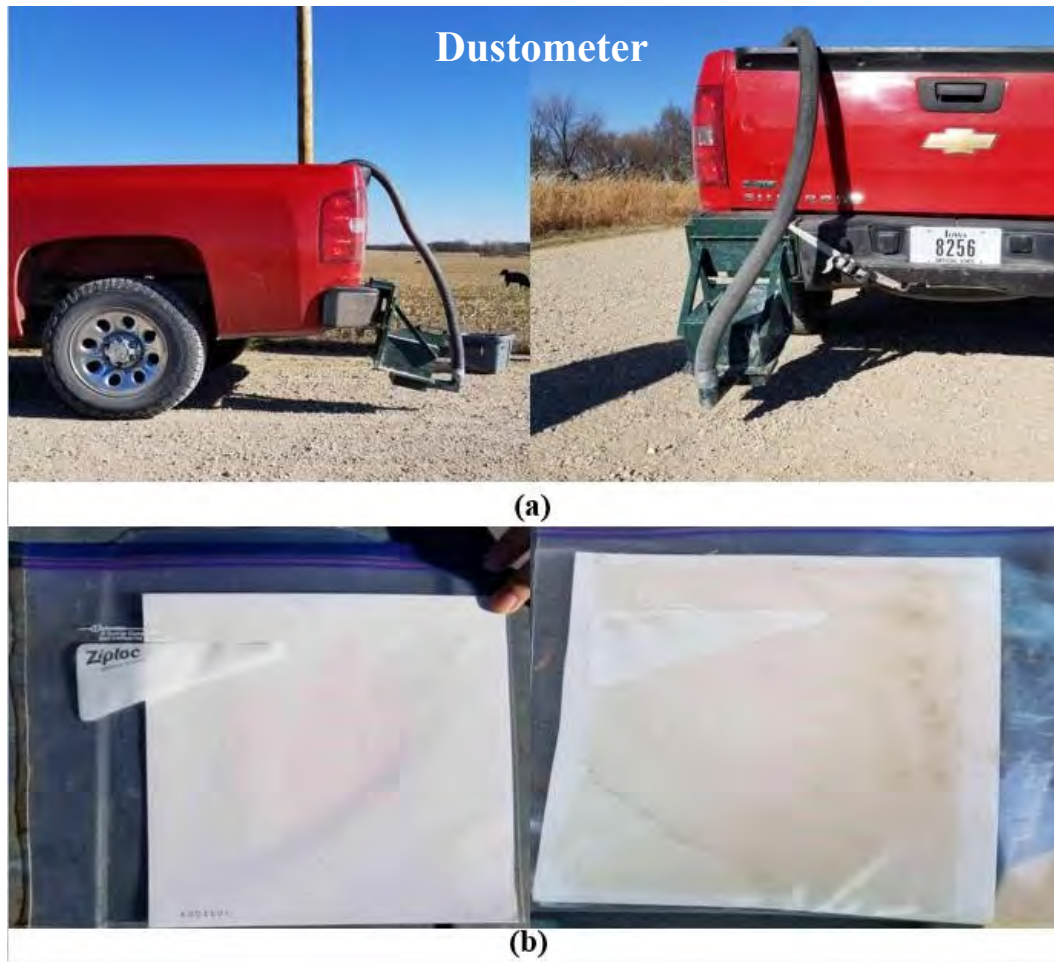


FIGURE 5 Dustometer test set up: (a) dustometer setup and (b) dust production measurement paper.

while driving at a speed of 72 km/h (45 mph). A 4,400-watt gasoline-powered generator provides power for the suction pump. The filter paper was removed after performing the test over a section and the mass of the dust on the paper divided by the length of the sections to determine the amount of dust per unit length.

International Roughness Index Tests

Roughness of the road surface as representative of ride quality is an important factor to evaluate the granular roadway performance, and lower IRI values reflect higher ride quality, lower fuel consumption, and longer service life (36). In the current study, the collection of road roughness measurements representative of road condition was done using a smart phone application named Roadroid. This software used a built-in smart phone accelerometer to evaluate roughness index of the different surfaces in a rapid and cost-effective manner (37). In this method, the smart phone was mounted on the windshield of a 1-ton truck and, after adjustments, the calculated IRI (cIRI) values were measured and stored in the phone while driving between 64 to 80 km/h (40 and 50 mph).

PERFORMANCE-BASED ECONOMIC ANALYSIS APPROACH

Dharmadhikari et al. presented four main steps for performing a LCBCA: (1) determining the project base case and alternatives; (2) defining the benefits; (3) cost and benefit calculation; and (4) determining the current value of costs and benefits (38). The base case is defined as a condition where no alternatives are suggested, and the alternatives are the other options to be considered for making the project more beneficial (38).

The determination of both the base case and the benefits should be conducted with extreme care to produce a valid and trustworthy cost analysis. Values of annual costs and benefits and the project's present value considering the proper discount rate should be included in an overall approach to the LCBCA (39). Jones et al. enumerated traffic forecasts, cost estimations, discount rates, value of life, safety, value of time, regional impacts, local impacts, equity, environmental impacts, and residual use as the major challenges in performing BCA for transportation infrastructure (40).

In the present case, Section 4, built with a mixture of local conventional Class A and clean local materials (Section 4 – 80% BFL Class A + 20% BFL clean), with the lowest construction cost among the other aggregate options, was considered to represent the base case scenario. If a granular road incurred a cost lower than the base case scenario (Section 4) it was considered beneficial (cost saving) in the economic analysis platform. For example, if one of the user costs associated with a road alternatives would be lower than the base case user costs, the difference between these monetary values was considered as a cost saving or benefit, while if a pavement's cost item was larger than the costs associated with the granular road built with local conventional material, the difference between these two costs items would be considered as a cost factor in the BCR.

The data required to estimate benefits (cost savings) and additional costs associated with the alternative granular roadway materials were collected from construction, maintenance, field measurements, and Iowa Department of Transportation (DOT) publicly available data sets. Since one of the most important factors in the economic analysis of granular roads is the frequency of maintenance activities (7), many performance measurements are conducted to compare new alternatives with the base case test section (Section 4). Then, based on similarities and differences among the alternative road sections and base case maintenance frequencies associated with an aggregate, the benefits of the each alternative section were estimated. Then, using maintenance frequencies of granular road material alternatives, benefits–costs associated with them were estimated. The major benefits–costs considered in the LCBCA platform are reduction–increase in road users' lost time, reduction–increase in maintenance costs through the life cycle, and reduction–increase in car-damage costs. Figure 6 summarizes the methodology suggested in this study.

Deterministic BCA involves utilizing point estimates (discrete values), resulting in a single output value (8, 9). If the ratio of benefits to net costs is larger than one (>1), while general economic arguments would support action to make the associated investment, there are issues associated with deterministic factors such as sensitivity of results to the chosen discount rate and a mismatch between the volatilities of underlying factors (uncertainty associated with initial costs, maintenance frequency, traffic volume, duration of service life, etc.) that can be addressed by building a stochastic economic analysis model. To this end, as in previous studies on pavement decision-making (10–15), a stochastic analysis approach was used to perform the economic analysis. A stochastic benefit–cost model would use Monte Carlo simulation and allow input variables to fluctuate through their probability distributions based on recent historical and regional changes.

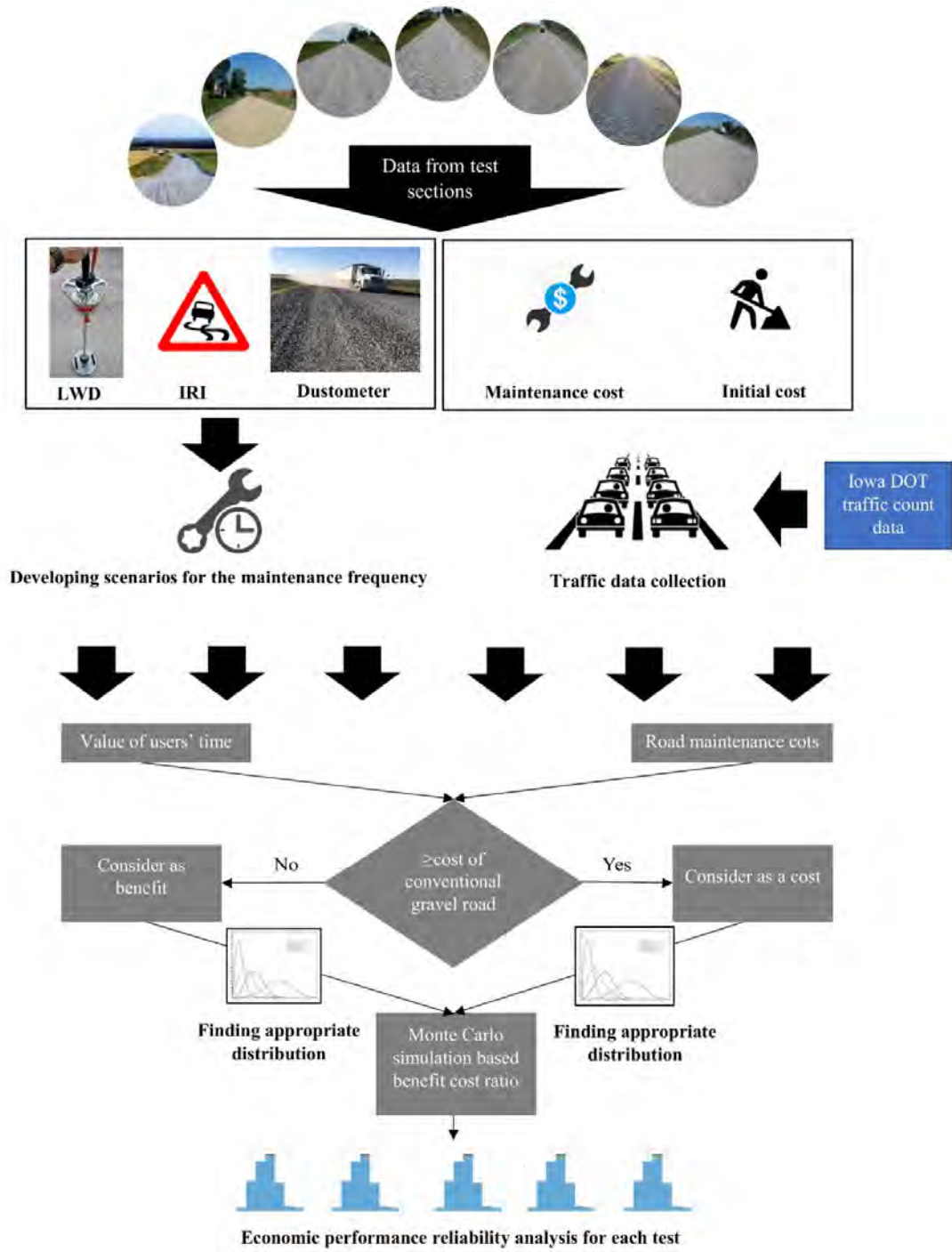


FIGURE 6 Research methodology.

FIELD TEST RESULTS LWD

The average results of the LWD composite elastic modulus for each test section, along with temperature and water content data over the past 2 years, are presented in Figure 7.

Immediately after construction on October 2016, the first set of LWD tests were performed on all sections, and the calculated LWD composite elastic modulus (E_{Comp}) for the sections were very close to each other and in the range of 54.8 MPa (7.95 ksi) to 81.8 MPa (11.86 ksi). However, the water content of the sections varied over a wide range (between 3.5% and 10.1%). The second set of LWD field testing was performed on November 2016 and the range of LWD E_{Comp} was between 54.8 MPa (7.95 ksi) to 80.5 MPa (11.68 ksi), similar to the values measured in October 2016. It is worth mentioning that the E_{Comp} of Section 3 (BFL Class A) changed insignificantly from that of October 2016 and all the other sections except for from Section 4 (80% BFL Class A + 20% BFL clean) had an increase in their E_{Comp} . The third set of LWD field testing was performed in December 2016, during the freezing season, and all sections exhibited a significant increase in their E_{Comp} values because of the very low temperatures, that made the ground frozen and the range was between 143.8 MPa (20.86 ksi) to 445.5 MPa (64.62 ksi). Note that since the water content range was smaller (between 2% to 6.3%) then it might be concluded that temperature effects were much greater than the effect of water content during the freezing season. The first LWD field testing in 2017 was performed in February, during the thawing season, and all sections exhibited a significant decrease in their E_{Comp} range due to thawing between 40.8 MPa (5.92 ksi) to 75.4 MPa (10.94 ksi). The second 2017 LWD field testing was performed in April, after freezing and thawing seasons. All of the sections exhibited a slight increase in their E_{Comp} values except Section 2 (OFD Class A), which the LWD E_{Comp} did not change for this section. The range of the LWD E_{Comp} was between 44 MPa (6.38 ksi) to 99.1 MPa (14.38 ksi). The last set of LWD field testing in 2017 was conducted on June 2017, after the major maintenance to monitor the effects of removing distresses (e.g., potholes, rutting, and wash-boarding) on the soil stiffness. All sections except for Section 2 (OFD Class A) exhibited an unanticipated slight decrease in their E_{Comp} values compared to those from February 2017. However, the range stayed similar to values from April 2017 between 42.5 MPa (6.16 ksi) to 96.4 MPa (13.98 ksi). The final set of LWD field testing measurements was performed on May 2018, after the second freeze and thaw season. The E_{Comp} values for Section 1 (LCF Class A) and Section 7 (70% BFL Class A + 30% CRG Clean) were virtually changed, and the E_{Comp} values increased for all the other sections except for Section 2 (OFD Class A). The highest E_{Comp} was observed for Section 4 (80% BFL Class A + 20% BFL clean) (13.51 ksi), and the lowest E_{Comp} was observed for Section 7 (70% BFL Class A + 30% CRG clean) (6.16 ksi).

The reason for the decrease in the E_{Comp} value for this section may be related to poor binding between the coarse and fine aggregate materials. It was observed that the coarse materials in this section (CRG clean) had moved gradually to the roadsides due to the lack of binding between the coarse and fine aggregates (Figure 8).

As shown in Figure 7 and also explained in detail above, E_{Comp} value (stiffness measure) cannot be used as a unified standard factor indicator for longevity of granular road serviceability, although some similar trends were observed for some of the sections. Therefore, the LWD results were not considered for the BCA and not used to estimate the appropriate maintenance frequency for the granular road sections.

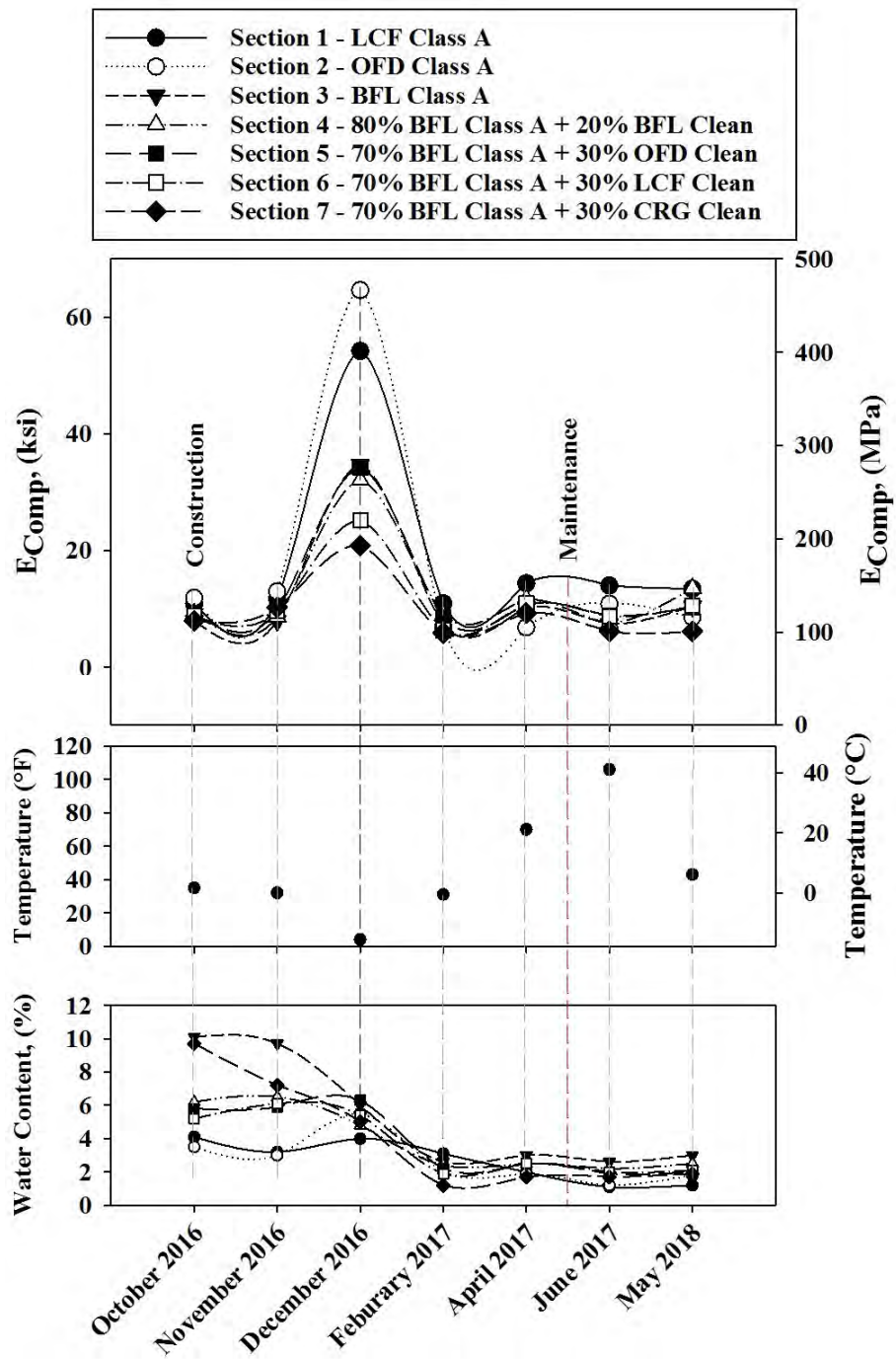


FIGURE 7 Average LWD composite elastic modulus results for each section over time.



FIGURE 8 Movement of the loose aggregate materials to the sides of the road for Section 7 (70%BFL Class A + 30% CRG clean).

Dustometer

Figure 9 shows the results of dust production per mile during 2 years of the project for all seven test sections. To focus on the important factors effective in dust production, the temperature, water content, and fines content (amount passing #200 sieve) data are presented in Figure 9.

To monitor surface material degradation, samples from the granular surfaces of each section were collected for gradation analyses. The results for fines content for each section were derived from gradation tests.

The first dustometer test was conducted on October 2016 (after construction) and the dust production was in the range of 0.72 g/km (0.0026 lb/mi) to 4.92 g/km (0.018 lb/mi). The ambient temperature was about 35°F, the water content was in a range between 3.5% and 10.1%, and the fines content was in a range between 9.14% and 15.08%. The second set of dustometer test was conducted on November 2016 at about the same ambient temperature with October 2016 (32°F).

Surface aggregate sample collection was not performed because since it was assumed that the materials had not deteriorated significantly at that time. The gradation and fines content of the materials was assumed to be the same as when the samples were collected following construction on October 2016. The moisture content of the surface materials was in a range between 3% and 7.2% and did not change significantly since October 2016. The rate of dust production placed between 0.62 g/km (0.0022 lb/mi) to 4.66 g/km (0.0165 lb/mi). The third

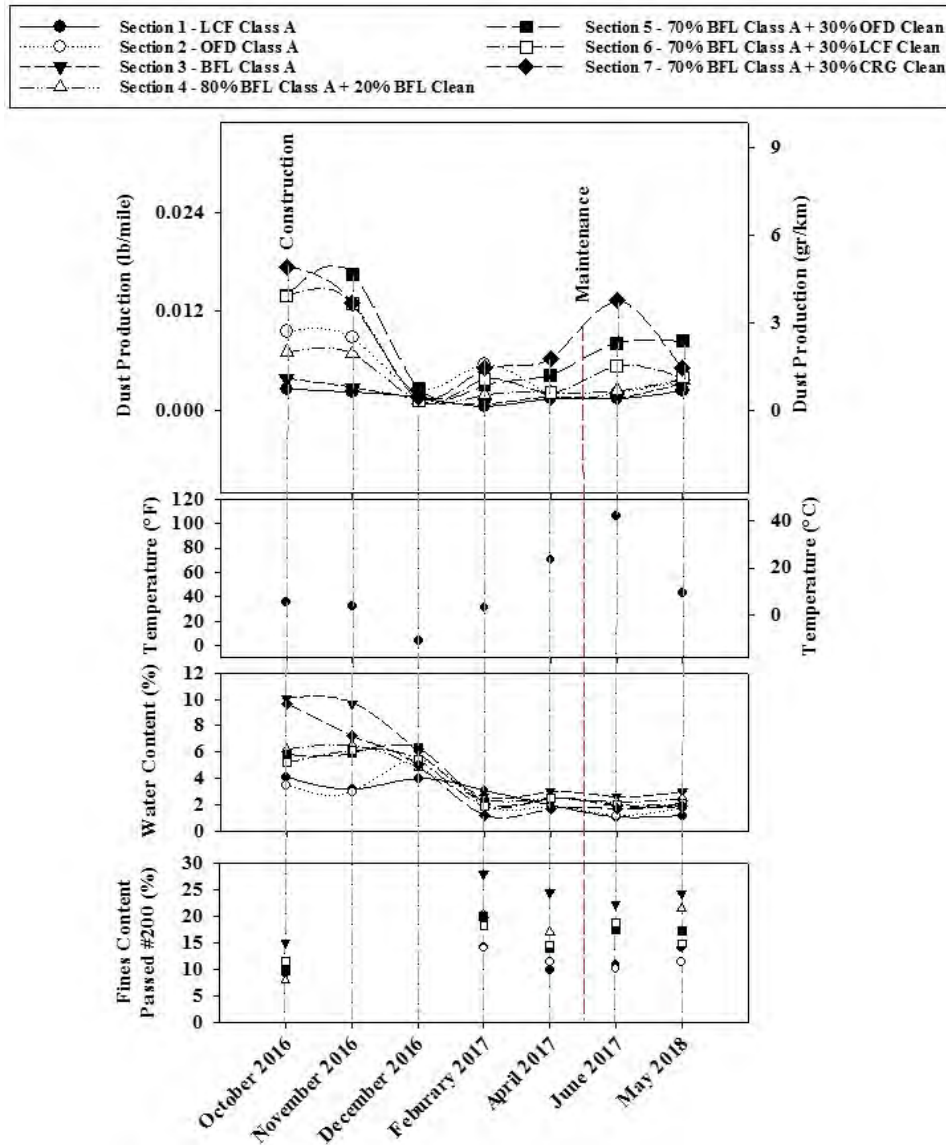


FIGURE 9 Dustometer results for each section over time.

set of dustometer tests was conducted on December 2016, during the freezing season (4°F). The water content range had decreased to a smaller range, between 4% and 6.3%. The dust production rate decreased significantly, and it was confined to a small range for all of the sections, between 0.33 g/km (0.0012 lb/mi) and 0.75 g/km (0.0027 lb/mi). The sample collection was not conducted on December 2016 due to frozen ground. The fourth set of dustometer tests was conducted on February 2017, during the thawing season, where the ambient temperature got close to that at the time of construction (31°F). The water content range decreased to 1.2% to 3.1%. The dust production rate was increased significantly from December 2016, and it was in the range of 0.13 g/km (0.0005 lb/mi) to 1.57 gr/km (0.0056 lb/mi). The overall fines content of the surfaces increased significantly from October 2016, due to deterioration of the surface aggregate during the freezing and thawing season. The fifth set of dustometer tests was

conducted on April 2017, after the thawing season (71°F). While virtually all the sections had almost the same range of dust production between 0.39 g/km (0.0014 lb/mi) to 1.77 g/km (0.0063 lb/mi) and the same range of water content between 1.7% to 3%, with February 2017. The sixth set of dustometer field tests was conducted on June 2017. A slight increase in the range of dust production, which was between 0.39 g/km (0.0014 lb/mi) to 3.74 g/km (0.013 lb/mi), was observed compared to February and April 2017 due to the higher temperature (106°F) and drier surface (1.1% to 2.6% water content). The final set of dustometer field tests was conducted on May 2018 after the second freeze and thaw season (43°F). The dust production was observed to be in the range of 0.66 g/km (0.0023 lb/mi) to 2.36 g/km (0.0084 lb/mi) with the water content in the range of 1.2% to 3%.

While dust production depends on many different factors such as condition of surface materials (wet or dry), temperature, wind, etc., an overall reading of the dustometer results shows that dust production after construction and maintenance was higher than at other times, depending somewhat on environmental conditions. Nevertheless, all times showed about the same amount of dust production. The average results of dust production for each section for the different times of performing dustometer test are shown in Figure 10. It can be concluded that Section 7 (70% BFL Class A + 30% CRG clean) had the maximum dust production value, 2.48 g/km (0.0088 lb/mi), and Section 1 (LCF Class A) had the lowest dust production value, about 0.48 g/km (0.0017 lb/mi). Sections are categorized by color codes in Figure 10.

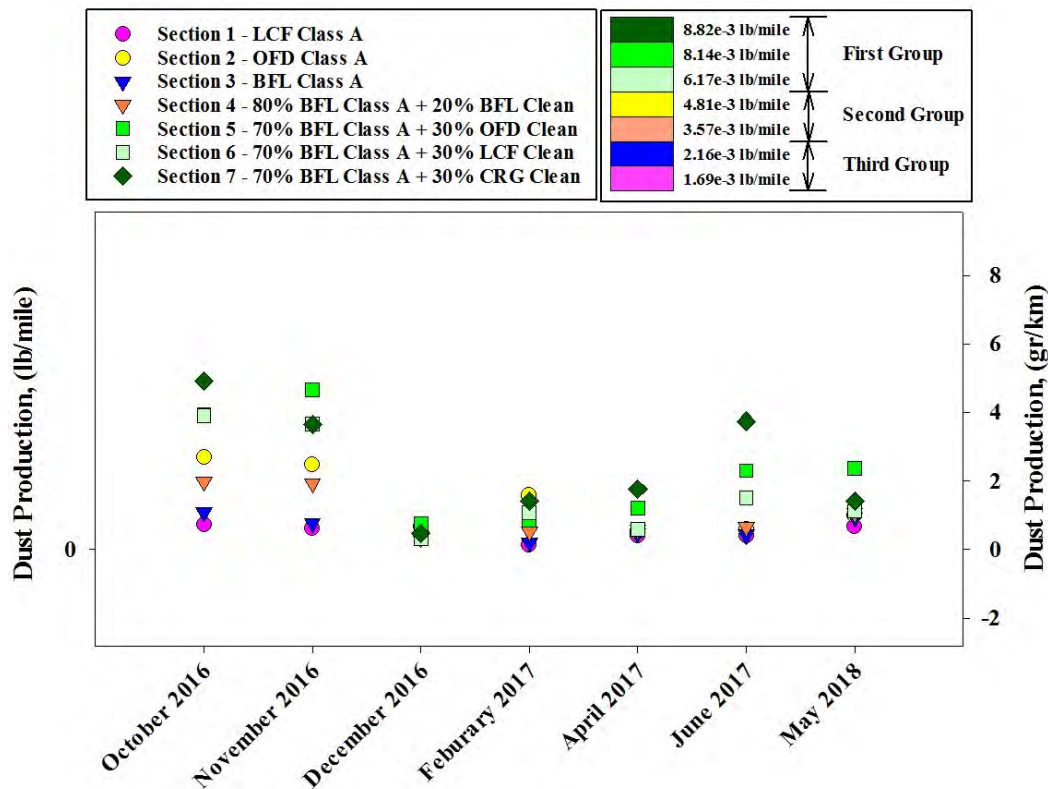


FIGURE 10 Test sections classification based on dust production.

Summarizing the above discussion, demonstration sections were divided into three categories (Figure 10). The first group with the highest dust production included Sections 7, 5, and 6; Sections 2 and 4 were the demonstration sections with moderate dust production; and Section 3 and Section 1 exhibited the lowest dust productions.

As mentioned in the test section descriptions, we know that the required maintenance time for the road built with conventional materials (Section 4, base case) would vary between 1 to 3 years depending on traffic volume, temperature, freeze–thaw cycles, etc., so for a stochastic BCA three possible scenarios were developed to determine maintenance time of conventional road sections (i.e., best case = 3 years; most likely case = 2 years; and worst case = 1 year). Moreover, by considering dust production as a performance measure of the gravel road section, two other scenarios were developed for the second and the third groups. Figure 11 presents the best, most likely, and the worst cases for the possible maintenance scenarios based on dust measurements for all sections. The base, most likely, and the best cases of the required maintenance times were considered to be, respectively, 3, 4, and 5 years for Sections 1 (LCF Class A) and 3 (BFL Class A); 2, 3, and 4 years for Sections 2 (OFD Class A) and 4 (80% BFL Class A + 20% BFL Clean); and 1, 2, and 3 years for the last three sections (70% BFL Class A + 30% OFD clean, 30% BFL Class A + 30% LCF clean, and 70% BFL Class A + 30% CRG clean). Although most test sections were in good condition, to monitor the condition of the road the maintenance was conducted after almost 1 year for all test sections. This is mainly due to the limited time associated with the project.

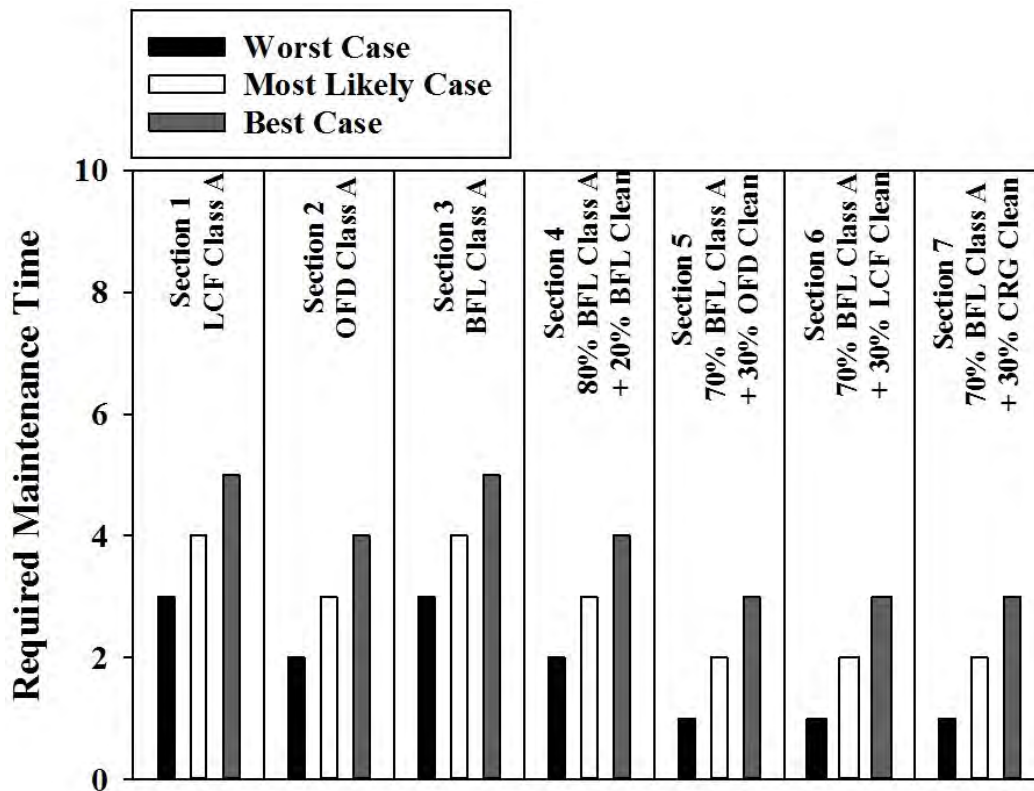


FIGURE 11 Maintenance frequency scenarios developed based on the dustometer results.

International Roughness Index

In this project, IRI was measured by Roadroid, an Android-based application. In order to remove any additional movement of the phone while performing IRI, a firm mount was used to connect the phone to the windshield. Moreover, the same truck and mounting location were used each time the test was performed. The calculated IRI (cIRI) with a narrower range of speed between 60 to 80 km/h was used, rather than of the estimated IRI (eIRI) which has a broader range of speed between 20 to 100 km/h. Therefore, cIRI values provided higher accuracy than eIRI values (41).

The cIRI values measured during this study are shown in Figure 12. Based on the IRI results, test sections were categorized into two different groups (fair performance or poor performance) (Figure 12). In addition, similar to the dustometer section, but based rather on IRI results, scenarios were developed for estimating required maintenance time of granular road sections for use in stochastic economic analysis (Figure 13).

COST–BENEFIT ESTIMATION

Costs and benefits associated with new granular roadway materials were estimated. As mentioned in the methodology discussion, if a section's incurred cost was lower than the base case scenario (Section 3, BFL Class A), this was considered to represent a benefit (cost saving). To conduct the economic analysis, reduction–increase in users' time was monetized along with cost/benefit associated with alternative unpaved road material maintenance operations. In the remainder of this section calculations for these two methods are explained in detail.

Construction and Maintenance

Figure 14a and 14b show the details of the construction and maintenance costs for each test section. Due to the sections having different lengths, while costs associated with the new gravel road material (gravel and hauling costs) were considered on a per-mile basis, it was observed that the equipment and in-site labor costs were virtually the same for all sections, so only the cost of hauling material and aggregate were considered in the economic analysis.

The differences between the costs of maintaining conventional granular roads with the new ones were estimated using maintenance frequency estimated values and actual costs of maintenance conducted on May 2017 (Figure 14). Equation 2 was used to calculate maintenance cost savings–additional costs estimations:

$$D_m = N_f \times N_c - C_f \times C_c \quad (2)$$

where

D_m = the difference in monetary value between the cost of maintaining conventional roads with new ones;

N_f = the new maintenance frequency;

N_c = the new gravel road maintenance cost (per mile);

C_f = the conventional road maintenance frequency; and

C_c = the conventional granular road maintenance cost.

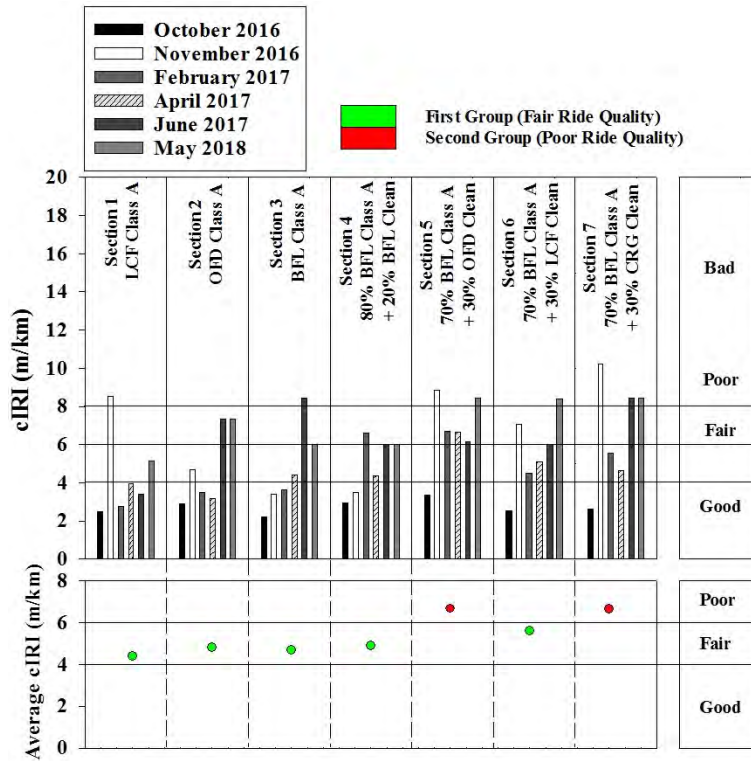


FIGURE 12 Roads roughness results for each section over time.

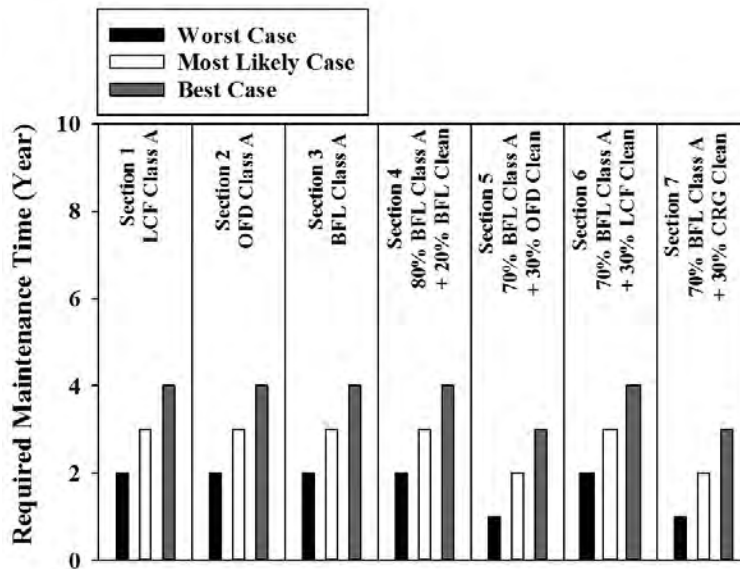


FIGURE 13 Maintenance frequency scenarios developed based on the IRI results.

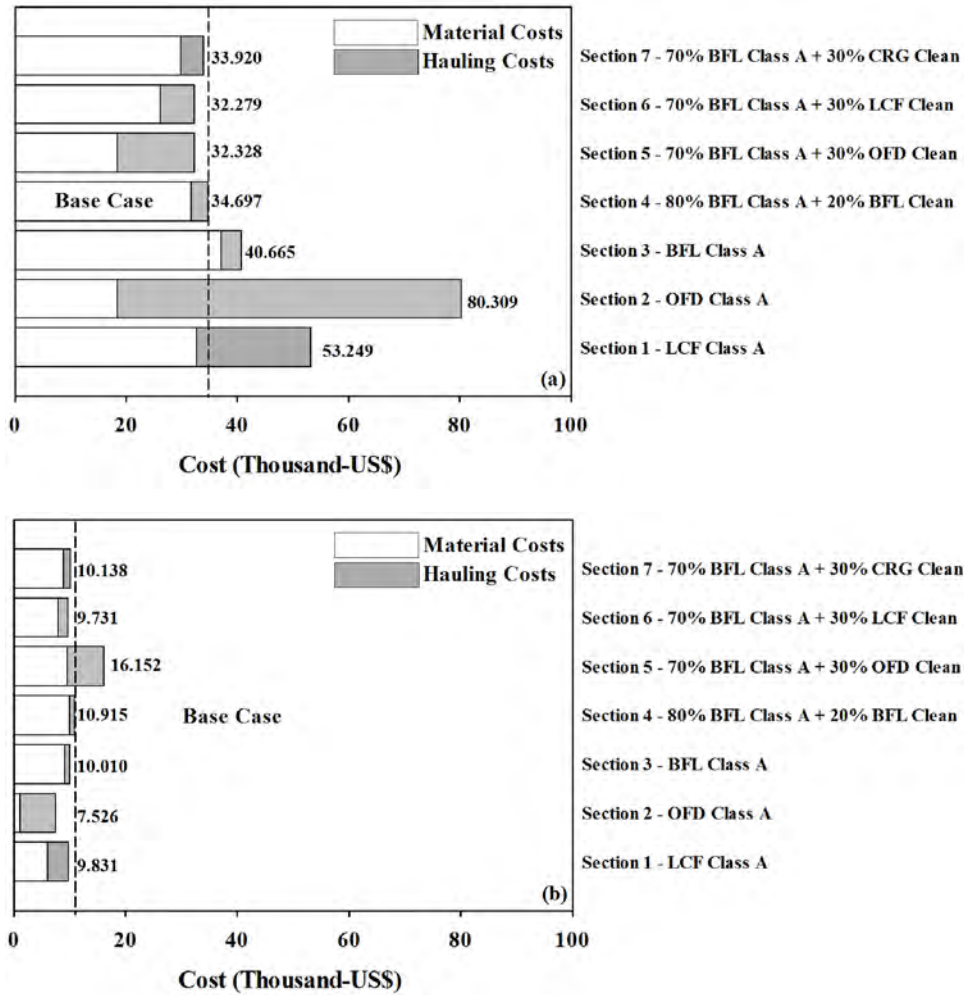


FIGURE 14 (a) Construction costs and (b) maintenance costs.

Value of Users' Time

Delay time is the extra travel time required either to pass through a work zone or to detour around it (42). Since travel time costs are given serious consideration and can become a significant factor when large queues occur, best-practice LCCA calls for consideration of not only agency costs, but also costs to roadway users (43). According to FHWA guideline (42), the following formula can be used to estimate travel delay cost:

$$\text{Travel delay cost} = (1 - T_t) \times P \times V_p + T_t \times V_t \tag{3}$$

where

T_t = truck traffic percentage (based on discussions with Iowa county engineers T_t assumed to be 25%);

P = personal travel;

V_p = value (US\$/h) of personal travel time; and
 V_t = value (US\$/h) of truck travel time.

Values of personal and truck travel time, obtained from the Bureau of Labor Statistics (BLS) data base (44), are US\$25/h and US\$54/h, respectively. Iowa granular roadway traffic data was obtained from the Iowa DOT traffic count data base (Figure 15) (45). Traffic volume was considered as one of the stochastic input variables in the economic analysis platform (further details are provided in the appropriate distribution selection section).

Figure 16 shows an actual route and detour for a generic gravel road. Based on site surveying during construction and maintenance, it was observed that since vehicles usually tried to change their routes when they encountered the construction sign, a detour was considered to be an alternative route during maintenance. According to Omni drive time calculator (46), since driving time for a 1-mi road would be 2 min, assuming a 30-mph speed, as shown in Figure 16, assuming 6 min (three times more than regular route) for driving through a detour would be reasonable. Unless otherwise signed, the Code of Iowa sets the speed limit on rural gravel roads at no faster than 55 mph between sunrise and sunset and no more than 50 mph between sunset and sunrise.

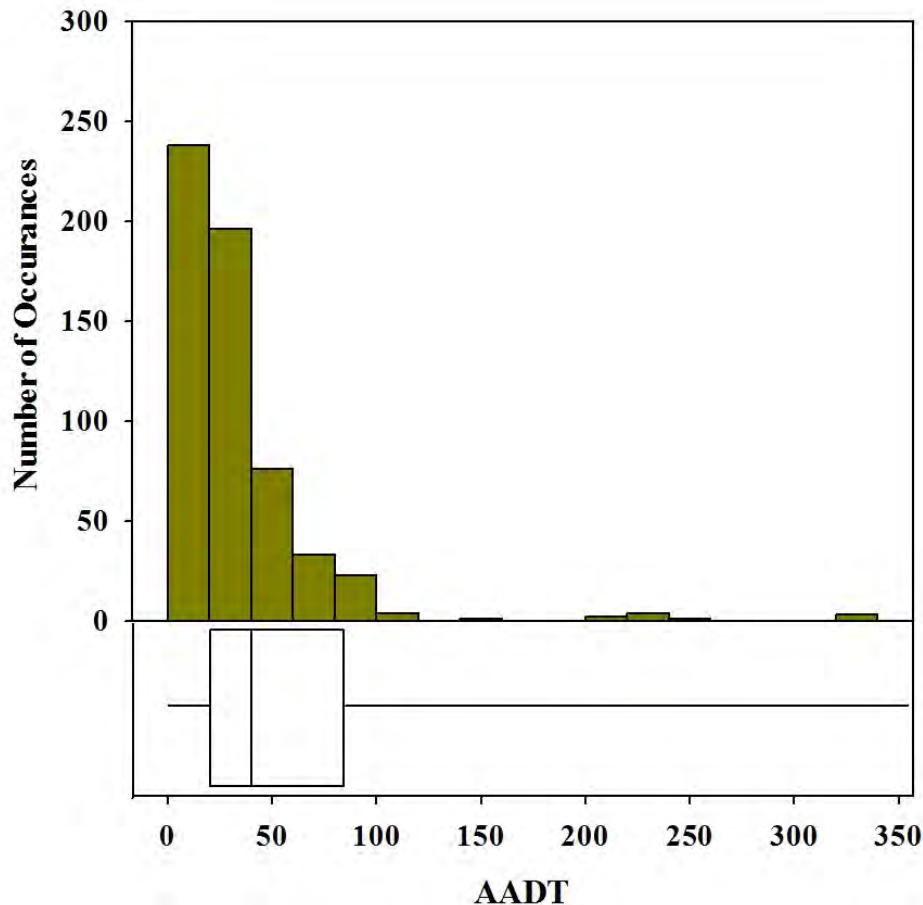


FIGURE 15 Traffic volume distribution in the state of Iowa.



FIGURE 16 Actual route versus detour.

Discount Rate and Service Life

Similar to other studies on pavement management, analysis period was taken as the projected number of years until either the final disposal of the road or the removal of the road materials was required (47, 48). In this study, since all the demonstration sections were placed on a local road with the same amount of traffic load and weather conditions, the same amount of service life was assumed have for all the sections. Since a FHWA life-cycle costs analysis bulletin recommends treating common variable among alternatives with the same value deterministically (20), an analysis period of 20 years was used for all the road sections (49).

Discount rate data for the previous 20 years were obtained from a Federal Reserve data base (50) and added to the stochastic economic analysis model.

Appropriate Distribution Selection

“Determining the appropriate probability distribution for each input variable is an important step in stochastic economic analysis” (51). In this paper, inputs associated with availability of sufficient historical data (i.e., discount rate and average daily traffic) were fitted to a distribution using a maximum likelihood method. To determine which distribution had the best fit, a chi-squared goodness-of-fit test (52), which is often used in business decision-making (53–55), was used (52). In addition, for other input variables with insufficient data availability (e.g., maintenance frequency), a triangular distribution was used, conforming to a common method for describing the distribution of such variables (56–59).

ECONOMIC ANALYSIS RESULTS

To perform stochastic BCA, commercial simulation software (@Risk) was used to develop a Monte Carlo simulation-based (MCS) economic analysis model. MCS were run with each simulation iterated 10,000 times, each iteration lasting from 35 to 75 s. Figure 17 shows the results of simulation for performance-based economic analysis based on dust production measurements.

Figure 17 presents simulation results in probability density functions format, which shows relative likelihood of BCR for each granular road test section. In addition, standard deviation values, “indicators of the amount of dispersions” of BCR (52), along with median values, were shown. “The median is a good measure because, regardless of distribution shape, half the values are above the median and half are below the median” (60).

As shown in Figure 17, among all the alternatives, Section 3 (BFL Class A) yields the highest median BCR. In addition, this section is the only granular road option that met 100% reliability, a probability that BCR stays above 1, which means that use of the aforementioned section would be a more favorable economic investment compared to use of a conventional aggregate option under all the conditions assumed in this study. In addition, although using Section 1 (LCF Class A) is also a secure economic investment with 100% chance of getting BCR above one, since this section has a lower median value than Section 3 (BFL Class A), the chance of yielding more benefits using Section 1 (LCF Class A) would be lower than for Section 3 (BFL Class A).

Section 6 (70% BFL Class A + 30% LCF clean) and Section 2 (OFD Class A) also had high reliability percentages (96% and 86%, respectively), making them good aggregate options for graveling low-volume roads in the state of Iowa. Among all the aggregate options, Section 5 (70% BFL Class A + 30% OFD clean) with 1.02 and 54% median BCR value and reliability, respectively, would be the worst economic investment.

Like stochastic economic analysis developed for the dustometer, point estimates in the deterministic BCR model based on IRI results were replaced with probability distributions and the output estimated in a quantity-variation format. Figure 18 shows the outcome of simulations for performance-based BCA developed for IRI results. There were 10,000 iterations, with simulation times ranging from 37 to 73 s.

The outcomes of stochastic economic analysis based on IRI measurements were close to the previous analysis based on dust production. Note that for Section 3 (BFL Class A), Section 4 (80% BFL Class A + 20% BFL clean), Section 5 (70% BFL Class A + 30% OFD clean), and Section 7 (70% BFL Class A + 30% CRG clean) the results were identical, because for all these aggregate products identical maintenance frequency scenarios were developed (Figure 11 and Figure 13).

As shown in Figure 18, as in the previous analysis based on dust production, among all of the alternatives Section 3 (BFL Class A) exhibits the highest median BCR and percentage reliability. However, because different aggregate options exhibited less difference in terms of roughness than for dust production, fewer scenarios were developed for IRI based economic analysis (i.e., two scenarios based on IRI and three for dust production). Therefore, median values of BCR based on IRI results were the same or lower than for dust production performance-based analysis. In general, BFL Class A along with Section 1 (LCF Class A) and Section 4 (80% BFL Class A + 20% BFL clean) exhibited the best economic performance among other alternatives [comparing with the base case aggregate option, Section 4 (80% BFL Class A + 20% BFL clean)].

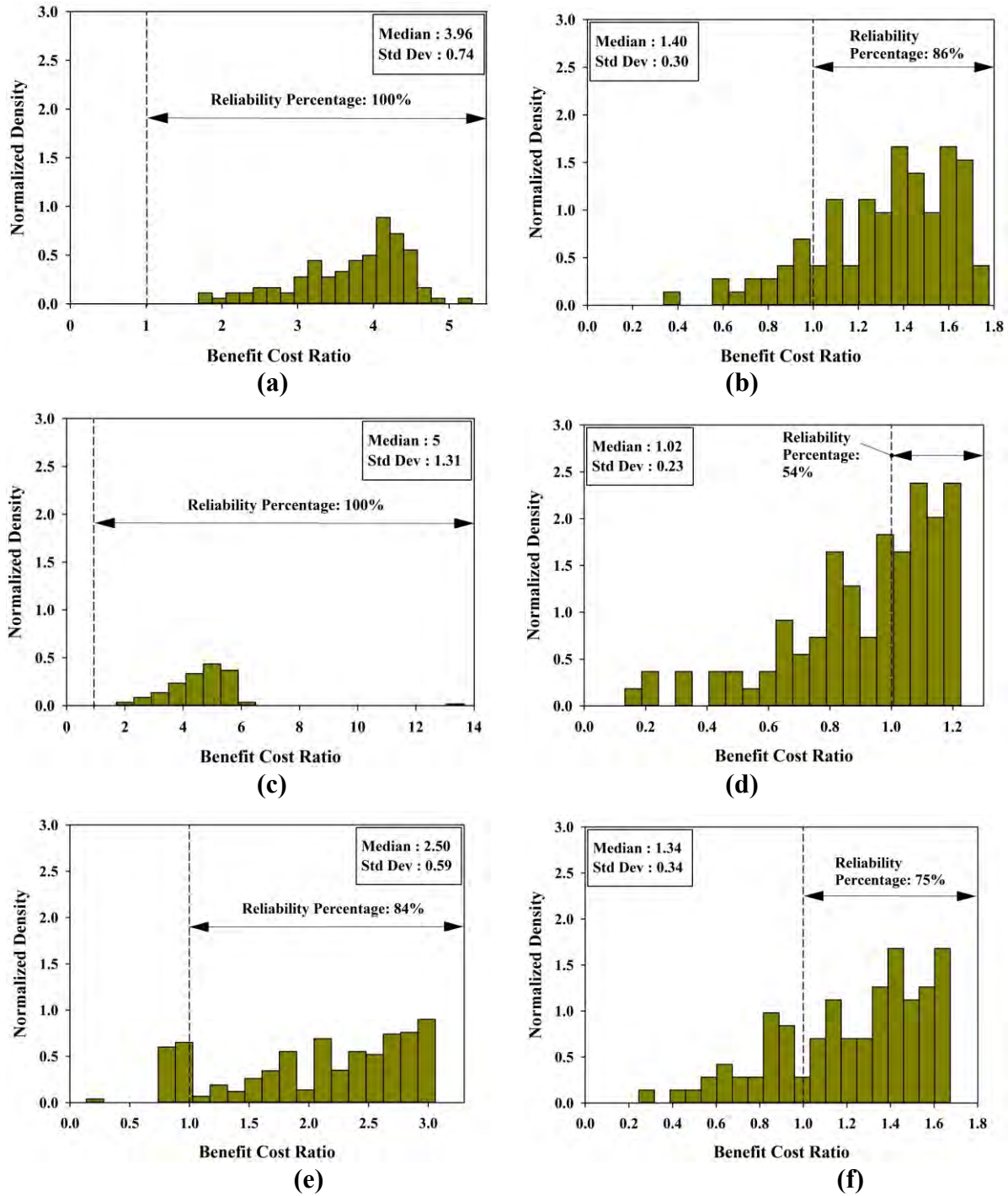


FIGURE 17 MCS results (scenarios developed based on dust production measurements):
(a) Section 1, LCF Class A; **(b)** Section 2, OFD Class A; **(c)** Section 3, BFL Class A; **(d)** Section 5, 70% BFL Class A + 30% OFD clean; **(e)** Section 6, 70% BFL Class A + 30% LCF clean; and **(f)** Section 7, 70% BFL Class A + 30% CRG clean.

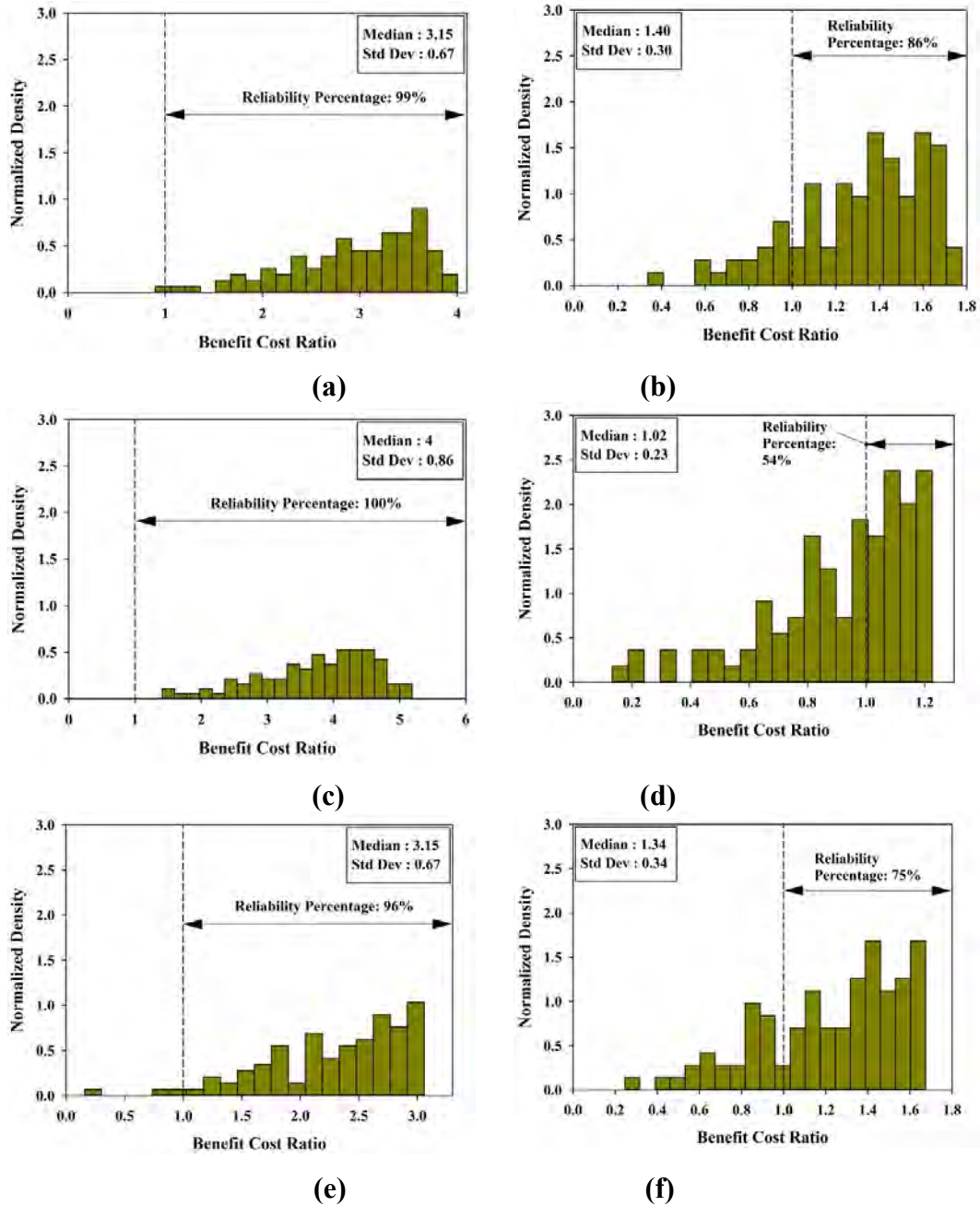


FIGURE 18 MCS results (scenarios developed based on IRI results): (a) Section 1, LCF Class A; (b) Section 2, OFD Class A; (c) Section 3, BFL Class A; (d) Section 5, 70% BFL Class A + 30% OFD clean; (e) Section 6, 70% BFL Class A + 30% LCF clean; (f) Section 7, 70% BFL Class A + 30% CRG clean.

CONCLUSIONS

This paper presents a stochastic performance-based LCBCA method for comparing economic performance of granular roads constructed in a rural road system. Seven different surface aggregate materials were obtained from four different quarries in Iowa and different performance measure tests were conducted over the last 3 years. Using LWD, IRI, and dustometer test results, scenarios were developed for maintenance frequency of test sections to be used in the economic analysis framework. This study used the IRI and dust production results as performance measures for estimating maintenance frequency. In addition, among alternatives, BFL Class A, compared to the base case aggregate option (Section 4, 80% BFL Class A + 20 % BFL clean), exhibited the best economic performance. These findings could help agencies, county engineers, and contractors in estimating the most beneficial material alternatives in terms of lower costs of hauling, material, labor, and equipment for construction and maintenance of granular roads.

The methodology developed in this study provides agencies with the probability that a preferred alternative can produce the lowest life-cycle cost. Recommendations that may result from this research project would be founded in fundamental economic analysis theory and can also provide various transportation agencies with an added level of confidence in predicting economic impact associated with granular road material alternatives of interest.

ACKNOWLEDGMENTS

The authors gratefully acknowledge sponsorship for this research study from the Iowa Highway Research Board (IHRB) and the Iowa DOT. The contents of this paper reflect the views of the authors who are responsible for the facts and accuracy of the data presented within. The contents do not necessarily reflect the official views and policies of the IHRB and Iowa State University. This paper does not constitute a standard, specification, or regulation.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: Study Conception and Design: C. Jahren, S. Satvati, and A. Nahvi; Data Collection: S. Satvati and A. Nahvi; Analysis and Interpretation of Results: S. Satvati and A. Nahvi; Draft Manuscript Preparation: S. Satvati and A. Nahvi. All authors reviewed the results and approved the final version of the manuscript.

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A Multi-Criteria Evaluation Approach for Prioritization of Low-Volume Roads for Maintenance and Improvement

S. SHANKAR

B.L.V.S.S. PHANEENDRA

National Institute of Technology

Roads play a significant role in the socioeconomic development of any country. For achieving these benefits, Governments have started many programs. Because of these recent programs, a vast Low-Volume Roads (LVRs) network was created in India which resulted in increased social and economic benefits all around. However, due to increased axle loads, lack of adequate and timely maintenance, it is bound to accelerate the process of deterioration of these roads. Because of this, the failures of LVRs have been predominant in countries like India. This has direct effects on both road agencies and road users and leads to loss of time, agriculture output, access, higher vehicle operating cost, increasing number of accidents, etc. In developing countries, the financial resources are minimal, so there should be a scientific and efficient method required to rank and prioritize the LVR projects, with minimum resources and maximum benefits by a scientific approach. To address this task, the multi-criteria concept was considered. The multi-criteria concept includes parameters like social, economic, and pavement condition as the main criteria. In this main criterion, other essential parameters were considered as a sub-criterion. Next task is to find the best suitable parameters and their weights through an expert's opinion survey. In this study, an expert opinion survey was conducted using the Delphi Technique (DT), Likert scale, pairwise comparison and ranking methods. Based on expert opinion survey results, two criteria systems were identified. These two criteria systems were suitable for different circumstances. Finally, the study proposed the maintenance criterion considering the socioeconomic, environmental, and pavement condition parameters for adequate maintenance of LVRs.

INTRODUCTION

LVRs play a vital role in the development of any country. Thus, the government of India introduces many programs to increase the LVR network. Some programs like the Pradhan Mantri Gram Sadak Yojana (PMGSY), basic minimum service program, and Minimum Needs Program have been already implemented in India. Due to this, a vast LVR network has been created. In India, the LVR network represents approximately 90% of the total road network. The increased LVR network has resulted in increased economic and social benefits. These LVRs provide the main links to the national highway transportation system. LVRs provide links from forests and mines to mills and provide links from farms and homes to markets, and they provide public access to essential education, administrative, health and other essential services. The LVR link between markets and raw materials is critical to economies nationally and locally in all countries around the world (1, 2).

However, with time, these benefits would decrease due to lack of maintenance. Due to poor maintenance, increasing axle loads, and changes in environmental conditions, the roads will deteriorate very quickly and reach the permissible limit of acceptable road condition very early. Excessive pavement deterioration leads to loss of time, loss of access, reduced agricultural output, higher travel time costs (TTC), higher vehicle operating costs (VOC), and increasing accidents, etc. Forgetting these benefits long-term, there is a need to do something different. One of the best solutions is regular maintenance of the roads. However, also, the pavement maintenance slows the pavement deterioration process and extends the pavement life. So, there is a need to focus on the maintenance of LVRs (3). However, the road network of India is the second largest road network in the world and government providing funds for maintenance is very limited. How to utilize these funds properly? The solution to this problem needs to have prioritization. The priority of road selection plays a significant role in determining which road will be maintained first. Prioritization analysis is a multi-criteria process that determines the best ranking list from a total list provided. Economic, technical, social and environmental aspects are crucial for prioritization of pavement maintenance. Prioritization of LVRs for maintenance and improvements depends on the present pavement condition, social parameters, economic parameters and environmental aspects (4–7).

A total of four links are seen in Figure 1. Now the task is to do prioritization for maintenance. How? Suppose consider a link 1, link 2 and assume that link one pavement condition is poor and link 2 pavement conditions also poor but economic benefits from link 1 is very less and economic benefits from link two very high and social benefits link one is high and from link two less. So, it very difficult to say one-first preference for maintenance. Multiple criteria should be considered for prioritization. In this, all four parameters are considered. However, what are the most suitable criteria's and their weights for maintenance and improvement? One of the best solutions for finding the most suitable criteria and their weights is an expert's opinion survey by using the DT (8). Network planning utilities are calculated by using the DT. The Delphi method was originated in the 1950s. Developing this method, RAND Corporation conducted a series of studies. The primary objective of the study was to develop a technique to obtain the most reliable consensus of group experts. The DT is designed as a group communication process that aims at conducting detailed examinations and discussions of a specific issue for goal setting, policy investigation, or predicting the occurrence of future events (9).

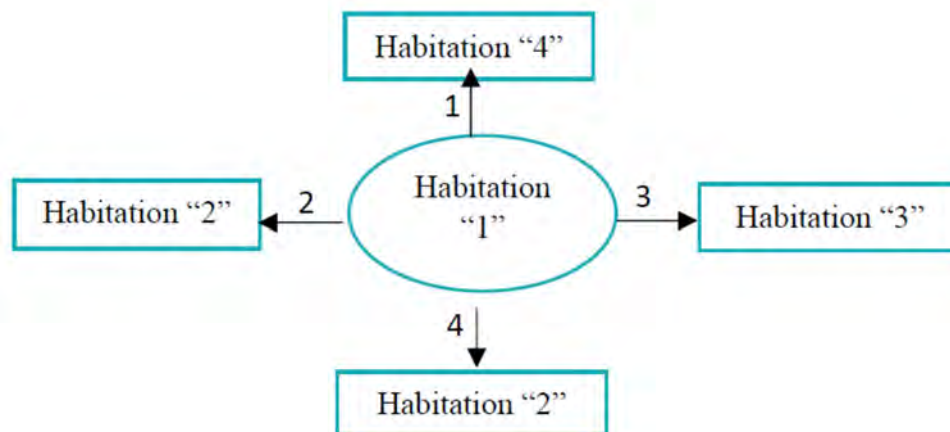


FIGURE 1 Why and how multiple criteria should be considered for prioritization.

Priority of rankings is given to select road sections based on two methods. The two methods are subjective rating method and on economic indicator method. Subjective rating method is done based on Maintenance Priority Index (MPI). MPI is a function of traffic volume, road condition index and drainage factor. In an economic indicator, the Present Net Value (NPV) is calculated using Highway Development and Management (HDM-4) tool and comparison are made between two methods (10). The optimum maintenance and rehabilitation strategy were obtained by maximum NPV–CAP value, which shows the section having a maximum NPV–CAP value would be maintained on a priority basis as compared to other sections (11). The MPI of road sections and economical maintenance alternative based on net present value and internal rate of return value (12).

A facility model was developed for selecting the roads for upgrading based on link weight and pavement condition index. These link weights were calculated by considering the population of villages and facilities like education, health, economic activity centers, transportation and communication centers etc. (15). Pre-determined criteria are social, economic and technical aspects. Points were given for the number of villages served by road, health centers, schools, industries, markets, and areas of high agricultural potential Scores were given for the “passability” of the road, availability of laterite surfacing material (the requirement for structural sections), difficult sections (e.g., swamps/rocky terrain) and availability of labor. The totals were summed and then divided by the road lengths to give an average ranking score (16). Multi-criteria prioritization model is based on the novel set of factors like growth centers, road utilization, connectivity, accessibility, backwardness, and the number of commercial vehicles using the road in order to rank roads to be selected for improvement. The survey among experts formulated the weight of each of the index used in a composite index calculation by using analytical hierarchy process (AHP). Composite Index (CI) was calculated based on the weighted sum method, and the roads were ranked (17). Growth priority index, connectivity index, accessibility index, backwardness index, a road utilization index, commercial vehicle density index, environmental and social index were developed. A CI derived from individual indices and selected roads for improvement are based upon the CI (18). Prioritization of pavement is done based on the multiple distresses by using an AHP. AHP is found to be one of the most straightforward and most suitable methods in multi-criteria decision-making processes. This method is based on pairwise comparisons which facilitate the judgments and calculations (18).

METHODOLOGY

The objective of this study is to find the most suitable criteria system and their weights for prioritization of LVRs for maintenance and improvement. Then the implementation of the criteria system is used for an existing LVR network. For this purpose, an expert opinion survey was done by using the DT and afterwards multi-criteria analysis was used for prioritization. The survey was conducted through e-mails and Google forms. The first-round survey was prepared by using a Likert scale and the scale adopted is from “not important” to “very important.” In the second-round survey for finding the weights, two methods were used. A pairwise comparison method was used for finding weights of main criteria, and a ranking method was used for finding sub-criteria weights (Figure 2).

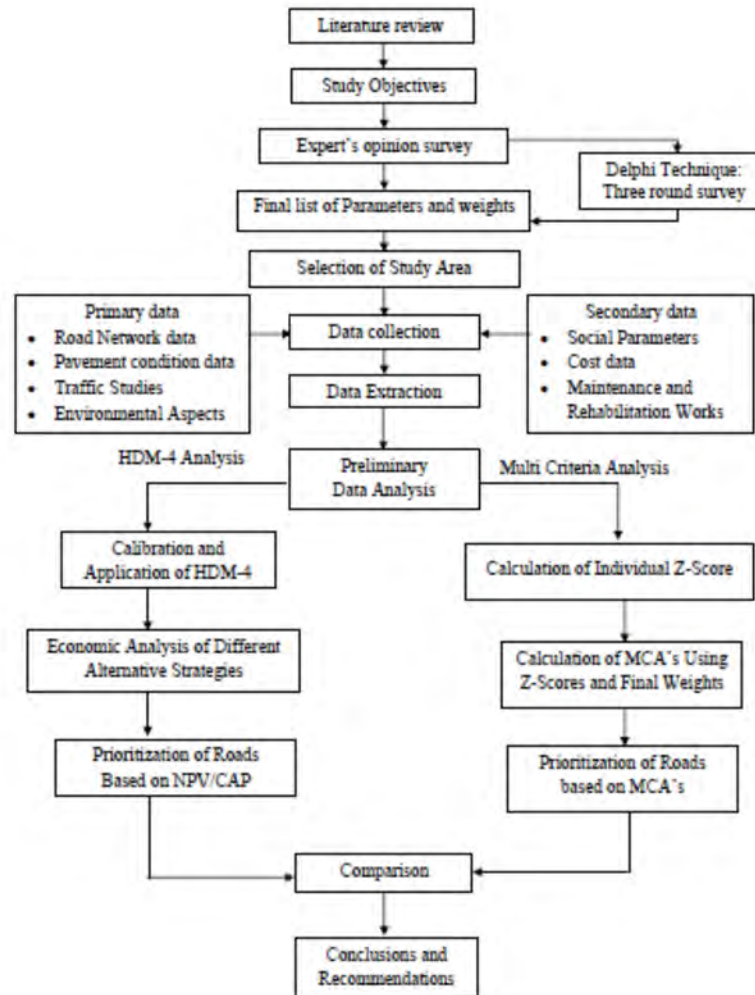


FIGURE 2 Adopted study methodology.

DEVELOPMENT OF PRIORITIZATION CRITERIA AND THEIR WEIGHTS BY USING EXPERT OPINION SURVEY

The initial list of the potential criterion was selected based on a literature review and through surveys with local village people, field engineers, and experts in the area of transportation engineering. Accordingly, the initial list of criteria and sub-criteria is prepared as presented in Table 1 and Table 2, respectively.

The final selection of LVRs evaluation criteria and their relative weights will be determined by applying the Likert scale method, Pairwise comparison and ranking methods are using the DT, based on the outcome of an expert's opinion survey conducted among the different academicians, policy makers, field engineers, and other stakeholders.

TABLE 1 Adopted Initial List of Criteria

Initial List of Criteria	Local village people
	Field engineers and research students
	Politicians

TABLE 2 Initial List of Main Criteria and Sub-Criteria for Prioritization of LVRs

Main Criteria	Sub-Criteria
Social parameters (SP)	1. Population served per km on road (PSK)
	2. Access to educational services (AES)
	3. Access to market centers (AMC)
	4. Access to administrative and health (AOS)
	5. Access to other roads (NH, SH, MDR, ODR, and VR)
	6. Road as a community priority (RCP)
Pavement condition (PC)	1. Pavement condition distress index (PCDI)
	2. Pavement condition roughness index PCRI)
	3. Pavement condition skid resistance index (PCSR)
	4. Pavement condition structural capacity index (PCSCI)
Economic parameters (EP)	1. Road construction cost (CC)
	2. Road maintenance cost (MC)
	3. VOC savings
	4. TTC savings
	5. Accident cost (AC)
	6. Pollution cost (PC)
Environmental aspects (EA)	1. Duration of submergence (DS)
	2. The possibility of landslide or flooding (PLF)
	3. The extent of positive impact on the environment (INS)

Round 1 Survey

The first-round survey aims to finalise the final list of criteria. For this purpose, the facilitator prepared the set of questionnaires by using the Likert scale and taking five significant levels from not important to very important. These questionnaire forms were then sent to experts via e-mail and Google forms. The facilitator received a total of 43 responses. Then the facilitator analyzed significant levels given by panel members, and the results are shown in Figure 3 and Figure 4. The questionnaire format is shown in Appendix A.

Overall, the social parameters criterion was considered very important or essential (37 members). Almost all members considered sub-criteria in social parameters as very important or essential. Overall, the pavement condition criterion was considered very important or essential (39 members). Almost all members considered roughness, distress, the structural capacity index

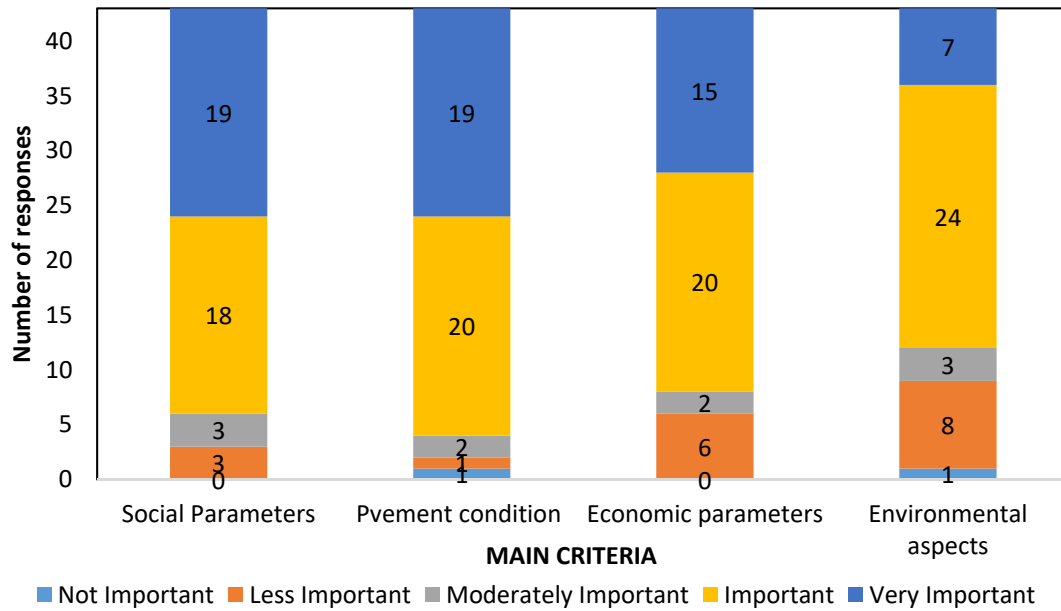


FIGURE 3 First-round survey analysis of main criteria.

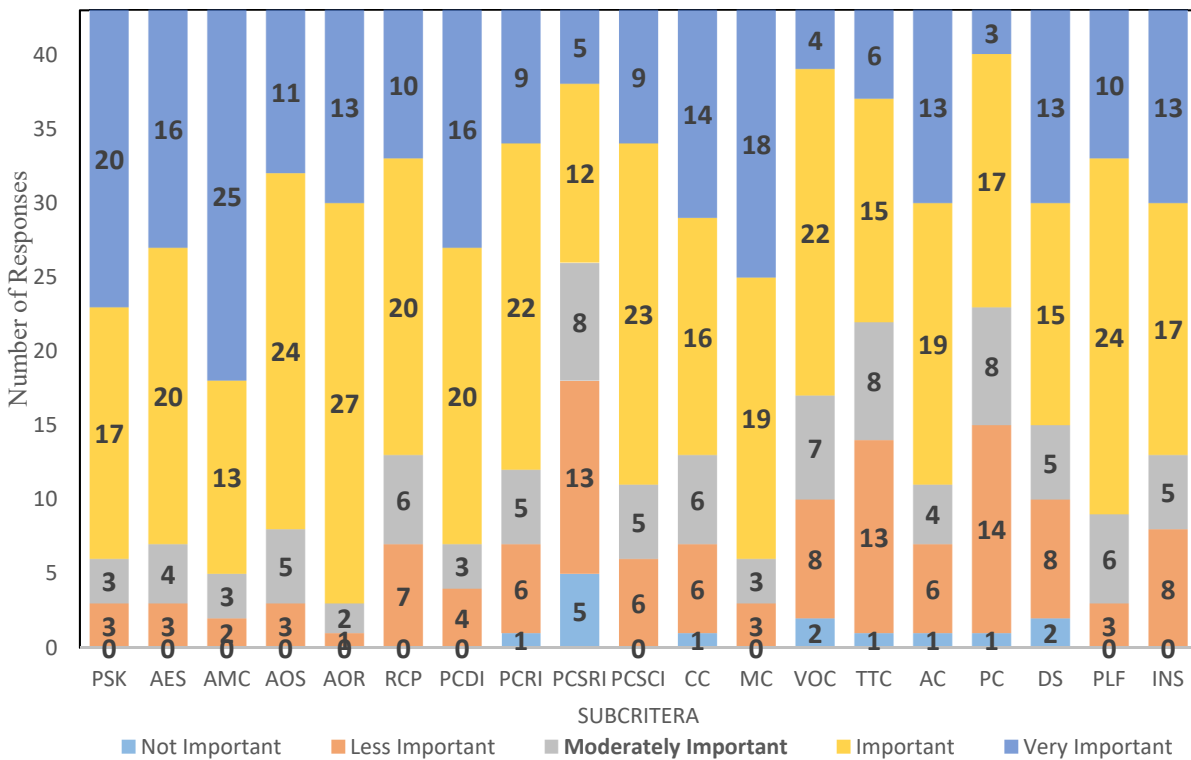


FIGURE 4 First-round survey analysis of sub-criteria.

as very important or important. Forty-two percent of members considered resistance index as less important than moderately important. Compared to the social, the members gave pavement condition less importance than economic parameters. Overall the economic parameters criteria were considered very important to important. In this, pollution cost is considered less important compare to the remaining sub-criteria, but still moderately important or higher. Compare to the remaining three main criteria; the environment is less important for prioritization. However, 31 members considered that this criterion was important or very important. Overall, all the parameters are important or very important. Based on comments of panel members and their opinions, the final list of criteria is prepared and presented in Table 2. After reviewing the first-round survey results, comments and previous studies on prioritization of LVRs, it is observed that economic parameters depend on pavement condition parameters. Hence, felt that it is not possible to include both parameters in single criterion. The author proposed two alternative criteria namely Criteria A and B. (Criteria A and B include social, pavement condition, environment parameters and Criteria 'B' includes, social economic and environmental parameters.

Round 2 Survey

The primary objective of the second-round survey is to find the weight criteria of each parameter. A weight criterion indicates the relative importance of one criterion compared to all other criteria. The facilitator prepared the questionnaires for the round survey by using a pairwise comparison method for the main criteria and a ranking method for sub-criteria. The questionnaire forms were then sent to experts. A total of 45 members participated in the second-round survey. Based on the responses, the facilitator calculated the weighting of all the criteria. For the main criteria, one pairwise comparison table was prepared for each respondent. The sub-criteria weights were calculated based on ranks given by the panel members. Next task was to aggregate the individual data. Based on literature research, the facilitator aggregated both main criteria and main criteria weights by using the geometric mean method. The final weights of all the sub-criteria, also called global weights, are calculated by multiplying the sub-criteria weights by the corresponding main criterion weights. Weights after second-round survey not shown here. Final weights are shown after a third-round survey. The questionnaire format is shown in Appendix B.

Round 3 Survey

This step is one of the main strengths of the DT. The main aim of the third-round survey is to reduce the differences among the group member opinions on the criteria weights and facilitate a consensus. In this round, the facilitator presents the panel members with a comparison with the average group response to their original responses. After that, they are asked if they are willing to revise their original responses or not. In the third-round survey, the panel members provided their revised weights using a 100-point scale method. The facilitator converts all the responses directly into weights. Some panel members did not respond in the third-round survey, so the author assumed that their second-round responses remained unrevised. The facilitator analyzed the second and third-round criteria weights by using "box-and-whisker" plots. The primary purpose of these boxes plot was to find the variation in the set of weights given by experts. After analyzing total results using different pie charts, bar charts, and box plots, the facilitator finalises the final weights of each criterion. Final criteria weights are shown in Figure 5 and Figure 6.

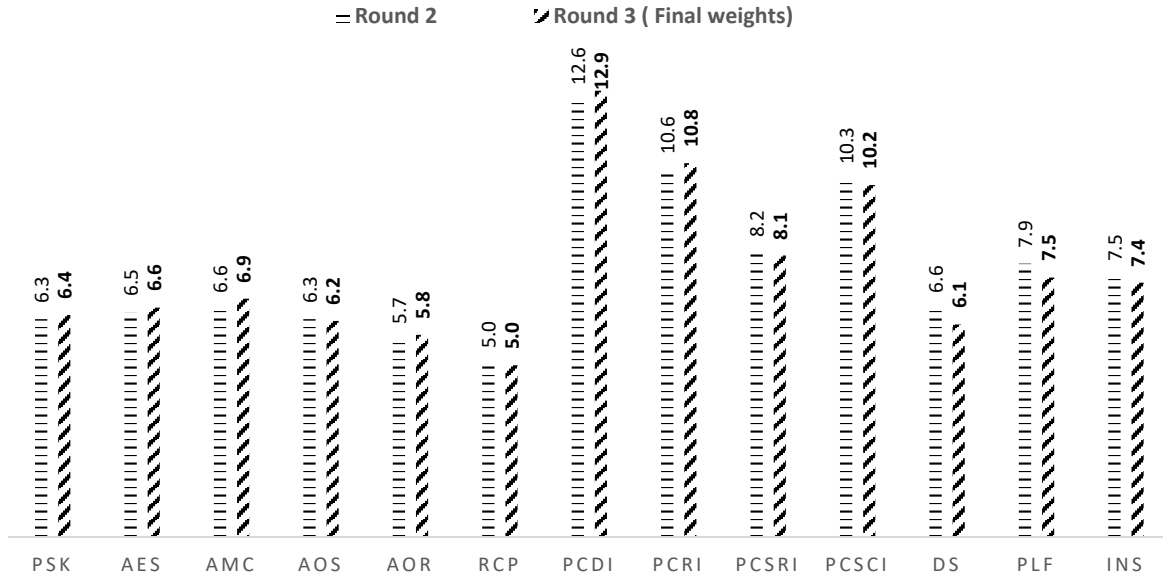


FIGURE 5 Criteria A global weights variation.

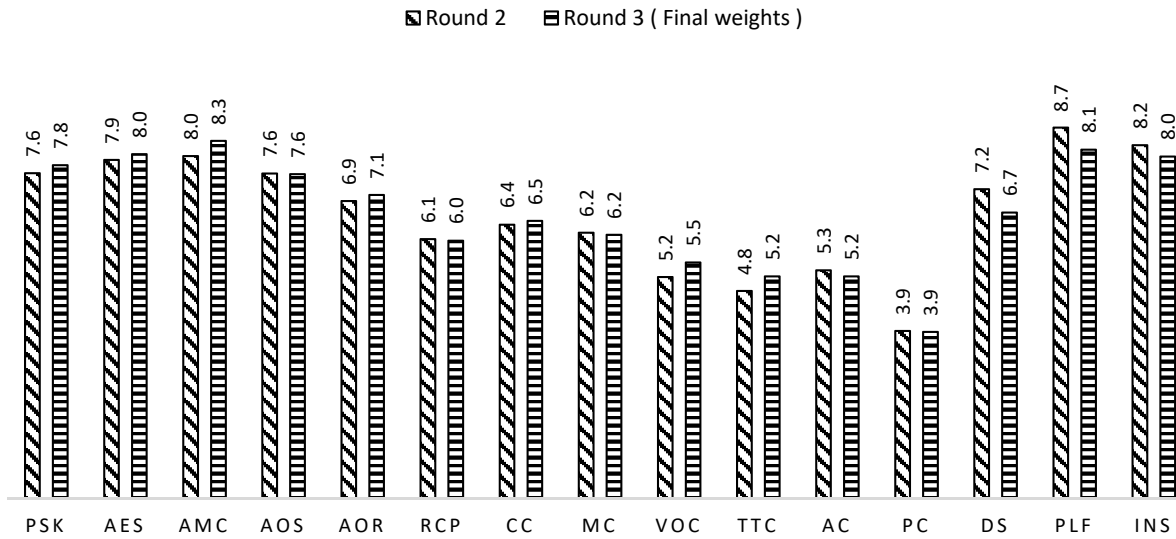


FIGURE 6 Criteria B global weights variation.

Major Observations After Third-Round Survey

1. In Criteria A the pavement condition criterion has the highest weight. Only pavement condition covers around 42% total weight.
2. In this pavement condition distress, roughness covers more than 50% of total pavement condition weight.
3. Social parameters cover about 37% of the total weight. In these social parameters, access to market centers has the highest weight. The least weight was environmental aspects.
4. In Criteria B the social parameter criterion has the highest weight. Only social parameter covers about 45% of the total weight. In this, access to market centers also covers more weight.
5. Economic parameters cover about 32% of the total weight. In this construction cost has the highest weight and maintenance cost occupies the next highest place.
6. Overall if we consider global weights, pavement condition distress index has the highest weight in Criteria A; access to market centers criteria has the highest weight in Criteria B.
7. The global weights of sub-criteria may vary depending on the number of sub-criteria taken under each main criterion.

Variations in Criteria Weights: Comparison Between Second- and Third-Round Survey

The facilitator analyzed the second- and third-round criteria weights by using box-and-whisker plots. The primary purpose of these box plots is to find the variation in the set of weights given by experts. In these plots, the average weight, the interquartile range, which implies the value between 25th percentile to 75th percentile value, and the maximum and minimum values were plotted. The interquartile ranges are shown in boxes, and the lower 25% and upper 75% of the weights are shown as lines extending from the boxes. Figure 7 shows the box-and-whisker plot for the main criteria weights. The facilitator also drew box-and-whisker plots for every sub-criterion and analyzed the results. This plot shows the variation of criteria weights in the second and third-round survey and comparisons. The facilitator aims to reduce the size of boxes and total range between the second- and third-round surveys. Figure 7 clearly shows that the interquartile ranges decrease after the second-round survey.

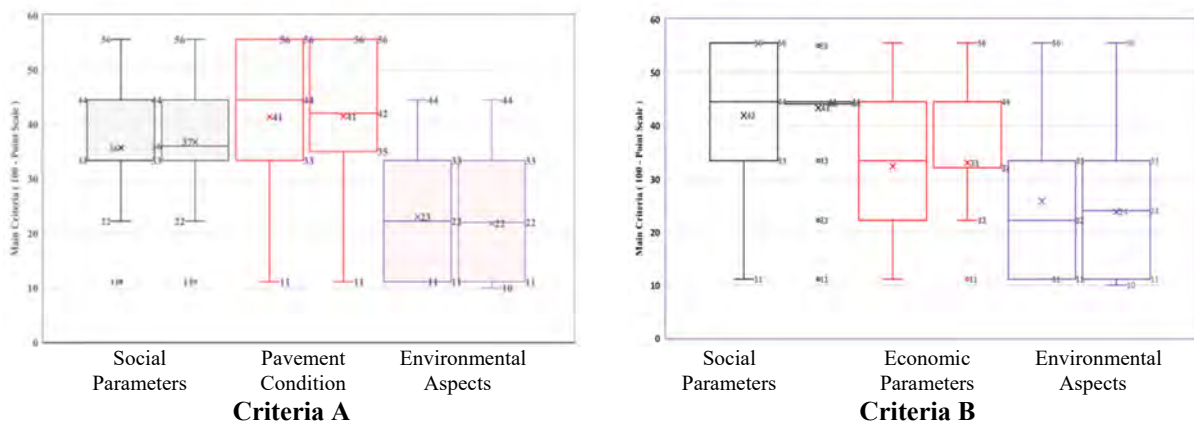


FIGURE 7 Variation of main criteria weights.

In Table 3, the weights are varying after the third-round survey. Round 3 results are final weights. The next task is to implement these weights to existing LVR networks and prioritize the LVR network for maintenance and improvements.

FINAL SCORES CALCULATION BY USING MULTI-CRITERIA ANALYSIS WITH Z SCORE AND GLOBAL WEIGHTS

Final scores (FS) are required to prioritize road sections by using multi-criteria.

$$FS = \sum (w_i \times x_i) \quad (1)$$

where

FS = final score;
 w_i = relative weight of each criteria; and
 x_i = value of Criteria i .

In this multi-criteria analysis, the criteria considered are financial parameters, other non-monetary parameters, and different units, and scales. It is challenging to bring all the parameters in a single scale for finding FS. So, in place x_i , can be used. z_i = normalized score:

$$FS = \sum (w_i \times z_i) \quad (2)$$

$$Z = \frac{X - \mu}{\sigma} \quad (3)$$

where

Z = normalized score;
 X = value corresponding parameter;
 μ = overall mean of all the parameters; and
 σ = standard deviation of all the parameters.

After completion of the score calculation, one has to calculate the final score.

Their FS will determine the prioritization order for maintenance and improvement of candidate LVRs. For finding the final score, a small tool was developed by using Java coding and HTML coding. After giving all the inputs, it will directly give the maintenance priority.

TABLE 3 Variation of Main Criteria Weights between Second and Third Rounds

Criteria A	Round 2	Round 3
SP	36	37
PC	42	42
EA	22	21

Criteria B	Round 2	Round 3
SP	44	45
EP	32	32
EA	24	23

CASE STUDY

Application of the multi-criteria evaluation for the selection of LVR projects was done in Warangal district. In Warangal district, Hanmakonda Mandal was selected for this study. Hanmakonda Mandal headquarters is in Hanmakonda Town. It consists of 32 villages and 24 panchayats. Ammavaripet is the smallest village and Paidipally is the biggest village in the Mandal. The total population of Hanamakonda Mandal is 87,503 living in 19,474 houses, spread across a total of 32 villages and 24 panchayats. In Hanamakonda, the summer highest day temperature range is between 33°C to 46°C. A total of eight road sections were selected for prioritization.

Consolidated Data for Prioritization

Social Parameters

In this criterion, the population served per kilometer was directly adopted such as access to educational services, market centers, access to other services, and access to other roads. Marking was given based on Panchayat Raj (name of local engineering department) engineer's opinion. Table 4 gives the consolidated social parameter data per low-volume road section.

Pavement Condition

Pavement condition parameters calculated using field studies data and equations provided by different authors (Table 5).

TABLE 4 Consolidated Social Parameters Data

	Parameters	LVR1	LVR2	LVR3	LVR4	LVR5	LVR6	LVR7	LVR8
1	Population served per km	377	2163	453	9,908	1,628	1,090	4,454	3,108
2	Access to educational services	1.3	3.8	1.3	6.3	3.8	1.3	10.8	3.8
3	Access to market centers	00	00	00	3.5	1.5	00	00	3.5
4	Access to other services (administrative and health centers)	00	4.7	00	4.7	13.5	4.7	13.5	2.5
5	Access to other roads (NH, SH, MDR, ODR, and VR)	04	4	02	4.8	02	02	2.6	3.6
6	Road as a community priority	05	05	05	05	05	05	05	05

TABLE 5 Consolidated Pavement Condition Data

	Parameters	LVR1	LVR2	LVR3	LVR4	LVR5	LVR6	LVR7	LVR8
1	PC–distress index	41	65	72	54	86	91	68	72
2	PC–roughness index	46	55	41	57	53	52	47	44
3	PC–skid resistance index	96	82	68	108	109	105	98	120
4	PC–structural capacity index	1.63	1.56	1.20	1.16	1.29	1.29	0.88	0.92

TABLE 8 Priority of Ranking

Criteria A			Criteria B		
Rank	Section ID	Overall Score	Rank	Section ID	Overall Score
1	LVR7	44.5	1	LVR2	66.54
2	LVR4	39.5	2	LVR7	52.28
3	LVR8	25.3	3	LVR4	43.88
4	LVR3	8.9	4	LVR8	41.65
5	LVR1	4.3	5	LVR1	-35.53
6	LVR2	-8.3	6	LVR5	-47.02
7	LVR5	-56.3	7	LVR3	-57.37
8	LVR6	-57.9	8	LVR6	-64.43

sections. In Criteria B, LVR2 got the highest priority. The reason for this is that traffic is more and savings from this section are also more. In both criteria, the least preferable is LVR6. The reason for this is that LVR6-Vkp was constructed recently so the pavement condition is good and the socioeconomic benefits from this section are also less. The ranking priority is varying based on the criteria used.

SUMMARY AND CONCLUSIONS

The author established two different criteria systems for the prioritization of LVRs for maintenance and improvements based on the expert's opinion survey using the Delphi method. The study showed that the DT is the most useful method for developing weight-age of criteria selected for prioritization. The expert opinion surveys indicated that the panel members differ in their opinions regarding the criteria weights. Although they move closer to a consensus opinion in a third-round survey, still some differences were observed. If more rounds are conducted, the results would have been more satisfactory.

A comparison was made for the priority ranking given for two different criteria systems. The results were inconsistent, but two sections results were comparable. Based on observation, Criteria A was most suitable for conditions like no budget constraints and when less time was available for doing projects. For Criteria A, data requirements were less. With budget constraints, Criteria B is most suitable. For doing economic analysis with many data required, Criteria B is more complicated, and collecting that much data required more time. For developing countries like India, these criteria systems are most useful. This is a general study for prioritization of LVRs for maintenance and improvement. Based on this study a set of criteria shall be developed for new construction, rehabilitation, and upgrading of roads individually based on local conditions, some additional criteria shall also be included.

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APPENDIX C: ROUND 3 SURVEY QUESTIONNAIRE FORMAT SAMPLE

Social Parameters

Below table shows the sub-criteria weights for the social parameters criterion.

Summary of sub criteria weights

Sub Criteria	Your Weights	Overall Average	Your Revision
Population Served per KM of a Road	14	17	
Access to Educational services	17	18	
Access to Market Centers	17	18	
Access to other services (Administrative, Health services)	17	17	
Access to other Roads (NH, SH, MDR, ODR & VR)	17	16	
Road as A community Priority	17	14	
Total	100	100	100

After seeing about this group response, would you like to revise your opinion and update your weights to come closer to the average weights?

If so, please fill the below questions with your revised weights on 100-point scale.

If you want to adopt the overall average weights as your revised weights, please choose YES option and don't Answer below question.

YES

APPENDIX D: PROGRAM FOR FINDING FINAL SCORES

```

1 <!DOCTYPE html>
2 <html>
3 <head>
4 </head>
5 <body>
6 <form method="post" action="cityForm.jsp">
7 <table>
8 <tr>
9 <td><p>Enter the no. of road sections </p></td>
10 <td><input type="number" name="cities"></td>
11 </tr>
12 <tr>
13 <td><p>Enter the no. of Parameters</p></td>
14 <td><input type="number" name="param"></td>
15 </tr>
16 <tr><td><input type="submit" value="submit"></td></tr>
17 </table>
18 </form>
19 </body>
20 </html>
    
```

Enter the no. of road sections

Enter the no. of Parameters

welcome \$	P1	P2	P3
LVR1	377	1.3	0
LVR2	2163	3.8	0
LVR3	453	1.3	0
LVR4	9908	6.3	3.5
LVR5	1628	3.8	1.5
LVR6	1090	1.3	0
LVR7	4454	10.8	0
LVR8	3108	3.8	3.5
Weightage	6.4	6.6	6.9
submit			

Influence of Lane Width on Semi-Autonomous Vehicle Performance

ALFREDO GARCÍA

Universitat Politècnica de València

INTRODUCTION

In a medium term, there will be more and more semi-autonomous vehicles before the final general deployment of fully autonomous vehicles. This change in the vehicle capabilities makes it necessary to analyze their interaction with road infrastructure capabilities, which were developed for human-driven vehicles.

Current systems present digital image processing technology using video cameras that read the oncoming road path and process the information. However, road markings were designed to be interpreted by drivers, not vehicles. Semi-autonomous systems, with Level 2 or higher of automation, use road markings located at the edges of the lane to automatically facilitate keeping the vehicle within the lane. Thus, the lane width is one of the geometric parameters to be assessed.

The objective of this research is to study the impact of lane width and road marking layout on semi-autonomous vehicle automatic lateral control (LKA). The automatic lateral control of one semi-autonomous vehicle was tested along some tangent sections of urban streets with different lane widths and road markings. The main hypothesis is the wider the lane and the more continuity of the road marking, the more automatic the lateral control.

The semi-autonomous system tends to fail on narrow lanes. There is a maximum lane width that always requires human lateral control, named human lane width. It was obtained for lanes 2.5 m wide (8.2 ft). Moreover, there is a minimum lane width that allows automatic lateral control to always maintain the control, named automatic lane width. It was possible for lanes 2.75 m wide. Finally, there is a lane width with the same probability of automatic and human lateral control, named critical lane width (CW). The obtained CW was 2.72 m.

This is a pilot test of the new systems using only one vehicle. Nevertheless, new lane width concepts have been proposed. Further testing with other vehicles and operational domains should be considered to compare results.

A semi-autonomous driving system should be able to take control of the vehicle under a given operational domain, continuously and uninterruptedly. However, these systems still need a driver to monitor them, collaborate in operation, and take control when needed. According to SAE J3016, this corresponds to current automation Levels 2 and 3. The automation levels ranking, established by SAE International, is being adopted by most countries (1). It considers six automation levels, from 0 (complete human driving) to 5 (complete autonomous driving). Current vehicles reach Level 2 and, under certain limited operational domains, Level 3.

Since these systems require continuous human presence and monitoring, a human-machine interface is needed. This system alerts the driver on when their attention and reaction are about to be required. Unfortunately, existing semi-autonomous systems are in a very early development stage, hence presenting frequent control losses and, therefore, annoyance to drivers.

This is especially risky, since it increases drivers' response times (2, 3).

The longitudinal road markings play an important role in autonomous driving. To identify and read them, most of current efforts are based on artificial vision due to the higher cost of other different systems with good results (4). Artificial vision systems must process a large amount of information per second, in real time and with high reliability. For that purpose, the information is first cropped to a certain region of interest, normally based on the instantaneous speed of the vehicle. Every photogram will be further decomposed into matrices, being the HSI system the most general one (5). Each one of these matrices is then processed to detect object boundaries and textures. Classic edge detection algorithms compare adjacent pixels, while ridge-type algorithms use more pixels and give better results (6).

This is a critical process, since road markings might not be in good state, thus leading to unclear zones, or shady areas that are difficult to interpret as a road marking. Cáceres et al. proposed a methodology to overcome this limitation, based on an RGB filtering followed by an intensity clustering (7).

With this information, another algorithm depicts a polyline centered in each road marking, and inverse conic restitution is generally later applied (4). This procedure takes visualized road markings and transforms them to projected coordinates. These coordinates can be used by other algorithms to detect if they are the road edge, centerlines, or another kind of pattern. Besides the correct detection of these lines for every photogram, it is necessary to integrate them temporally, i.e., information provided by consecutive photograms must be consistent (4). In most cases, stochastic methods are used.

There are some additional methodologies to compare the layout of the boundary lines that conform a lane (which should be parallel in nearly all situations). These kinds of methodologies help the system at confusing zones. Du and Tan applied a ridge detection methodology for this purpose (6). They also applied previous filtering processes to enhance the quality of the photograms.

The relative position of the vehicle can be directly determined based on inverse conical restitution, but more advanced systems such as stereoscopic vision and a helping camera are currently under development (4).

The literature review began to establish the scope for this research, as there are no experimental studies of the impact of the actual lane width in the performance of automatic lateral control.

METHODOLOGY

To meet the objective, one semi-autonomous vehicle with automation Level 2 had been driven throughout some arterial tangent sections. The same driver performed all tests. These sections presented a wide range of lane width. The semi-autonomous vehicle was a BMW 520d, manufactured in 2017 and equipped with the package Driving Assistance Plus, with Adaptive Cruise Control and LKA. With both systems activated the vehicle takes control of acceleration, brakes and steering, keeping within a lane thanks to edge lines detection by means of two video cameras, located behind the interior mirror.

The study site was an urban arterial ring road in Valencia City, 5.4 km long, with diverse lane widths, range between 2.28 and 3.80 m (average 2.70 m). The total number of observed lanes was 81, all located at tangents.

Driver is required to continuously monitor the semi-autonomous system and he is also asked to keep constant contact with the steering wheel, although it can be released for a few seconds, before the system emits an acoustic warning.

By means of sensors this system can track the ongoing lane, as well as the surrounding objects and vehicles. The system controls the vehicle with all this incoming information. If any error, inconsistency or confusion takes place within the process, the system transfers control to the driver, but no audible warning is emitted.

The vehicle was equipped with a high-definition video camera with GPS. This camera was mounted besides the driver's head, for simultaneous recording of the roadway, navigation map, dashboard, hands position on the steering wheel, and the driver's comments.

The minimum number of trips for every lane was 10, with the registration for every lane and trip with or without automatic lateral control. Then, human and automatic lateral control rates were determined for every lane.

FINDINGS

In the Figure 1, the automatic and human lateral control rate is represented for every lane, with different widths. The main hypothesis is confirmed as the automatic control rate increases proportionally to the lane width. In contrast, the human lateral control rate decreases.

The human lateral control is totally mandatory up to a 2.5 m lane width. This means that, for this technology and currently, the proposed concept human lane width is 2.5 m.

Moreover, the automatic lateral control is able to always perform for lanes 2.75 m wide or wider. This means that, from the observations, the automatic lane width is 2.75 m.

In the Figure 2, the average automatic and human lateral control rates are represented for every lane width class (5 cm wide). The above-mentioned performance is observed, and the CW can be calculated as the intersection of both lines, determining the same probability for human and automatic lateral control at 2.72 m.

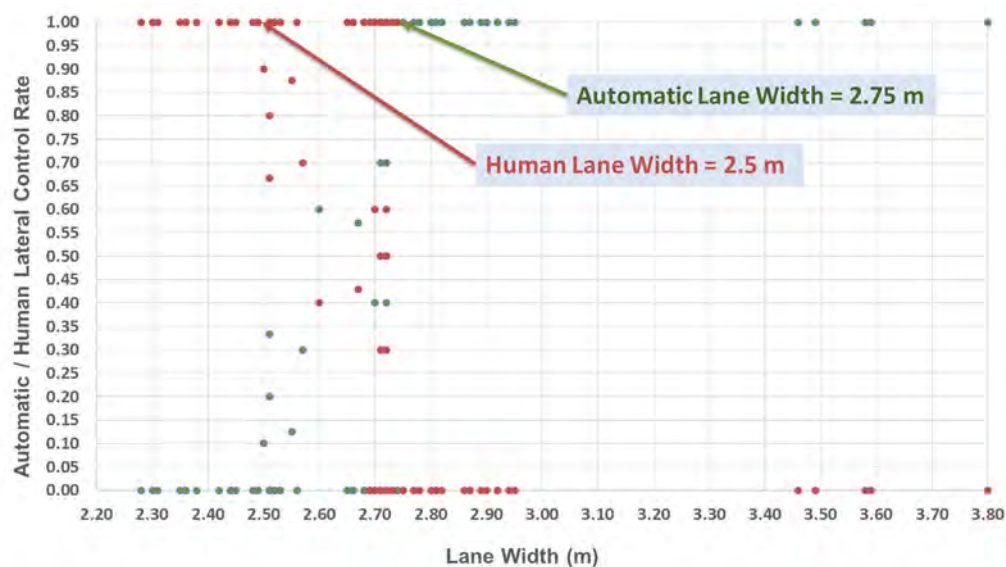


FIGURE 1 Automatic control versus human control.

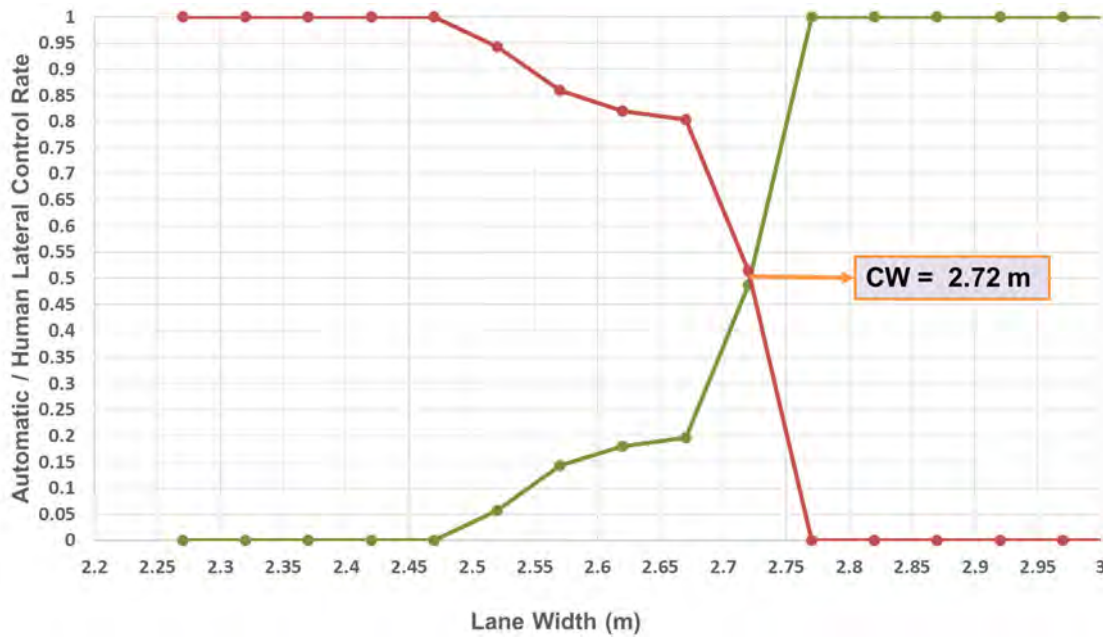


FIGURE 2 Critical lane width.

It is observed that the automatic lateral control is lost very quickly for widths lower than the automatic lane width, that means a very high sensibility of the automatic system, i.e., if any error, inconsistency or confusion takes place within the process, the system transfers control to the driver.

Taking into account the automatic lane width and the total width of the experimental vehicle, 2.75 m and 1.87 m, respectively, the minimum lateral space that the automatic system needs is 0.44 m wide, just for tangent sections.

Therefore, assuming the same lateral space for heavy vehicles and their usual maximum width (2.60 m in United States and 2.55 m in Europe), the automatic lane width for heavy vehicles can be estimated, resulting 3.43 to 3.48 m wide.

CONCLUSION

Three new lane width concepts are proposed and determined for tangent sections: the human lane width (≤ 2.50 m) for total human control; the automatic lane width (≥ 2.75 m) for keeping the lateral control automatically; and the CW (2.72 m) with the same probability of human and automatic control. Therefore, there is a very short lane width range (0.25 m) for losing the automatic control. The current LKA needs a lateral space of 0.44 m that inferred an automatic lane width for heavy vehicles of 3.43 to 3.48 m.

For the current level of development of the vehicle automation, the lane width cannot be reduced and there will be performance problems for heavy vehicles with lanes no wider than 3.50 m. Along low-volume roads, only segments with enough lane width will allow the performance of automatic driving systems, assuming a good visible and continuous road marking for the edge lines. Another issue is related to the common high curvature of the horizontal alignment that provoke many transfers of the control to the driver for the usual operating speed.

Some experimental issues were raised during the observations: related to the road marking discontinuity, color (white and yellow), and visibility of paint and some dust; some shadows over the carriageway; and, the sun glare (east–west orientation).

In a further research, more observations with other makes and models, including heavy vehicles, should be carried out in different environments. Moreover, other road elements, such as horizontal and crest vertical curves, should be analyzed. Adverse weather, night driving and sun glare could be studied. Other aspects related to digital image processing of the semi-autonomous system should be covered so as to perceive and react with less lateral space.

Since increasing the widths and improve road markings on rural road networks is just not going to happen any time soon in any country, using semi-autonomous vehicles on some rural roads may force overridden the systems in the vehicle, thereby compelling a Level 0 condition until the progress of technology.

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Unpaved Roads Management 1

Guide Specification for Gravel Road Surfacing

STEPHEN MONLUX
LVR Consultants LLC

INTRODUCTION

The quality of gravel road surfacing materials varies widely for numerous reasons, many of which relate to rock crushing specifications that are inadequate and poorly enforced. State department of transportation specifications establish good standards and acceptance practices for base course aggregate, but normally do not contain adequate requirements for gravel road surfacing and do not provide reasonable contract administration methods for low-volume road agencies with limited resources. Gravel can be crushed so it lasts much longer when managers use a more comprehensive specification and focus attention on certain critical details during crushing. Monlux provides a guide that has been used by counties in Idaho, Montana and Wyoming since 2015 (1) and has been reviewed by five crushing contractors. In addition to providing suggested specification limits and contract clauses for quality and quantity assurance, it also has a User Guide that explains the rationale behind each part of the specification. This helps personnel decide whether specific provisions are valid as is, or need modification to fit local conditions.

METHODOLOGY

The information presented below and in the referenced specification is a collection of successful approaches to crushing gravel road surfacing that has worked well for low-volume road agencies. Updates are reviewed annually by numerous county road supervisors, crushing contractors, and consulting engineers that are closely associated with low-volume road gravel surfacing.

Materials Source Exploration

Even if there is an exposed work face in the gravel source, dig exploration pits within the pit boundaries to assure materials are consistent, and there is enough material present to produce the desired quantity. Compare the pit run gradation with the gradation specification to determine if the limits are realistic. Adjust as necessary. Consider the costs and benefits of adding select borrow or clay. If necessary, employ a geotechnical–materials engineer familiar with current unpaved road materials practices.

Suggested Process for Changing Specifications

Meet individually with local crushing contractors to explain your objectives and consider their feedback on the proposed specification and the likelihood of bid increases. If realistic, use the old specification name followed by the word “modified” in parenthesis. If there are concerns

about excessively high bids, consider making only a portion of the changes during the first contract.

Contract Prework Meeting

Schedule a meeting that includes on-the-ground supervisory personnel to discuss the specification and note any concerns. Try to resolve the concerns prior to starting mobilization.

Verification of Contractor Testing

Conduct a crusher site meeting between the owner's consultant laboratory personnel and contractor testing personnel when the crusher starts operations to ensure sampling and testing equipment and procedures are correct.

Composite Daily Acceptance Samples

The simple acceptance sampling process eliminates sampling bias and disputes and is critical when the owner wants to reduce quality assurance costs. This process is popular with crushing contractors because the sample represents the daily average instead of snapshot samples taken throughout the day. The daily acceptance sample is taken from a small stockpile formed by multiple bucket loads taken at regular intervals throughout the workday. Samples are taken by the contractor and tested by both the contractor and the owner's consultant. Maximum allowable differences between test results are contained in the specification. Here is an excerpt that shows the level of detail in the acceptance sampling process:

“Build acceptance sample stockpiles by taking full bucket load samples with a front-end loader for every 200 tons crushed. Follow ASTM D75 Section 5.3.3.1. procedures. Sample from fixed rotary stacking conveyors by sampling from the fresh pile face between the coarse and fine sides of the pile. Sample from telescoping rotary stacking conveyors by diverting the stream to fill up a loader bucket. When stockpiling with hauling vehicles, take bucket loads from the surge pile or by filling the bucket from the surge bin. After the acceptance sample stockpile contains 5 or more bucket loads mix the sampling stockpile with a front-end loader, and then flatten to a 12 to 15-inch-thick layer by back dragging the bucket cutting edge. Make a composite acceptance sample from eight to ten random locations on the flattened surface such that a sample of at least 150 lbs. is available for splitting to testing size.”

Gradation and Clay Requirements

Specification limits should be based on what can be realistically produced from local gravel sources. If the sources do not contain the desired amount of clay, there are five options for adding clay that are explained in the next section. The simple payment adjustment process shown in Table 1 encourages contractors to produce better materials and simplifies dispute resolution when gravel does not meet specifications. A 5% bonus payment is available only if significantly better gravel is provided. Marginally qualified contractors, or those that traditionally bid low, do not meet specification or submit numerous claims, will typically bid higher because the payment adjustment process for out-of-specification gravel is part of the contract.

TABLE 1 Requirements for Gradation, Percent Fracture, and Plasticity Index

Pay Adjustment Factor	1.05	1.00	0.95	0.75	0.50	0.00
Sieve Size	Percent Passing					
5/8 inch	100	100	100	100	100	100
3/8 inch (8)	69-81	67-83	65-85	63-87	61-89	<61 or >89
No. 4 (6)	50-66	48-68	46-70	44-72	42-74	<42 or >74
No. 16 (4)	27-40	25-42	23-44	21-46	19-48	<19 or >48
No. 40 (3)	19-28	17-30	15-32	13-34	11-36	<11 or >36
No. 200 (2)	10-16	8-18	7-19	6-20	5-20	<5 or >20
% fracture, one face, min. (15)	75	65	60	55	50	<50
Plasticity index (5)	4-10	4-10	2-10	0-10	0-12	0 or >12

NOTE: Values within parenthesis are maximum allowable differences between consultant and contractor test results. The pay factor selection is based on the average of all gradation tests when gravel is stockpiled. Specification limits in this table must be adjusted to make the requirements realistic for agency-owned or local commercial gravel sources.

Options for Adding Clay Binder

Other than gradation, clay binder is the most critical component in gravel surfacing and should be included to reduce dusting, wash boarding, raveling, potholing, and blading maintenance. One of five options for adding clay must be selected by editing the specification. Here is an excerpt that shows the five options as they appear in the specification.

Option I For gravel sources with no clay. Uniformly add material containing clay in the amount needed during crushing to meet requirements in Table 1. Clay may need to be pulverized such that the crushed gravel has no more than 2% clay lumps retained on the No. 4 sieve. The plasticity index requirement can also be achieved by adding processed clay or bentonite from private offsite sources.

Option II For gravel sources where the amount of clay is unknown: If necessary, add offsite clay or processed bentonite to meet plasticity index specifications in Table 1. Clay may need to be pulverized such that the crushed gravel has no more than 2% clay lumps retained on the No. 4 sieve.

Option III Where the agency provides a stockpile of clay: While crushing and stockpiling aggregate, load, haul, and uniformly add stockpiled clay shown on the drawings in the amount needed during crushing to meet requirements in Table 1. Clay may need to be pulverized such that the crushed gravel has no more than 2% clay lumps retained on the No. 4 sieve.

Option IV Where the agency designates a clay source and specifies a by weight percentage to add to the aggregate—plasticity index requirement in Table 1 is deleted: Add clay from the designated source in the percent by weight of aggregate indicated on the drawings or in the schedule of items.

Option V Where the agency knows clay is present in the desired amount in the source or the agency plans to add clay to the stockpile or to gravel on the road after placement: Delete this subsection and also delete requirements for plasticity index in Table 1.

Stockpiling Requirements

Stockpile area floor construction methods are provided as well as requirements for stockpiling that reduce segregation. A bonus payment is provided where specific methods and equipment are used that reduce segregation.

Suitability and Utilization of Agency-Owned Materials Sources

Wording in this section will help eliminate claims from contractors that cannot meet specifications, provided the owner has done a reasonable gravel source investigation and the selected specification limits for gradation and plasticity index are realistic.

Measurement by the Cubic Yard

The cubic yard payment has several advantages over payment by the ton. Although tonnage is the preferred method of measurement by contractors, it is often difficult for low-volume road agencies to assure accuracy since they have a small workforce. Accurate drone survey equipment and software now exist to make volume measurements practical and avoid quantity assurance problems related to verifying quantities from conveyor belt or loader scales that can be manipulated by contractors. Volume measurements also improve the accuracy of quantity accounting for internal audits within road agencies. The specification for stockpile measurement by the contractor is shown below:

“Determine cubic yard quantities of crushed aggregate by conducting measurements before and after stockpiles are built by employing a licensed surveyor to conduct measurements via a 3D or 2.5D reconstruction. Determine volume using unmanned aerial vehicle photogrammetric processes that capture a ground sample distance of no more than 5 cm. Adequate ground control points and check points must be incorporated whereby all points are collected according to industry-standard survey practices. Provide all survey data to the agency for verification.”

Payment Between 90% and 110% of Specified Quantities

The range for acceptable quantities was established to reduce contractor bidding contingencies related to not getting paid for the quantity of gravel actually crushed.

Suggested Information to Include on the Drawings

The specification user note provides a list of items to include on the pit plan for designated gravel sources. This list should help owners with gravel pit development and utilization as well as reducing contract claims when specifications are difficult to meet.

FINDINGS

Counties using some or all of the suggested practices have found that gravel and dust abatement performance life increases significantly and that these increases more than offset the relatively small increase in crushing cost attributed to requiring more realistic and comprehensive specifications. Unfortunately, gravel performance life measurement requires a long-term commitment that most

road managers find difficult to complete. An anecdotal indicator of improved performance is simply having to crush less rock per year after better specifications are used.

CONCLUSION

Annual costs for gravel surfacing can be reduced and quality improved by adopting better specifications for crushing. Other benefits include less road blading, reduced dust abatement costs, lower resource depletion, and more satisfied road users.

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Benefits and Limitations of Installing Driving Surface Aggregate at Two Federal Lands Sites

BETHANY K. KUNZ

U.S. Geological Survey

ERIC H. CHASE

STEVE M. BLOSER

Penn State University

MAUREEN A. KESTLER

U.S. Army Corps of Engineers

BRANDON JUTZ

U.S. Fish and Wildlife Service

INTRODUCTION

The worldwide network of unpaved roads is estimated to include at least 14 million km (8.7 million miles) (1). Although they are vital for local communities, these roads are expensive to maintain and may cause environmental damage through sediment and dust pollution (2). Among aggregate-surfaced roads, locally available materials are often used as a surface wearing course, with little or no testing and sometimes no formal specification. The materials vary widely in quality and may deteriorate quickly. As a result, road managers may be forced to increase the frequency of maintenance grading and aggregate replacement to compensate for the poor performance. Improving the quality of surface aggregate on unpaved roads is one strategy for increasing road performance while also reducing environmental impacts. Although higher-quality aggregates require greater up-front investment, they can result in lower overall life-cycle costs by extending road life and reducing maintenance costs.

Driving Surface Aggregate (DSA) is an aggregate specification developed by the Pennsylvania State University Center for Dirt and Gravel Road Studies that is designed to achieve maximum compaction and resist erosion. The gradation of DSA, coupled with recommended optimum moisture and placement guidelines, results in a smoother, more tightly bound surface that preserves fine material rather than allowing it to escape as sediment or dust. In previous studies, DSA has been shown to reduce sediment runoff by 80% to 90% (3) and dust production by up to 90% (4) compared to existing road surface gradations. Although DSA has been used extensively in the state of Pennsylvania the specification is almost unknown elsewhere. The objective of this study was to demonstrate the benefits and limitations of DSA when deployed across a wider geographic area.

Road sections of DSA were installed at two federal lands sites in the eastern United States. Sediment runoff, dust production, and road surface condition on these sections were measured approximately 12 months post-construction. At both sites, DSA reduced sediment runoff by up to 91%, relative to traditional aggregates. At one site in Indiana, DSA also reduced

dust production and aggregate loss. At the other site in Vermont, DSA and traditional aggregate sections performed similarly in dust production and road condition. Overall, this study (1) demonstrates that DSA can be an effective and environmentally responsible aggregate choice for unpaved roads, and (2) provides information on site conditions (e.g., roads near headwater streams) under which DSA is likely to be particularly beneficial.

METHODOLOGY

Road sections of DSA were installed at the Muscatatuck National Wildlife Refuge (NWR) near Seymour, Indiana, and at the Green Mountain National Forest (NF) near Rochester, Vermont. These sites were chosen to represent broad ecological areas in the eastern half of the continental United States. Prior to construction, samples of DSA mix from each supplying quarry were analyzed for size-gradation, LA abrasion resistance, pH, and plasticity to confirm compliance with all DSA specifications (5), including the requirement that DSA be produced from crushed aggregate. Because DSA may be used as an alternative to dust abatement or other unpaved road treatments, no additional treatments were applied to test sections. At each site, an additional road section was surfaced with an aggregate that was traditionally used for previous unpaved road projects at the site (i.e., business as usual). Measurements of DSA performance were compared to the traditional aggregate to ensure that any benefits observed could be attributed to DSA, rather than simply to the addition of new aggregate.

At Muscatatuck NWR, a 1.5-mi (2.4-km) section of DSA and a 0.25-mi (0.4-km) section of traditional aggregate were placed on a one-way tour loop through the refuge. The road was 12 feet (3.7 m) wide with grades up to 3% and primarily open canopy vegetation. Traffic consisted of passenger vehicles [estimated 50 average daily traffic (ADT)] and the speed limit was 25 mph (40 km/h). Each road section was graded to establish a 4% to 6% crown prior to placement. As recommended in DSA guidance (5), DSA was brought to optimum moisture based on standard proctor testing, placed in a single 6-in. (15-cm) lift using a motor paver, and then compacted to 4.5 in. (11 cm) with a vibratory compactor. Traditional aggregate was placed using standard local procedures of tailgate spreading without optimum moisture and compaction with a vibratory roller mounted to a skidsteer. Placement differences were expected to have less influence than material characteristics on aggregate performance, based on a previous 3-year study with DSA and other aggregates (6). The DSA specification was quite different from that of the traditional aggregate used at Muscatatuck NWR (Table 1).

At the Green Mountain NF, a 0.75-mi (1.2-km) section of DSA and a 0.25-mi (0.4-km) section of traditional aggregate were placed on Chittenden Brook Road, which provides seasonal access to a popular campground. The road was 18 ft (5.5 m) wide with grades up to 14% and primarily closed canopy vegetation. During the open season (May 1 to December 15), traffic consisted of passenger vehicles (estimated 25 to 50 ADT) and the speed limit was 25 mph (40 km/h). Local quarries were limited near the Green Mountain NF, which increased haul distances and associated costs for the DSA installations. Paver availability was also limited; therefore, DSA and traditional aggregate were tailgate-spread. Although the installation did not comply with standard DSA placement guidance (5), it provided a representative test case for other remote sites that will likely face similar challenges. Prior to placement, the road was graded to establish a 4% to 6% crown where possible. DSA was tailgate-spread at optimum moisture, shaped to 6-in. (15-cm) depth with a motor grader, and compacted to 4.5 in. (11 cm) with a

vibratory compactor. Traditional aggregate was tailgate-spread without optimum moisture. The DSA specification was quite similar to the existing Forest Service aggregate specification in use at the Green Mountain NF (Table 1).

Performance of DSA and traditional aggregates at both sites approximately 12 months after construction were compared using three metrics: (1) sediment runoff, (2) dust production, and (3) surface condition. Sediment runoff was measured using a rainfall simulator and runoff collection system developed by the Center for Dirt and Gravel Road Studies (3). The system simulated a rainfall event [0.6 in. (1.5 cm) in 30 min] on a 100-ft (130-m) section of road. The rate of runoff was measured and runoff samples were taken to estimate total sediment load produced by each section. Dust production (total particulate matter) was measured using a vehicle-mounted DustTrak DRX Aerosol Monitor (Model 8533, TSI Incorporated, Shoreview, Minnesota). Each road section was driven three times with a sampling rate of one sample per second, yielding three dust profiles per section (7, 8). All measurements were taken at a height and speed recommended by previous studies (9, 10). Road surface condition was assessed using a protocol modified from Central Federal Lands Highway Division (11) to quantify the severity of four common surface distresses—wash boarding (corrugations), raveling (loose material), rutting, and potholes—and then assign relative ratings of performance.

FINDINGS

At Muscatatuck NWR and the Green Mountain NF, DSA reduced sediment runoff by 91% and 64%, respectively, relative to traditional aggregate sections (Figure 1). Differences in sediment production were more dramatic at Muscatatuck, which was expected because of larger differences in the traditional aggregate and DSA specifications. Muscatatuck also is open to traffic year-round, which likely increased wear on the road. To put sediment runoff at Muscatatuck in perspective, each mile (1.6 km) of DSA-surfaced road could be expected to

TABLE 1 Specification for DSA and Actual Material Characteristics of DSA Mixes Compared to the Traditional Aggregate Specifications Used at Muscatatuck NWR, Indiana, and Green Mountain NF, Vermont

Passing Sieve Size	Percent passing each sieve by weight										Plasticity Index
	mm	38	25	19	9.5	4.76	2.36	1.19	0.420	0.074	
	in	1.5	1	0.75	0.37	0.187	0.094	0.047	0.017	0.003	
sieve					#4	#8	#16	#40	#200		
Driving Surface Aggregate											
DSA Specification	100			65-95		30-65		15-30		10-15	0-6
Indiana DSA (actual)	100	99	85.3	49.3	31.7		16.6		11.2	0	
Vermont DSA (actual)	100	100	100 ^a	79.6	58.1		35.7 ^a		11.5	0	
Traditional aggregates											
Indiana Coarse Aggregate #53 ¹	100	80-100	70-90	55-80	35-60	25-50	10-12		5-10	0-5 ^b	
USFS Surface Gradation F ²	100	97-100	76-89	56-68	43-53		23-32	15-23	10-16	2-9 ^c	

^a Values slightly out of range of DSA specification

^b In addition to Plasticity Index range of 0-5, the liquid limit shall not exceed 25

^c Plasticity Index range is 2-9 if passing 0.074mm sieve is less than 12%, but must be 0 (non-plastic) if passing 0.074mm sieve is more than 12%

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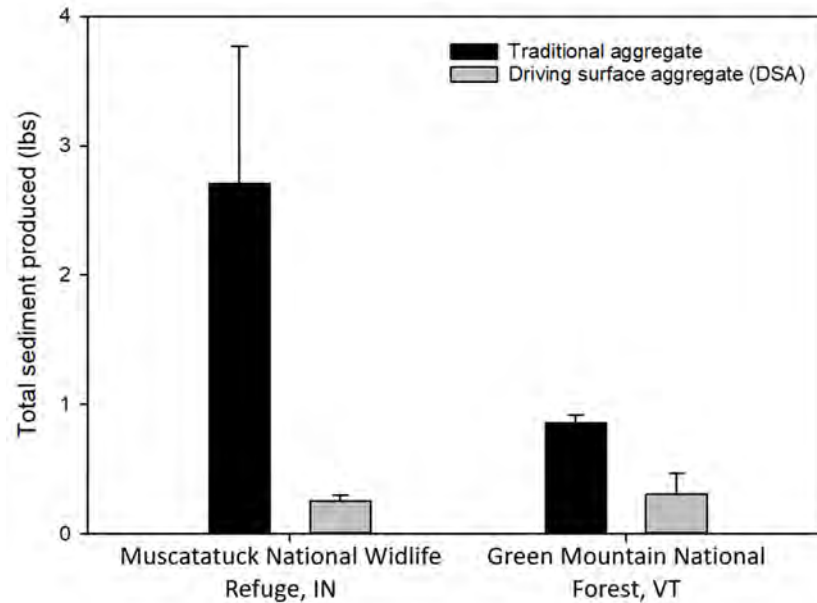


FIGURE 1 Average sediment production from each road section during a simulated rain event (0.6 in. in 30 min). Bars represent means of three rain events per section. Error bars represent standard deviations.

produce approximately 13 lb of sediment per 0.6 in. (1.5 cm) in 30 min rain event, while each mile (1.6 km) of traditional aggregate would produce more than 143 lb.

For dust production, the DSA section at Muscatatuck NWR produced 77% less dust than the traditional aggregate section (Figure 2). At the Green Mountain NF, dust production was essentially equivalent between the DSA and traditional sections. This pattern likely occurred as a result of (1) the similarity of the two aggregate specifications and (2) the closed canopy conditions, which helped maintain road moisture on both sections. At both Muscatatuck NWR and the Green Mountain NF, the overall concentrations of dust measured were limited by unexpected rainfall prior to sampling.

For road surface condition, the primary benefit of DSA relative to the traditional aggregate was in the reduction of raveling or loose aggregate. At both sites, the DSA section had less loose material on the road surface (Table 2), highlighting DSA's ability to maintain a compact surface and reduce aggregate loss. These benefits would be expected to result in reduced need for surface maintenance, reduced frequency of aggregate replacement and lower life-cycle costs.

CONCLUSIONS

Overall, this study adds to the evidence that DSA can be a useful technology for improving unpaved road performance. At both Muscatatuck NWR in Indiana and the Green Mountain NF in Vermont, DSA reduced sediment runoff by up to 91%, relative to traditional aggregates. In this

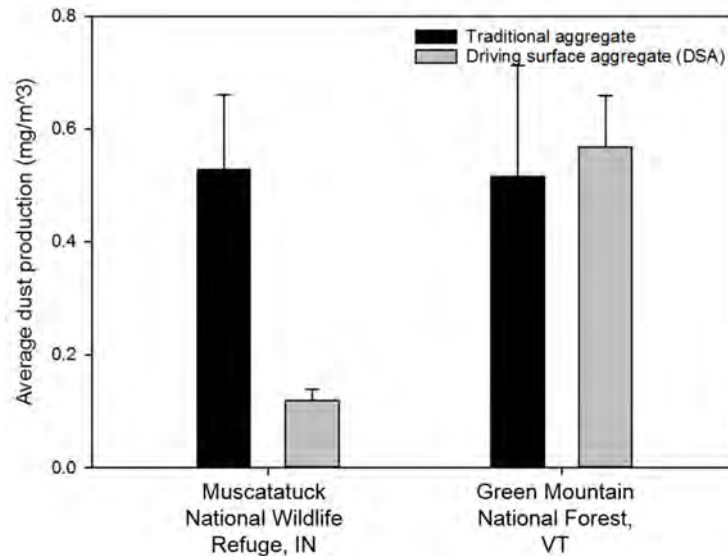


FIGURE 2 Average dust production for each aggregate type at each test site. Bars represent means of three runs per section. Error bars represent standard deviations.

TABLE 2 Road Condition Evaluation Results ~12 Months After Installation of DSA Sections, Relative to Sections Surfaced with Traditional Aggregate; Ratings Were Assigned Based on Depth of Surface Distresses According to Appendix B of Woll et al. (11)

Site	Section	Washboarding		Raveling		Rutting		Potholes	Overall rating
		Mean depth (mm)	Rating ^a	Mean depth (mm)	Rating	Mean depth (mm)	Rating	Rating ^b	
Muscatatuck National Wildlife Refuge, IN	Traditional aggregate	0	10	17	6	8	8	10	8.5
	DSA	0	10	10	7	8	8	10	8.8
Green Mountain National Forest, VT	Traditional aggregate	1.6	9	7	8	4	9	10	9.0
	DSA	0	10	3	9	7	8	10	9.3

^aAll ratings on a scale of 0 to 10, with 10=best condition

^bNo potholes were observed on any section; therefore, all sections received a rating of "10"

NOTE: 25.4 mm=1 in

case the use of DSA would translate into the elimination of 130 lb of sediment runoff per mile (1.6 km) of road per 0.6-in. (1.5-cm) rain event. Therefore, the adoption of DSA may be especially beneficial at sites where unpaved roads are adjacent to water resources such as headwater streams. At Muscatatuck NWR, DSA sections also produced less dust and exhibited less aggregate loss than traditional aggregate sections, thereby improving driving conditions for visitors and staff. At the Green Mountain NF, DSA and the traditional aggregate performed similarly in dust production and road condition. Site-specific differences in this demonstration provide useful information on local conditions that influence the relative benefit of choosing DSA.

DSA is likely to be most beneficial at sites where traditional aggregate specifications are quite different from DSA, the blending and hauling of the material is not cost-prohibitive, and the road section in question has typically required frequent maintenance. DSA may not be a cost-effective choice for some remote or high-altitude sites, primarily because of hauling concerns. DSA is also likely to be less beneficial on roads that have only limited issues with dust and surface distresses.

Although DSA may not be the best choice for all sites, the benefits demonstrated here highlight the importance of more rigorous characterization and more careful selection of aggregates for unpaved roads—the subject of a growing body of work worldwide (12–14). The most convenient or cheapest aggregate is rarely the best choice when considering full life-cycle costs, including maintenance blading and frequency of aggregate replacement. Although improved aggregates may require more up-front investment, the result is a road that lasts longer, has less environmental impact, and costs less to maintain.

DISCLAIMER

Any use of trade, firm, or product names is for descriptive purposes only and does not imply endorsement by the U.S. government.

ACKNOWLEDGMENT

This research was funded by Federal Highway Administration Federal Lands Highway Coordinated Technology Implementation Program.

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Clay Content Optimization Test for Gravel Road Surfacing

STEPHEN MONLUX
LVR Consultants LLC

BETHANY DURIE
Holman Consulting Engineers

INTRODUCTION

This paper addresses surface course aggregates for which it is anticipated a clay additive is necessary to improve gravel life. In current practice, gravel road surfacing specifications generally consist of requirements for plasticity index (PI) or gradation. Gravel surfacing meeting these traditional criteria often underperforms and may experience excessive dusting and loss of aggregate. The use of chloride for dust control is considered essential in dry climates on gravel roads with heavy truck traffic regardless of the gradation and clay characteristics. The clay optimization procedure uses chloride as a standard additive and determines the optimum amount of clay needed to control chloride leaching and limit the potential for rutting issues in the wetter months of spring.

The clay content optimization process involves evaluation of compacted chloride-treated aggregate–clay mixtures (that typically fall close to or within traditional specified parameters) over a range of clay contents. This method utilizes California bearing ratio (CBR) and chloride retention tests that indirectly measure aggregate packing characteristics and void structure through measurement of the rutting potential and tendency for leaching of chloride. Minimum thresholds for the CBR value and percent chloride retention have been set based on personal field observations of the author. Optimal clay content is typically the highest value at which both thresholds are met.

Others have suggested a more comprehensive approach in evaluating anticipated performance of surfacing aggregates by considering parameters set for additional criteria such as grading coefficients and shrinkage products (*I*). These methods are not excluded from the clay content optimization process, and this process can be used to further evaluate and optimize surfacing gravels that fall within those parameters.

Since 2006, use of this methodology has provided improvement to surfacing performance and life-cycle costs associated with 15 or more projects in Montana, Idaho, Wyoming, and Alberta, Canada.

METHODOLOGY

The process described assumes the aggregate used requires addition of clay to achieve proper performance and that 1.5% calcium chloride for dust abatement is going to be used. Use of higher percentages of chloride can create issues during construction by causing aggregate to hold onto moisture at levels exceeding optimum. Use of lower percentages of chloride have been

found to be less effective in that they abate dust for significantly shorter duration of time. Durie (2) provides a detailed explanation of the process and testing forms. An overview of the process for clay optimization testing is explained in the following sequential steps.

Aggregate Sampling

Aggregate must be sampled carefully to ensure it represents the overall gradation of the material to be utilized. There are numerous published guidelines that can be consulted for information regarding approved methods of sampling.

Aggregate Test Sample Formation

Field samples should be visually compared in the laboratory for differences in gradation or composition. Separate clay content optimization determinations may be warranted for each distinct material encountered. Samples representing each of the separate materials encountered should be homogenized and reduced into representative portions for testing using standard methods. The sample size required for each aggregate–clay blend is approximately 100 to 150 lb.

Initial Aggregate Testing

The accuracy of the test sample reduction process should be confirmed by performing sieve analysis, PI, and moisture content testing on two randomly selected sample splits. If tests do not compare within reason, field samples may require recombination, additional mixing, and re-splitting. Test results from field samples should also be compared with those from crushing quality control testing to determine if the field sampling is representative. If differences are significant, validity of field sampling and crushing quality control testing may need to be investigated before proceeding with further testing.

Additive Sample Sources and Characteristics

Samples of clay used in the optimization testing process should be from the same sources and in the same form that is intended for use during construction. Use of commercially processed bentonite clays is preferable to pit or bank run clays because they are more uniform and are already reduced to sand size or in powder form. If intending to use pit or bank run clays, they must have a low enough moisture content to allow pulverization to less than $\frac{1}{4}$ in. The gradation and PI of each clay sample should be determined prior to use in the optimization procedure. This will assist in selecting the percentages to use for trial blending.

While calcium chloride may be used in the field in either liquid or dry form, it is recommended that the chloride be incorporated in the liquid form when performing laboratory tests on aggregate–clay mixtures. This will better distribute the chloride throughout the mixtures during the testing process thereby producing more consistent results. Dry calcium chloride pellets should be made into a salt brine at approximately 38% concentration. Brine concentrations in the laboratory should be verified before proceeding with clay optimization testing.

Selection of Clay Contents for Optimization Testing

If using bentonite, trial percentages of 2%, 3%, and 4% are suggested starting points. If using bank run clays, trial percentages of 3%, 5%, and 7% are suggested. All blending should be done based on dry weights of both aggregates and clay.

Mixing Aggregate with Additives

Aggregate–clay mixtures should be prepared at three different percentages of clay. One large batch can be prepared for each aggregate–clay blend from which material for individual specimens can be split, or individual batches can be prepared for each compacted specimen (recommended). 1.5% calcium chloride (in brine form) by dry weight of aggregate–clay blend should then be incorporated into the mixture. Additional water should be added to the material for each specimen to reach target moisture contents. After bringing to desired moisture content, material should be placed in covered containers and allowed to sit overnight prior to compaction.

CBR Testing (AASHTO T193)

After selection of clay contents to be used in the trial mixtures, optimum moisture and maximum dry density of each of the mixtures should be determined using the modified compaction method (AASHTO T180). Three CBR specimens should be prepared at optimum moisture for each of the clay contents. These specimens should be compacted to target densities that span a range from approximately 92% to 98% of maximum dry density. Specimens should then be soaked for four days with a 10 lb surcharge. After the soak period, CBR penetration testing should be performed on each of the specimens. Following penetration testing, specimens should be removed from the molds and retained for subsequent testing.

Chloride Retention Testing

Material from the unmolded specimens should be oven dried to constant weight. It is critical that dry weights of specimens be recorded immediately upon removal from the oven to avoid potential inaccuracies caused by calcium chloride's affinity for absorbing moisture. Specimens should be transferred into plastic containers and covered with enough water for adequate free liquid above the aggregate for specific gravity testing of the solution. The weight of added water should be recorded.

The material should be thoroughly mixed with the water to allow the chloride to enter into solution. The sediment should be allowed to settle until the supernatant solution is clear. The calcium chloride concentration of the solution should then be determined by specific gravity testing, and the dry weight of calcium chloride calculated. The percent chloride remaining in the sample after CBR testing is the weight of calcium chloride relative to the dried sample weight minus the weight of the calcium chloride. The percent chloride retention is the percent chloride retained by the specimen after CBR testing relative to the 1.5% that was originally added to the material prior to compaction.

Data Tabulation and Analysis

Values for CBR and chloride retention at 95% compaction should be determined for each of the three aggregate–clay trial blends. These values should be plotted against the corresponding clay contents in the aggregate–clay mixtures. The optimal clay content is selected by determining the highest added clay content that meets the minimum CBR of 40 and minimum chloride retention of 70%. CBR values above 40 generally have reasonable resistance to rutting after being subjected to spring snowmelt saturation, provided road crowns are maintained in the fall season prior to winter. Values above 70% chloride retention normally provide good chloride life and resist leaching. Example plots of CBR and chloride retention versus percent clay used for optimal clay content selection are shown in Figure 1.

FINDINGS

The clay optimization process has been found to be useful in improving performance of gravel road surfacing. This process provides a more comprehensive means of evaluating the quality of aggregate–clay mixtures used for gravel road surfacing than traditional methods that consider PI alone in determining percentages of clay additive. This process takes into account characteristics of the compacted mixtures in determining the optimum clay content. It quantifies the ability of the aggregate–clay blends to retain chloride which extends the life of dust abatement, thereby reducing costs. This process also identifies rutting potential that may be caused by addition of

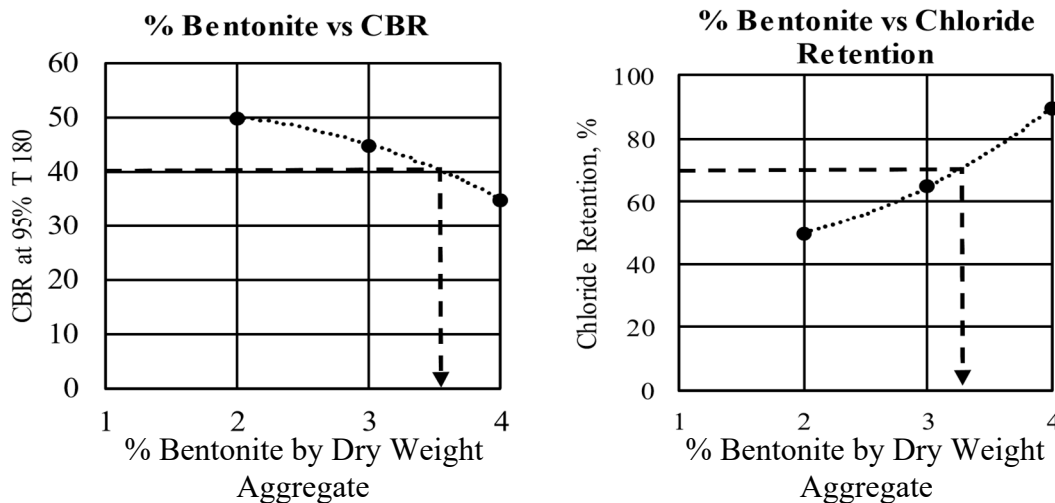


FIGURE 1 Clay and chloride mix design summary. CBR and chloride retention values at 95% modified compaction for each aggregate–clay mixture are plotted against its percentage of added clay. Optimized clay content is the maximum clay content at which the minimum thresholds for CBR value (40) and chloride retention (70%) are met. The minimum CBR value of 40 is met with 3.6% or less added clay. The minimum retention value is met at 3.2% or more clay. The optimized clay content for this aggregate–clay mixture is 3.6%.

too much clay and thereby provides a means to minimize cost associated with overuse of expensive clay additive and rutting damage to aggregate surfacing.

It has been found that clay treated aggregate containing chloride can be slippery in the spring for a week to 10 days in drier climates of the northern Great Plains states and in central Canada. However, this problem can be reduced by maintaining 4% crowns and is quickly forgotten by road users and agency personnel when they realize that the aggregate–clay–chloride mix provides a dust free smooth road surface for 4 to 6 months and that road blading and aggregate replacement is significantly reduced.

CONCLUSIONS

The clay optimization test process has the potential to offer benefit to gravel road surfacing projects by providing the means to evaluate performance characteristics of surfacing aggregates in the laboratory prior to use in construction. In so doing, optimal blends can be selected to maximize performance in the field. Implications for its benefit are improved chloride retention and reduced rutting potential. These benefits can also translate into savings associated with maintenance and life-cycle costs.

Due to observed success in producing satisfactory end products over the course of more than a decade of use, the author has continued to advocate for and utilize this process.

While this process has been found useful, further testing and observations of its benefits may lead to further refinement of the process and more widespread use. This could in the future result in general acceptance and incorporation into specifications used by agencies overseeing construction of low-volume roads.

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Use of Gravel Loss Prediction Models in the Design and Management of Unsealed Road Pavements

IAN VAN WIJK

DAVID WILLIAMS

MEHDI SERATI

University of Queensland

Many countries have large unsealed road networks which are essential for business efficiency, social connectedness, and community safety. These roads are maintained at considerable cost mainly through blading and re-graveling. The latter is a major component of the maintenance and is determined by gravel loss. Gravel loss prediction models can be used to assess the effect of different wearing courses, which provides useful input into the design and management of unsealed roads. This paper reviews the origins, input parameters, and output of four available gravel loss prediction models—the Transport and Road Research, HDM-4, Australian, and South African models. It was found that the predictive accuracy of the models is in general low and they predict very different gravel loss results. There is also a lack of integration between the design and maintenance, leading to the wearing course properties recommended for design not being directly linked to gravel loss. These findings led to further analysis of the models and the development of re-graveling frequencies based on traffic, climate (annual rainfall), and material property (plastic factor), which can be used in the selection of the appropriate wearing course material and in the determination of re-graveling budgets. This presents a simplified approach to the use of existing gravel loss prediction models in the design and management of unsealed roads which mitigates some of the shortcomings in the use of uncalibrated models.

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Long-Term Study on the Cost-Effectiveness of Dust Control on Untreated Aggregate-Surfaced Roads

GLEN LEGERE
ALLAN BRADLEY
FPInnovations

A long-term study of treated and untreated aggregate resource roads in Canada was conducted. The objective was to investigate the cost-effectiveness of annual dust control treatments where the hypothesis is that annual applications may prolong aggregate life. Seven sections along two road segments with different traffic levels were studied over 5 years. A survey of road users revealed that 88% agreed that the treated sections were safer because of the increase in visibility and quicker dust settlement times. Evaluation of surface aggregate indicated some aggregate wear but there were no significant differences between treated and untreated sections. The source and quality of crushed aggregate has an impact on road performance. The condition of the running surface did not indicate any major performance differences between the treated and untreated sections. Regardless of treatment, age, or aggregate sources, a general downward trend in Unsurfaced Road Condition Index was observed, indicating wearing course degradation over time. The study revealed a strong correlation between traffic volume and maintenance intensity. Moderately higher travel speeds were measured on the treated versus untreated sections. When the cost of treatment and maintenance was compared with historical costs, the dust control scenario was more expensive. However, when log hauling cost savings from increased travel speeds were introduced, the dust control was approximately cost neutral in low traffic scenarios and moderately better for high traffic. If non-quantifiable benefits, such as increased safety, were to be considered, application of dust control treatment is recommended.

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Geosynthetics in Low-Volume Roads

Fiberglass ACE Mat as a Temporary, Rapid Alternative to Unpaved Road Surfaces

DANIEL HARDER

LYAN GARCIA

JOHN RUSHING

U.S. Army Corps of Engineers Research and Development Center

Temporary, low-volume roadways often challenge designers with balancing cost of improving weak soils with the performance requirements of the intended application. Construction costs can be high, depending on the strength of the native soil and improvements required to sustain the vehicle loadings that are applied. In an effort to provide a solution for ground improvement that balances cost, logistics, and performance, the U.S. Army Corps of Engineers (USACE) developed a multipurpose, medium-duty matting system constructed of woven fiberglass reinforcement and polyester resin. This paper discusses three demonstrations in which the U.S. Army Corps of Engineers Mat (ACE Mat) was used as a surfacing for temporary roads. First, ACE Mat was used to support weak soil crossings for logging operations at the Apalachicola National Forest in Florida. Second, the matting was tested for its capability to support loads from a U.S. Marine Corps Medium Tactical Vehicle Replacement (MTVR) and a U.S. Army M1 Abrams main battle tank crossing sand and silt soils at the U.S. Army Engineer Research and Development Center in Mississippi. Finally, ACE Mat was demonstrated as a ground surfacing to stabilize a riverbank during temporary bridging training at Fort Knox, Kentucky. Attributes and performance characteristics of ACE Mat in comparison to typical unpaved road design and construction using compacted aggregate are also discussed.

INTRODUCTION

Temporary, low-volume roadways often challenge designers with balancing the cost of improving weak soils with the efficiency of the intended application. Depending on the strength of the native soil and the vehicle loadings that are applied, some construction costs for temporary roads can be very high. For applications such as timber harvesting or oil and gas production, significant improvements are needed to support large equipment and heavy loads. Typical unpaved road design and construction using compacted aggregate can be expensive and can detrimentally affect the environment through permanent environmental changes and excess waste materials. Over the years, many different methods have been employed for stabilizing weak soils. Blinn et al. (1) described methods for using wood mats, panels and pallets, bridge decking, expanded metal grating, and wood aggregate among others for temporary wetland crossing options for forest management. Kestler et al. (2) investigated stabilization techniques including chunk wood, tire chips, geosynthetics, tree slash, tire mats, and wood mats for improving mobility over thawing soils. Sand stabilization has been investigated through the use of fibers (3, 4) and by the use of geocells (5). Rushing and Howard (6) discussed matting solutions for improving mobility over weak soils. Selecting the most appropriate methods for

creating temporary road systems requires consideration of the cost, durability, constructability, environmental impact, etc. Many of the available techniques have considerable tradeoffs when considering these factors.

In an effort to balance cost, logistics, and performance, the USACE developed a multipurpose, medium-duty matting system for ground stabilization. The ACE Mat is constructed from fiberglass, measures 6 ft 8 in. x 6 ft 8 in., and weighs approximately 120 lb. Its modular design enables manual emplacement in a variety of configurations by two persons, uses minimal space for transportation, and supports typical military truck traffic over moderately weak soils. It has been documented to support over 5,000 passes of a military transport truck over sand with a California bearing ratio (CBR) of approximately 15% (7). Anderton and Gartrell (8) demonstrated ACE Mat as a landing surface for rotary wing aircraft and as a way to construct parking aprons for aid for C-130 fixed-wing aircraft.

In this study, ACE Mat was demonstrated as a surfacing for temporary roads in three different scenarios. First, the matting was used to support weak soil crossings for logging operations at the Apalachicola National Forest in Florida. Second, the matting was tested for its capability to support loads from a U.S. Marine Corps MTVR and a U.S. Army M1 Abrams main battle tank crossing sand and silt soils at the U.S. Army Engineer Research and Development Center in Mississippi. Finally, ACE Mat was demonstrated as a ground surfacing to stabilize a riverbank during temporary bridging training at Fort Knox, Kentucky. The objective of this paper is to provide a practical perspective on attributes and performance characteristics of the ACE Mat system.

MATTING PROPERTIES

ACE Mat is constructed from fiberglass and polyester resin using a closed molding process, resin transfer molding or vacuum resin transfer molding. The panels are designed in such a way that two adjacent edges are recessed at the panel bottom while the opposite two adjacent edges are recessed at the panel top to allow overlapping for connection during placement. The recessed edges (shown in Figure 1) contain 2.5-in.-diameter holes that align with each other for connecting and anchoring the matting panels. The thickness of the panels in the center is 0.36 in., and the panel edges are 0.18 in. The resin includes ultraviolet radiation inhibitors to minimize degradation during outdoor use. The fiberglass reinforcement is E-glass type and consists of a multi-axial woven roving bonded to chopped strand backing. The ratio of fiberglass reinforcement to resin is approximately 55:45 by mass. Required properties of ACE Mat panels are listed in Table 1.

For this study, installed matting was connected and anchored using 46-in. long commercial screw anchors made of cast aluminum 356 alloy (Figure 1). To minimize exposure and potential tire damage, the 2-in. hexagonal head was modified by cutting the head from a height of 1.875 in. to a reduced height of 0.75 in. The anchors were installed using a hydraulic impact wrench.



FIGURE 1 ACE Mat panel and aluminum screw anchor.

TABLE 1 ACE Mat properties.

Property	Requirement	Test Method Reference
Tensile strength, x- or y-axis (min.)	32,000 psi	ASTM D3039/M
Flexural strength, x- or y-axis (min.)	42,000 psi	ASTM D790
Thermal expansion (max.)	1.8×10^{-5} in./in./°C	ASTM D696
Ignition temperature, flash (min.)	450°F	ASTM D1929
Hardness, top (min.)	45	ASTM D2583
Hardness, bottom (min.)	45	ASTM D2583

DESCRIPTION OF FIELD SITES

Apalachicola National Forest, Florida

The Apalachicola National Forest in northwest Florida lies in a coastal plain with a sandy soil containing organic material near the surface. A logging road was used as a demonstration area during a timber harvest. Prior to the installation of mats, dynamic cone penetrometer (DCP) (ASTM D 6951) data were collected to determine the bearing capacity of the soil at different depths. CBR values were computed using DCP data according to equation 1, where PR is the penetration rate of the DCP. The upper 1 ft had CBR values of approximately 4%. From 1 to 2 ft, the soil strength increased to approximately 10 CBR (Equation 1).

$$\text{CBR} = 292/\text{PR}^{1.12} \quad (1)$$

The site was rough-graded, using a D-7 dozer, to remove major rutting and trees located in the test area. A section of ACE Mat was placed with two panels next to each other to fit the width of the road, resulting in a section five panels long on either side of the road centerline (Figure 2) for a total of 10 panels. An unsurfaced control section without any matting was also prepared immediately preceding the ACE Mat section so natural ground rutting could be



FIGURE 2 Apalachicola National Forest ACE Mat installation.

compared to areas surfaced with matting. Rut depth measurements were collected with a ruler and a straightedge on the control section and the matting sections of ACE Mat (Figure 3) after 50 passes with a loaded dump truck. While the adjacent bare soil exhibited over 5 in. of rutting, the soil with ACE Mat installed showed very little rutting. Normal traffic operations took place on the road for approximately 2 months with no major issues.

U.S. Army Engineer Research and Development Center, Vicksburg, Mississippi

ACE Mat was placed on two different soil types in an outdoor testing facility (Figure 4) to measure performance under military vehicle loads as part of a site stabilization experiment for the Improved Ribbon Bridge–Bridge Supplementary Set (9). First, a loose sand test area was prepared using a locally available poorly graded sand (SP). This SP material was cohesion less and provided adequate bearing capacity. The sand test area was prepared by placing the material from dump trucks over the native ground. No compaction was performed during placement. The material was placed to a depth of approximately 4 ft across the test section. The sand was bladed smooth using a skid steer loader and bucket attachment prior to mat placement.

A second test area at the Engineer Research and Development Center (ERDC) site used the in situ natural material at the test site. Prior to ACE Mat installation, a motor grader removed the upper 2 to 3 in. of soil and vegetation. The resulting bare soil consisted of a clayey silt with moderate cohesion. DCP test results showed that the sand site had a CBR value of approximately 15% while the clayey silt area had surface (upper 1 ft) CBR values of approximately 10% with lower layers (1 to 2 ft) having CBR values in the 2% to 4% range.

In contrast to the traffic operations at the Apalachicola test site, operations at the ERDC site occurred over a period of 1 week. Traffic during the testing consisted of an MTVR having a total vehicle weight of 44,600 lb and 28-psi tire pressure in the front two-drive wheels and 35-psi tire pressure in the rear four wheels and an M1 Abrams battle tank weighing 142,000 lb (Figure 5) with track pressure of 11 psi. Traffic was applied in a sequence of 90 passes of the MTVR followed by 10 passes of the M1. Rut depths were measured after each traffic sequence using a rod and level. Lead blocks weighing approximately 4,000 lb were placed in the wheelpath during measurement to deflect the matting. After 1,000 combined passes (900 MTVR, 100 M1) on the sand, the rut depth of the matted area was approximately 1.5 in. For comparison, 20 passes of



FIGURE 3 Rut depth measurement of control section, Apalachicola National Forest.



FIGURE 4 Military truck performing turning operations on SP test section, ERDC site.



FIGURE 5 M1 Abrams tank on SP test section, ERDC site.

the M1 were applied to uncovered sand, resulting in a rut depth of approximately 8 in. For the clayey silt soil, traffic was discontinued after three traffic sequences totaling 300 combined passes (270 MTVR, 30 M1). Rutting on the matted area was approximately 5 in. Fifty passes of the M1 on the uncovered clayey silt test area resulted in a rut depth of approximately 12 in.

Fort Knox, Kentucky

At Fort Knox, an army bridging unit was attempting a river crossing but could not access the far bank of the river due to thick vegetation, weak soil, and a steep slope. After clearing away vegetation and smoothing the ground to a roughly 15% grade, vehicles still struggled to climb the bank. Engineers placed ACE Mat on the slope to provide a roadway 12-ft wide and 96-ft long (Figure 6). The matting climbed the 15% grade onto level ground and followed a horizontal curve to reach the paved road on the top of bank. Placement of the road was completed with a few soldiers in about 4 h using one electric impact wrench to install the anchors (Figure 7).



FIGURE 6 A 96-ft-long section of ACE Mat being placed on a horizontally and vertically curved route up a river bank.



FIGURE 7 Army soldiers placing the fiberglass mat up a riverbank using only an electric impact wrench.

The fiberglass road significantly improved traffic ability by the off-road vehicles. Trucks that struggled to climb the bare soil with all-wheel drive engaged could now quickly climb the bank using only rear-wheel drive. Rutting and tire-induced erosion were nearly halted and recovery of stuck vehicles was no longer necessary.

Tracked vehicles with road pads have been tested previously on the fiberglass mat without issue. However, during this field test, a bulldozer with steel tracks crossed the mat to drive down the bank. The teeth on the bulldozer tore through the fiberglass material at the edge of the mat, and when the bulldozer was fully on the mat, the friction between the steel and fiberglass was too low on the steep grade, causing the bulldozer to slide several feet down the bank (Figure 8). The damage observed indicates that the use of the mat should be limited to vehicles with rubber contacts only, either tires or tracks with road pads. Steel tracks, such as on bulldozers or excavators, must not be allowed to use the ACE Mat road.

OBSERVATIONS FROM FIELD PLACEMENTS

Installation Rate

During the tests at the ERDC, researchers measured the installation time of the ACE Mat system. Installing a 102-ft-long by 12-ft-wide section required approximately 1 h for two persons. This placement included 34 panels. While the 34 panels can be laid in under 10 min, the majority of the required time is for anchoring the panels using the screw anchors. The time for recovering the system is very comparable to installation time. Installation time can be greatly reduced by increasing the number of personnel used.

By comparison, installing gravel over a weak soil can be performed in less than half the time. Assuming a 1-ft-thick layer of gravel is placed as a temporary roadway, the same width and



FIGURE 8 Damage to ACE Mat from bulldozer, Fort Knox test site.

length of road (102 ft x 12 ft) requires three 15-yd-capacity dump trucks. If all three trucks arrive at the site at the same time, minimal time is required to lay the gravel. Each truck empties its load that then is spread by a separate piece of equipment. This process can fairly easily be performed in half an hour.

Cost

Each individual panel of ACE Mat costs approximately \$250 to manufacture, based on the current market. Additional costs are required for connection systems and soil anchors. For the scenarios used in this study, the screw-type anchors were used, and the number of anchors required equals 1.5 times the number of matting panels when installing a two-panel-wide road system, since one anchor can secure the corner of multiple panels. These anchors cost approximately \$80 each. Based on the 102-ft geometry, the total raw material cost for using the ACE Mat system is \$12,580. Since minimal equipment is needed, the installation cost is assumed equal to the labor cost. Assuming a rate of \$15 per hour, the labor cost for this section is \$30 using two personnel.

By comparison, the raw material cost for the gravel design is that of three trucks of gravel. Assuming \$300 per load, the total cost of the gravel is \$900. Additional costs include the spreading equipment and operator labor. Assuming a typical weekly rental rate for a small spreading device (bulldozer, front-end loader, skid steer, etc.) of \$2,500, the daily rate would be \$500. If the operator could spread the area in 20 min, the effective rate for this length of gravel would be \$21, and the labor cost for this time could be assumed to be \$9, resulting in a total installation cost of \$30, the same as for the matting solution.

If these costs were extrapolated for a mile of temporary roadway, the total cost for the ACE Mat would be \$652,750 while the gravel option would be \$48,150. Assuming that the gravel is left in place, the recovery cost is equal to zero while the recovery cost for the ACE Mat is equal to the labor cost of \$1,550. The second time the matting is used, its total cost is now equal to the initial installation cost, the recovery cost, and the labor cost to re-install which equals \$655,850. The gravel option cost simply doubles for a total of \$96,300. Repeating the process results in the gravel solution's not surpassing the cost of the matting solution until the matting is used the 15th time. One of the major influences in the cost comparison is the anchor system used. Multiple types of anchors could be substituted, including bullet, arrowhead, or other types of cabled anchors. Additionally, an alternative connection system could be used to join matting panels, resulting in a greatly reduced quantity of required anchors. Assuming the anchors were substituted with a version that cost \$10 each, the cost comparison could result in the matting option having a reduced cost after the 10th installation. Anchors receive little wear during installation and use; on average, an individual anchor can be used dozens of times before it must be replaced. The fiberglass mats receive some wear around the hole when installed incorrectly. Which proper installation, the mats can be used around 15 times before needing to be replaced as use does not cause significant deterioration of the mats.

Logistics

Matting systems achieve favorable comparison to alternative approaches such as placing gravel for a temporary road when the logistical requirements involved are compared. A single flatbed trailer has the capacity to carry enough matting to cover 0.25 mi of road, assuming a two-panel

wide design. The resulting load would be 440 mats with a total weight of 52,800 lb. By comparison, the same 0.25 mi of roadway would require 39 dump trucks (15-yd capacity) to place a 1-ft-thick layer of gravel. Because the system is manually emplaced, little equipment is needed to support installation. The matting panels can be unloaded using a forklift or by hand. They can also be transported using smaller lightweight equipment in areas constrained by road geometry or soil strength. Additional equipment depends on the type of anchor used but is most likely hand tools. Specialized skillsets are not required for the labor force tasked with installation.

Flexibility

Many heavy-duty mats are unable to handle curves in the roadway and can be used only on straight sections. However, lightweight fiberglass mats can be placed to create any road geometry. Their physical ability to flex allows them to handle gentle changes in vertical curvature, while the open holes in the edges of the panels allow for creation of horizontal curves. Additional holes can be drilled into the mats using a cordless drill and 2-in. hole bit to allow anchors and connectors to be placed anywhere on the mat to ensure secure placement.

Environmental Impact

One of the most unique advantages of mats compared to gravel roads is the ability to quickly, completely, and cheaply remove the roadway after a project is completed. This provides two major environmental advantages. Road improvement is needed in areas where the soil is weakest; this often means it crosses creeks or marshes. By removing the road when traffic is no longer needed, there is no risk of blocking water paths with large quantities of gravel that can cause localized flooding. Additionally, removing these roadways discourages unauthorized use of and limits personal vehicles from traveling into unauthorized areas.

CONCLUSIONS AND RECOMMENDATIONS

While the use of traditional gravel roads is simple and has its place in many low-volume road applications, using lightweight fiberglass mats for temporary roads allows agencies flexibility to quickly reach inaccessible locations when needed and can be removed and reused later. Logging companies, forestry agencies, and other entities that require such occasional access to remote locations near weak soils should consider using ACE Mat when existing road conditions are insufficient for truck traffic, but where logistic or environmental concerns restrict the practical use of gravel road improvements.

ACKNOWLEDGMENTS

The demonstrations presented in this paper were conducted by the ERDC Geotechnical and Structures Laboratory, Airfields and Pavements Branch, with funds and support provided by the Federal Highway Administration and U.S. Army Tank–Automotive & Armaments Command. The support of ERDC and U.S. Forest Service personnel is gratefully acknowledged.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: Study Conception and Design: Daniel Harder and Lyan Garcia; Data Collection: Daniel Harder and John Rushing; Analysis and Interpretation of Results: Daniel Harder, John Rushing, and Lyan Garcia; Draft Manuscript Preparation: Daniel Harder, John Rushing, and Lyan Garcia. All authors reviewed the results and approved the final version of the manuscript.

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Monitoring the Performance of Unreinforced and Geosynthetic-Reinforced Pavements on Forest Roads

JUSTIN LONG
ALEX NAGYGYOR
TOM SZYMONIAK

Washington Department of Natural Resources

INTRODUCTION

The performance of logging roads is greatly impacted by the bearing capacity of the subgrade material. Subgrade bearing capacity on saturated forest roads is often poor when the material is a clay or silt. In order to compensate for the low strength of the subgrade, engineers use a variety of techniques. The most common technique is applying additional aggregate to the subgrade. Engineers frequently use historical data to determine appropriate aggregate depths. While this may be suitable where aggregate is abundant, using alternative methods based on specific site characteristics may provide cost savings where aggregate resources are limited.

Several methods including computer programs, tables, and nomographs consider soil and aggregate bearing capacity in determining aggregate depth. The Washington Department of Natural Resources (DNR) Forest Roads Program has developed an aggregate depth design tool to assist engineers in determining the appropriate depth of aggregate based on site-specific characteristics. The program is an Excel-based computer model based on an algorithm developed by the U.S. Army Corps of Engineers in 1978. This equation considers the level of traffic, quality of materials, and critical performance standards (Barber et al., 1978).

Geosynthetic reinforcement (geotextile fabrics and biaxial and triaxial geogrids) may be used to provide additional bearing capacity over weak subgrades and provide a separation layer between the aggregate and the subgrade. Geogrids are used extensively in civil engineering for slope stabilization, soil erosion control, and reinforcement of weak soils. While nonwoven geotextile products are often used for separation, they do offer reinforcement in addition to separation.

The DNR Forest Roads Program developed an aggregate study to evaluate the performance of four different subgrade and aggregate configurations and determine whether an opportunity exists to either increase performance or reduce road cost. In addition we wanted to answer the following questions:

- What is the difference in road performance between alternative depth design methods?
- Can geosynthetic-reinforced sections on soft subgrades reduce the depth of aggregate?
- Is there a potential to reduce road construction costs by using alternative depth design methods?

METHODS

We evaluated the performance of four subgrade and aggregate configurations (Table 1) on a wet weather constructed road in Lewis County, Washington. Performance was measured by two criteria: (1) The change in elevation of the running surface over time, and (2) the rut depth which was considered as the difference in elevation between the wheelpath and centerline over time. Road surface geometry was monitored using a total station over a 2-week period of haul.

Subgrade soils consisted of thrash silty clay loam, which is a medium to high plasticity silt (MH as defined by the USCS). This material when saturated has very low bearing capacity and requires a greater thickness of aggregate as compared to gravels or sands.

Field measurements were performed on the constructed subgrade to determine the aggregate depths for the geogrid, geofabric, and DNR Aggregate Depth Design Tool treatments. Prior to construction 10 dynamic cone penetrometer (DCP) tests were performed along the treatment area at a minimum depth of 12 in. to determine subgrade bearing capacity. DCP values were converted to a California bearing ratio (CBR) using published literature and established correlations.

CBR is one of the parameters used in pavement design. The measured, average value of CBR = 2.7 was used to describe subgrade bearing capacity. CBRs below 5 are considered very poor subgrades. The total traffic was estimated using total equivalent 18-kip single-axle loads (ESALs). Allowing for log haul, mobilization, and other vehicle traffic an estimated 1,300 ESALs was used in the design analysis. The data determined the aggregate thicknesses listed in Table 1.

A total of 210 loads were counted using a tree-mounted motion-detecting game camera. Rut depths were measured as the change in elevation of the running surface due to vehicle use. Hubs were placed on both sides of the road at an interval of 25 ft to determine measurement locations at each road segment. These locations served as permanent measurement plots throughout the life of the study. Points were collected between hub locations using a total station periodically during the 2-week period of timber haul.

A total of 1,139 data points were collected over a period of 4 days from March 21, 2018, to April 16, 2018. Point data was used in AutoCAD CIVIL 3D to create surface profiles to model rut depth over time. After log-hauling was complete a destructive survey was performed to determine the final shape of the subgrade. A mini excavator was used to remove the aggregate layer down to the final subgrade location. Points were taken at each location both before and after the aggregate was removed.

TABLE 1 Aggregate Treatment Descriptions

Treatment	Stations	Aggregate Depth (in.)	Rut Depth (in.)
13-ft biaxial geogrid (BXG120)	3+00 to 4+50	8	2.3
15-ft nonwoven geotextile (HP270)	4+50 to 6+00	12	6.4
Aggregate depth design tool	6+00 to 7+50	16	5.2
Engineers design in road plan	0+00 to 3+00, 7+50 to 8+70	16	5.2

FINDINGS

Rutting exceeded the critical depth of 2 in. on all of the treatment sections prior to the end of construction on April 2. A total of eight loads of spot rock were placed on April 9 as rut depths exceeded 6 in. on the geofabric, DNR Aggregate Depth Design Tool, and the engineer’s design sections.

The geogrid section performed significantly better than the other treatments. An average rut depth of 2.3 in. was measured with an aggregate depth of 8 in. The DNR Aggregate Depth Design Tool and the engineer’s design sections each had an average rut depth of 5.2 in. with an aggregate depth of 16 in. The geofabric section had the highest average rut depth of 6.4 in. with an aggregate depth of 12 in. Average rut depths by treatment are shown in Table 1.

Surface profiles, shown in Figure 1, show one of two distinct patterns. First, a W shape developed in the running surface eventually causing the subgrade to heave through the centerline above the aggregate layer, and second the running surface settled as a unit causing heaving on the edges of the road.

The heave pattern was observed in the geofabric, DNR Aggregate Depth Design Tool, and engineer’s design sections, but more pronounced in the geofabric section. Figure 2 shows a section of roadway experiencing heave, where the subgrade at centerline had pushed above the running surface exposing mineral soil at centerline (also known as subgrade failure). Although roads with heaving may still be drivable with a log truck there are increased risks for sediment delivery. First, the deep wheelpaths can channelize water, and second, exposed mineral soil can be tracked by machinery and mobilized by precipitation into the channelized wheelpath.

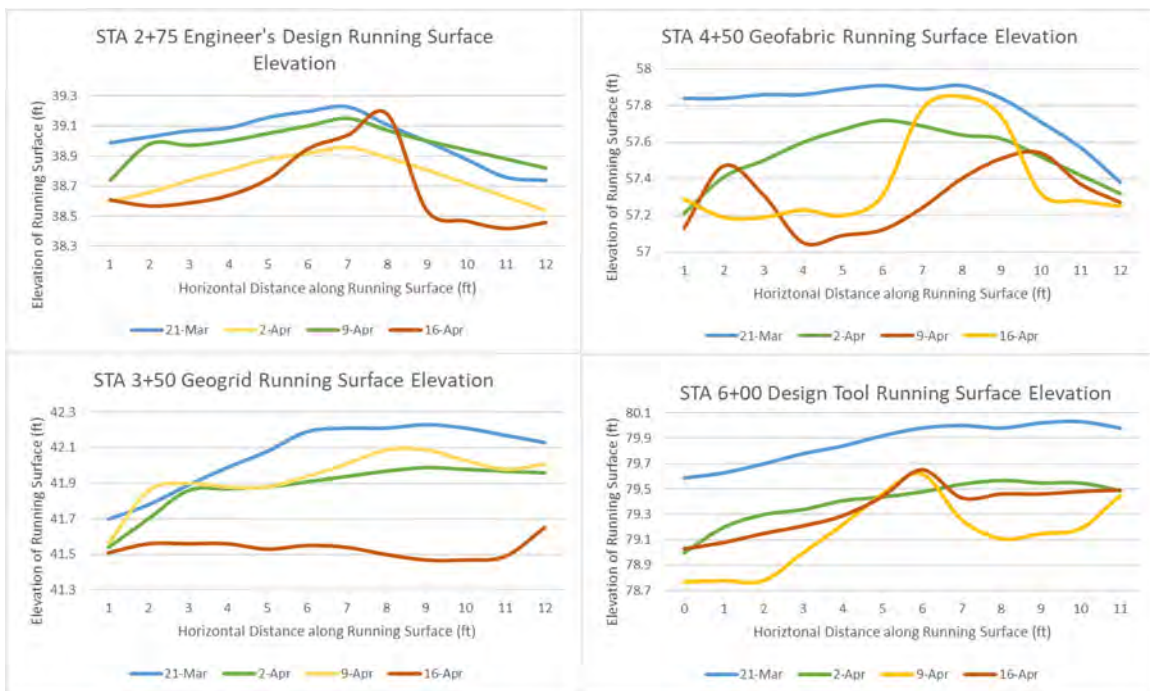


FIGURE 1 Surface profiles.



FIGURE 2 Centerline heave.

The settling of the entire running surface is shown in Figure 3. This phenomenon was observed exclusively in the geogrid section. In this case the entire running surface migrated downward as a single unit. While the running surface was sinking, it was still holding its original shape giving the appearance of little to no rutting. This was caused by the reinforcing properties of the geogrid. Instead of a localized bearing failure in the wheelpaths the grid held the layer together as a single unit. The weight of the geogrid, aggregate, and loads surpassed the bearing capacity of the soil beneath it causing the entire layer to drop. This process was usually observed with heaving outside of the geogrid edge. While there was no heaving of the subgrade above the running surface, the sinking of the entire road did not allow water to drain off because the road was at the lowest point.

After haul was complete a destructive survey was performed to determine the final shape of the subgrade. A mini excavator was used to remove the aggregate layer down to the final subgrade in six locations. Photographs were taken of the final subgrade and used to create a 3D model using RECAP 3D modeling software. The models verified the observations from the surface profiles. Four of the six models showed bearing failure beneath the wheelpath and heaving of the centerline (geofabric, DNR Aggregate Depth Design Tool, and engineer's design). In each of the four locations, the subgrade had heaved above the running surface at centerline (Figure 3). The stresses were so significant that the geofabric failed (was torn) at all heave and wheel track intersection points. The two geogrid sections, however, showed the subgrade retaining its original shape, while heaving was observed outside of the geogrid edge (Figure 4). Some geogrid segments failed, which appeared to be a result of wrinkles in the grid due to construction techniques. The wrinkles were a result of vehicles traveling on uncompacted



FIGURE 3 Outside heaving of the geogrid treatment.



FIGURE 4 Heaving and rupture of the geofabric section.

aggregate during aggregate application. The wrinkles limited the ability of the geogrid to distribute the vehicle load to the subgrade resulting in subgrade failure

CONCLUSION

Three of the four treatment sections had rut depths significantly greater than the 2 in. failure criterion. The geofabric section had the greatest average rut depth at 6.4 in., while the DNR

Aggregate Depth Design Tool, and the engineer's design had average rut depths of 5.2 in. Each of these sections, however, remained passible by log truck traffic throughout the life of the Topper Sorts Timber sale.

Season of construction and haul plays an essential role in determining aggregate depth. Ideally construction and aggregate placement should occur during the summer months. Road construction in the drier months allows for subgrade and aggregate compaction, thus requiring less aggregate compared to similar soil in wet weather construction. Therefore, when possible construction in unreinforced marginal and weak soils such as silts, clays, and organic soils should be limited to the summer months.

Wet weather construction and haul frequently occurs on forest roads. However, wet weather construction and haul often results in lower bearing capacity of the subgrade and aggregate, and requires greater aggregate quantities to maintain timber haul. Limiting wet weather construction to stronger soils such as sands and gravels can be a method for conserving aggregate resources in wet weather construction. Marginal and weak soils such as silts, clays, and organic soils may be reinforced using geosynthetic materials particularly in wet weather construction and haul.

The use of geosynthetic reinforcement provides designers with the opportunity to improve strength characteristics of weak and marginal soils. Improving strength characteristics allows for; reduced aggregate thickness on weak and marginal soils, which results in cost savings for road construction, thereby prolonging the lifespan of dwindling aggregate resources.

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Mechanistic Analyses of Geosynthetic-Reinforced Low-Volume Rural Roads

JEB S. TINGLE

U.S. Army Corps of Engineers

Large-scale laboratory box tests and a full-scale traffic test were performed by the U.S. Army Engineer Research and Development Center to evaluate the performance of geosynthetic-reinforced aggregate road sections constructed with marginal base materials over a typical subgrade. The large-scale laboratory testing and full-scale test section included eight different instrumented aggregate road sections including three different aggregate base materials and two different geosynthetics. Mechanistic analyses of each pavement section were conducted using linear elastic, nonlinear elastic, and nonlinear anisotropic models to predict the critical pavement response parameters. The analyses show that mechanistic tools can be effectively used to estimate the critical pavement response parameters for unpaved roads.

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Experimental Studies on Coir Geotextile-Reinforced Low-Volume Rural Roads

V. SUNITHA

MATHEW SAMSON

V. ANUSUDHA

BABU CHITHU

National Institute of Technology, Tiruchirappalli, India

INTRODUCTION

To satisfy the requirements of low-volume rural roads in India, at times the road network has to be aligned through places where the subgrade soil is not suitable for road construction. Construction and maintenance of roads along the unsuitable soils have been problematic due to their inherent potential for volume change in the presence of water, which affects the performance of roads. The subgrade has to be properly treated at the construction stage itself. Otherwise, the road user and maintenance costs will increase substantially due to the deteriorated pavement performance.

One way to treat subgrade is to reinforce it with geotextiles. Natural geotextile, particularly coir geotextile, is being recognized as an ideal material that is capable of offering an environmentally friendly and ecologically sustainable solution. Coir geotextiles are ideally suited for low cost applications because coir is available in abundance at very low price compared to other synthetic geotextiles.

This study focuses on the interface behavior of the coir geotextile between subgrade and granular subbase layer. This will be helpful in evaluating the strength properties of pavement section with and without coir geotextile and ultimately in design of an economical section for a particular geotextile.

Many researchers have studied the behavior of geotextile-reinforced pavements. The behavior of two types of jute geotextiles treated with copper sulphate under various laboratory tests were studied (1). Rut depth reduction using jute geotextile as reinforcement at interface of subgrade and subbase or base layer was observed. Fatigue test indicated that pavement reinforced with jute can reduce surface and subgrade deformation by 40% and 65% respectively for 106 repetitions of load. The monotonic plate load test and repeated load test conducted on test section showed that the plastic surface deformation under repeated loading was greatly reduced by the inclusion of coir geotextiles within the base course irrespective of base course thickness (2). A study on strength of granular subbase (GSB) underlain by a subgrade layer in terms of California bearing ratio (CBR) and bearing capacity reported that as GSB thickness increases, CBR increase from 2.5% to 55% for soil to GSB and the corresponding shear strength from 0.91 kg/cm² to 1.75 kg/cm² (3).

Most of the literature were based on laboratory plate load tests. It is very difficult to obtain the same field condition in a laboratory and thus results were varying. Also compacting the subgrade to required density and optimum moisture content for pavement construction in a

tank in the laboratory is questionable. Some of the above studies assumed the pavement as two-layer system, which is not the case in actual field.

The objective of this study is to compare the results obtained in the field by the conduct of plate load test and Benkelman beam deflection (BBD) test for pavement sections with and without coir geotextile. Also the pavement section with coir is with reduced thickness of granular subbase. In this study, field plate load tests are used which gives more reliable results than laboratory results. Benkelman beam method is also used to find the rebound deflection in reinforced and unreinforced pavement sections.

The plate load test results indicated that the inclusion of H2M6 coir geotextiles increased subgrade reaction value when compared to that without coir geotextile. BBD results indicated that the deflection values for coir reinforced sections were lesser than that of the Control Section.

METHODOLOGY

Plate bearing test is conducted to calculate the modulus of subgrade reaction. Plate load testing consists of applying a load to a pavement through a rigid circular plate and measuring the deflection produced by this load. Modulus of subgrade reaction is the reaction pressure sustained by the soil sample under a rigid plate of standard diameter per unit settlement measured at a specified pressure or settlement. The K -value has been measured at 1.25-mm settlement. A graph is plotted with the mean settlement (mm) on x -axis and load (kN/m^2) on y -axis. The pressure, p , corresponding to a settlement of 1.25 mm is obtained from the graph. The modulus of subgrade reaction K is calculated from the following formula:

$$K = \frac{P}{0.00125} \text{ kN/m}^3$$

A heavy reaction load is required for soils with high K -value when a plate of diameter 75 cm is used for testing. In this study 30-cm diameter plate was used and thus corresponding corrections were done.

BBD readings were taken at every 25-m interval as per IRC 81-1997. The point on the pavement to be tested was selected and marked. The point selected was 60 cm from the pavement edge. The initial reading is recorded when the rate of deformation of the pavement is equal or less than 0.025 mm per minute. An intermediate reading is recorded when the rate of recovery of the pavement is equal to or less than 0.025 mm per minute, when the truck was at a distance of 270 cm from the point. The final reading is recorded when the truck was at a distance of 900 cm from the point. Pavement temperature and the tyre pressure were checked at 2- or 3-h intervals during the day.

Apparent deflections were corrected by means of the following formula:

$$X_T = X_a + 2.91Y$$

where

X_T = true pavement deflection;

X_a = apparent pavement deflection; and

Y = vertical movement of the front legs.

The rebound deflection is the twice of the X_T value. Temperature and moisture corrections were done and the results were compared with those of last year.

This paper proposes the practical implementation of improvement of weak subgrade soil by using coir geotextile. Under Bharat Nirman Phase III, 18 roads (total length of 37.88 km) have been constructed with coir geotextile reinforcement. Of those, Narasipuram to Poondi road (length of 3.786 km) of Thondamuthur block in Coimbatore district has been studied in this paper. Figure 1 shows the plan and longitudinal section of the selected site.

The total length of the road was divided into three sections as shown in the Figure 1. This was done to analyze the performance of coir reinforced pavements with sections of different thickness and types of coir geotextiles. Coir geotextiles H2M5 and H2M6 of widths 6 and 8 m were used. The Control Section is laid without coir geotextile and also sand soling was provided as the subgrade was weak. The thickness of the GSB in Control Section is 175 mm. The Coir Section I is with coir geotextile of various types and widths laid above sand soling. In this the GSB thickness has reduced to 125 mm. In Coir Section II, the coir geotextile is directly laid above the subgrade and also the thickness of the GSB is 125 mm without providing sand soling.

Of these sections below results are discussed for Control Section and Coir Section I with H2M6 (6-m width). The plate load test results indicated that the inclusion of H2M6 coir geotextiles increased subgrade reaction value when compared to that without coir geotextile. BBD results indicated that the deflection values for coir reinforced sections were lesser than the Control Section.

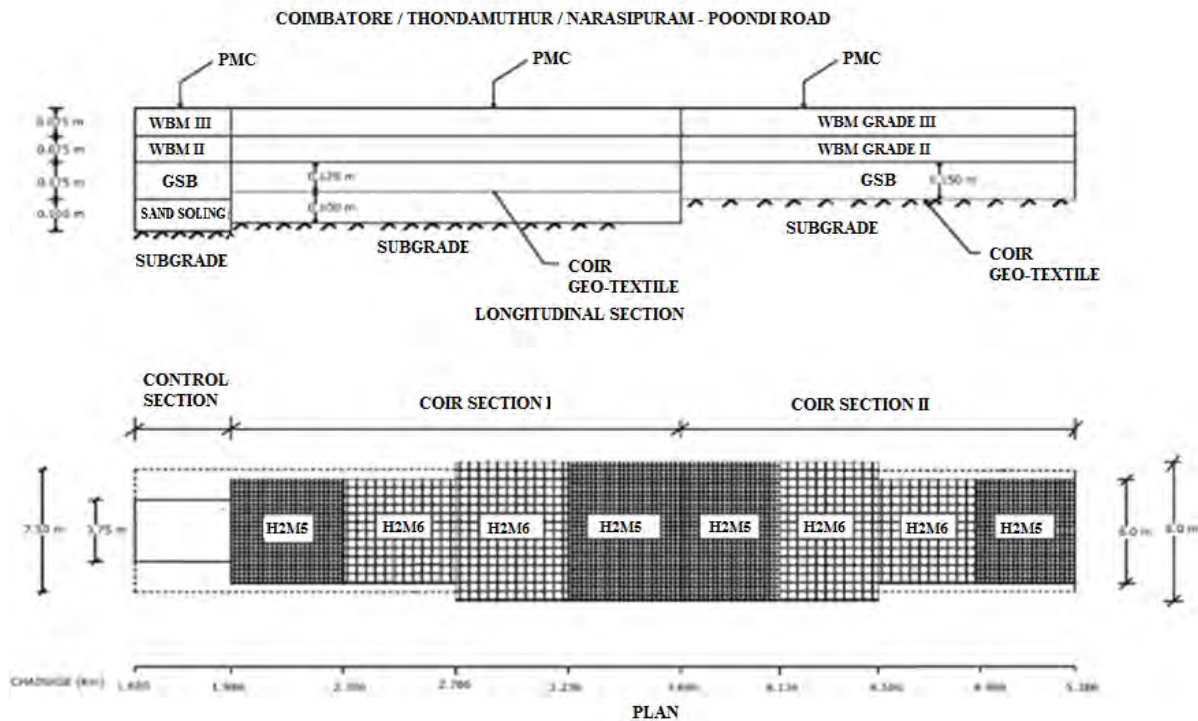


FIGURE 1 Plan and longitudinal section of Coimbatore/Thondamuthur/Narasipuram–Poondi Road.

FINDINGS

Plate load tests were done in Control Section (above sand soiling) and in Coir Section 1 where H2M6 is used (above the coir geotextile). Table 1 shows results of the plate load tests.

The K -value of the Coir Section with H2M6 coir geotextile was 0.75 MPa/cm and is higher than the Control Section K -value of 0.36 MPa/cm. BBD readings were taken at every 25-m interval and results indicated that Coir Section with H2M6 was having lesser deflection of 0.738 mm compared to Control Section having a deflection of 1.334 mm. The above two results emphasised the effectiveness of coir geotextile in the field even with the reduced thickness of GSB layer.

CONCLUSION

This paper summarizes the study carried on the reinforcement behavior of the coir geotextile when used in the interface of subgrade and subbase. The plate load test has been conducted in the field on subgrade soil and modulus of subgrade reaction of the soil was studied. The specific findings from the field test are:

1. By conducting plate load test above the sand soling in Control Section, the K -value obtained is 0.36 MPa/cm. The same test when conducted above H2M6 coir geotextile in Coir Section acquired a K -value of 0.75 MPa/cm, which is almost double that of Control Section.
2. The rebound deflection by BBD test measured a value of 1.334 mm for Control Section. But on the Coir Section with H2M6, the deflection value was reduced to 0.738 mm.

The plate load test and BBD test in the field revealed the efficiency of providing coir geotextile even by reducing the thickness of granular subbase.

ACKNOWLEDGMENT

The authors acknowledge the Central Coir Research Institute, Coir Board, Ministry of Micro, Small and Medium Enterprises, Government of India for the sponsored research project Pavement Performance Studies on Coir Geotextile-Reinforced Rural Roads in Tamilnadu.

TABLE 1 Results of Plate Load Tests

Chainage (m)	Plate Load Test		BBD Test
	Position of Plate	K -Value (MPa/cm)	Rebound Deflection (mm)
2.600 (CS 1- H2M6)	H2M6 coir underlying sand soling and subgrade	0.75	0.738
1.800 (CS)	Sand soling above the subgrade	0.36	1.334

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Enhancing Pavement Performance Using Tire-Derived Geocylinders for Low-Volume Roads

P.V. VIJAY

JUSTIN S. SMITH

West Virginia University

Pavements are subjected to many forms of stress during their service life including cracking and failures. Many of the failures can be attributed to poor subgrade, freeze–thaw cycle variations, and fatigue under repetitive axle loadings. Fatigue related cracking can lead to rutting, pot holes, and cracks in the wheelpath, etc. One way to improve pavement performance is to enhance the mechanical properties of the base–subbase. This research investigates mechanical concrete comprised of waste tire-derived geo-cylinders (TDGCs) filled with coarse aggregates to improve mechanical properties of base/sub-base for low volume roads. Mechanical concrete helps in creating a more stable platform for the pavement system through confinement effects and distributes the stresses from axle loads to the ground and increases the base/sub base moduli. This paper focuses on the presentation and discussion of laboratory scale test results conducted on representative pavement sections consisting of mechanical concrete with different tire diameters and its effectiveness in enhancing the pavement base/subbase moduli under both static and fatigue loads up to 3 million cycles.

INTRODUCTION

P

Mechanical concrete consists of thin-walled recycled tire cylinders that are filled with coarse aggregates. The cylinders are derived from waste automobile tires by stripping their side walls for use in pavement systems and several other applications. The confined aggregate provides a stiffened base–subbase and helps to strengthen the pavement against applied vehicular loading particularly for low volume roads. The objective of this research work is to evaluate the material and system behavior of the mechanical concrete pavement system. The typical standard size of a thin-walled tire-derived geo-cylinder (TDGC) is approximately 600 to 700 mm in diameter with an approximate height of 200 to 225 mm. The TDGC are recommended to be filled with aggregates meeting AASHTO #57 aggregates. TDGCs have been implemented in multiple settings exposed to large vehicular traffic, such as oil and gas drilling well-pads and road shoulder reinforcement in states such as West Virginia and Texas. Mechanical concrete is described to have performed better under field applications (Bonasso, 2013).

Several factors control failure modes in flexible–rigid pavements. These factors include:

- Insufficient compaction;
- Moisture presence in the subgrade;
- High-intensity loads;
- Freeze–thaw cycles; and

- Various other in-situ and constructability issues.

Constructability constraints can be attributed to the lack of adherence to pavement construction procedures in terms of suggested minimum temperature ranges, inadequate compaction, and poorly managed construction rate. This article provides understanding of enhancement of base–subbase properties through the use of mechanical concrete.

CONSTRUCTION OF LARGE AND SMALL TEST BINS FOR REPRESENTING PAVEMENT SECTIONS

The materials discussed in the following section relate to the construction and material makeup of the bins utilized for testing. Two bins, a 2.1- x 3.7-m (7- x 12-ft) large bin and a 1.2- x 1.8-m (4- x 6-ft) small bin, were constructed. A test bin replicating the field cross-section was necessary to evaluate the effectiveness of TDGCs used for strengthening pavement properties. Construction details of the 2.1- x 3.7-m bin are shown in Figure 1.

Representative roadway section wooden bins were constructed using 50- x 250-mm (2- x 10-in.) dimension lumber and reinforced with appropriate vertical and angle bracing schemes to support loading conditions. The interior of the constructed bin was lined with heavy-duty polyethylene sheeting. A skid steer was used to move the soil and aggregate into the large bin. The large bin was filled and compacted with soil and compacted in three 150-mm (6-in.) lifts. Following each lift, a vibratory plate compactor was used to achieve desired compaction. Lower degree of compaction of used in large bin and higher compaction was used in the small test bin as per AASHTO 204.11 (2018). A woven geotextile separation fabric was placed between the compacted soil and TDGCs. The TDGCs were placed uniformly throughout the bin. Using a skid steer, AASHTO #57 aggregates were placed inside the bin on one side. The aggregates were filled within and between the TDGCs using garden tools to a height of approximately 50 mm (2 in.) above the top surface of TDGC. The smaller bin with dimensions of 1.2 x 1.8 m was built in a similar fashion. Static and fatigue tests were conducted with TDGCs with the use of these representative roadway sections.

SOIL TESTING

Locally available soil was used as subgrade in the pavement test bin. The soil specimen information was obtained based on the National Resources Conservation Service (NRCS) and the U.S. Department of Agriculture (USDA). Those sources indicated that roughly 26% or more of the soil located in Monongalia County, West Virginia, is derived from silty sandstone material.

Tests such as Atterberg limits, sieve analysis (grain size distribution), specific gravity, nuclear gauge density, and soil classification analyses were performed on the soil specimens in the Geotechnical and Structural Engineering Labs at West Virginia University. The soil sample used in the bin was obtained from the same in-situ soil (subgrade) location for all the tests. The soil was classified in accordance with the American Society of Testing and Materials (ASTM). Based on the testing, the in-situ soil specimen was classified as a silty sand (SM) (ASTM D2487) with a specific gravity of 2.67. Atterberg limit test (ASTM D4318) was



FIGURE 1 Bin construction: (a) frame construction; (b) polyethylene sheet-lined wooden bin; (c) soil placement; (d) soil compaction, three 150-mm lifts; (e) placement of separation woven geotextile fabric on compacted soil; (f) placement of TDGCs over separation fabric; (g) placement of AASHTO #57 aggregate as infill for the TDGCs; and (h) aggregates placed 50 mm above TDGC top surface and ready for testing.

conducted and the liquid limit (LL), plastic limit (PL), and plasticity index (PI) were found to be 47, 30, and 17, respectively, which were used to calculate the LL (Figure 2).

Based on the Atterberg limit testing, data was plotted corresponding to the activity line (A line). Knowing the PI and LL, the soil was observed to fall below the A line. From the USCS-based Plasticity Chart, a soil is classified as SM if more than 50% is retained on the No. 200 sieve and the criteria of $PI < 4$ or the plot is below the A line.

Grain size distribution of the soil sample was performed by using necessary sieves (as per ASTM D422 and AASHTO guide), sieve shaker, and mortar and pestle. A grain size distribution curve was developed to represent the percentage of passing particles as shown in Figure 2b and a good distribution of particle sizes was noted.

A specific gravity test was performed as per ASTM D854-Method B, which specifies the use of an oven-dried specimen for the procedure. Temperature dependent specific gravity is given by Equation 1:

$$\rho_s M_s G_t = \rho_{w,t} \frac{M_{pwt} - (M_{\rho w s,t} - M_s)}{M_{pwt} - (M_{\rho w s,t} - M_s)} \quad (1)$$

where

- ρ_s = the density of the soil solids Mg/m^3 or g/cm^3 ;
- $\rho_{w,t}$ = the density of water at the test temperature, g/mL or g/cm^3 ;
- M_s = the mass of the oven dry soil solids (g); and
- $M_{\rho w s,t}$ = the mass of pycnometer, water, and soil solids at the test temperature (g).

Within the bin representing pavement section, soil was compacted as explained in the next section and the density and moisture content data were found from nuclear gauge testing. Table 1 provides density and moisture data at locations in the large and small test bins that were constructed to represent pavement (roadway) sections.

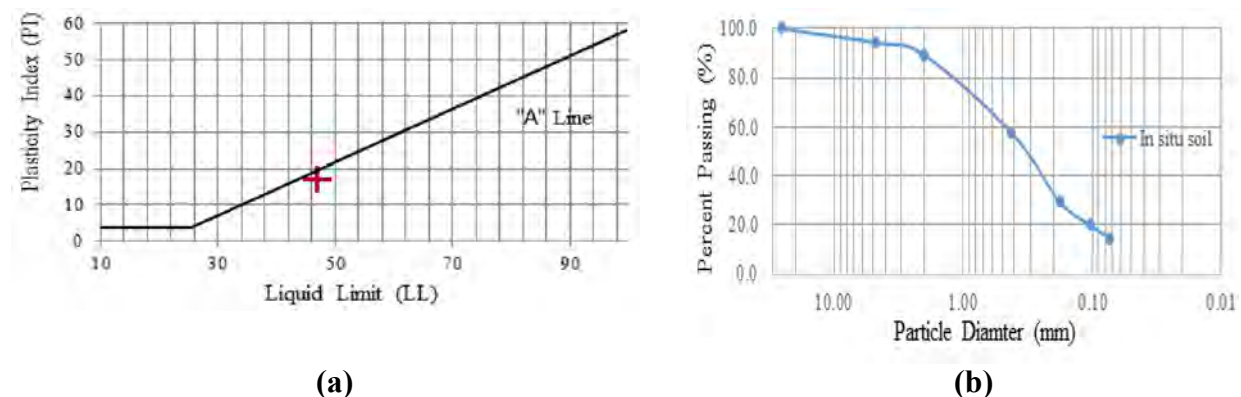


FIGURE 2 (a) Plasticity chart of in situ soil specimen, A line, and (b) grain size distribution of subgrade soil used in the wooden test bins. [Note: A line on the plasticity chart is a sloped line beginning at $PI = 4$ and $LL = 25.5$ with an equation of $PI = 0.73 (LL - 20)$.]

TABLE 1 Subgrade (Soil) Testing Data Acquired from Nuclear Gauge Testing (1 pcf = 0.157kN/m³)

Details	Soil Sample Locations						
	Large Bin					Small Bin	
Nuclear gauge test	S1	S2	S3	S4	S5	S1	S2
Dry density (kN/m ³)	11.5	12.3	12.2	11.3	13.6	16.2	17.5
Wet density (kN/m ³)	14.8	15.4	15.6	14.9	15.7	17.6	19.7
% moisture	28.4	25.6	27.7	31.4	15.1	9.0	12.5

MATERIAL TESTING

The TDGC material typically consists of 34% natural rubber, 24% fillers (such as carbon black), 21% steel, 11% synthetic polymers, and 10% curing compounds (U.S. Tire Manufacturers Association, 2018). Average density values of the steel reinforced TDGC used in this study was found to be 0.00000143 kg/mm³. Tension and compression testing were performed using Instron Testing System for steel reinforced thin-walled tire specimens (Figures 3a and 4a). Tension specimens with a c/s dimension of about 9 x 29 mm and compression specimens with a c/s dimension of 25 x 23 mm were tested. Stress–strain relationships are shown in Figure 3b and Figure 4b. An average tensile stress of 22 Mpa (3.2 ksi) and compressive stress of 414.0 MPa (60 ksi) were obtained from the testing.

STATIC AND FATIGUE TESTING OF TDGC PAVEMENT SYSTEM IN TEST BINS

The large and small test bins constructed as representative pavement sections were tested through load application using an MTS hydraulic actuator. The displacement and load were measured through a linear variable differential transducer (LVDT) and 222.4-kN load cell, respectively. Data was recorded via an automatic Strain Smart 8000 data-logger system. As described earlier, the bins were filled with SM soil specimen and compacted in three lifts of 150-mm depth each. For each soil lift, a vibratory compactor was used to tamp the soil surface as per AASHTO 204.11 (2018). The degree of compaction was measured to be about 93% with the help of nuclear gauge testing. Within the bin, static load testing was conducted on the pavement section consisting of compacted soil subgrade, drainage separation woven fabric, and tire derived geocylinders filled with AASHTO #57 aggregates filled to a height approximately 50 mm above the cylinder height. Several test configurations were evaluated during this testing within the large and small bins. Three load plate diameter sizes of 610 mm (24 in.), 381 mm (15 in.), and 305 mm (12 in.) with a thickness of 25 mm were utilized to apply the loads. According to Yap (1989), the given load area for a dually truck is 73,548 mm² (114 in.²) which corresponds to the surface area of a 305 mm (12 in.) diameter plate. The 38-1 and 610-mm plates were utilized during testing to observe stresses and moduli responses of the base–subbase with regards to multiple repetitive large single tire (construction equipment) and dually tire loads.

For large bin testing, static loads were applied and the TDGCs were loaded using 600 mm (24-in.) and 375 mm (15-in.) diameter plates. Plates were loaded gradually and load versus displacements were recorded for limits of 0.635 mm (0.025 in.), 1.27 mm (0.05 in.), 1.91 mm

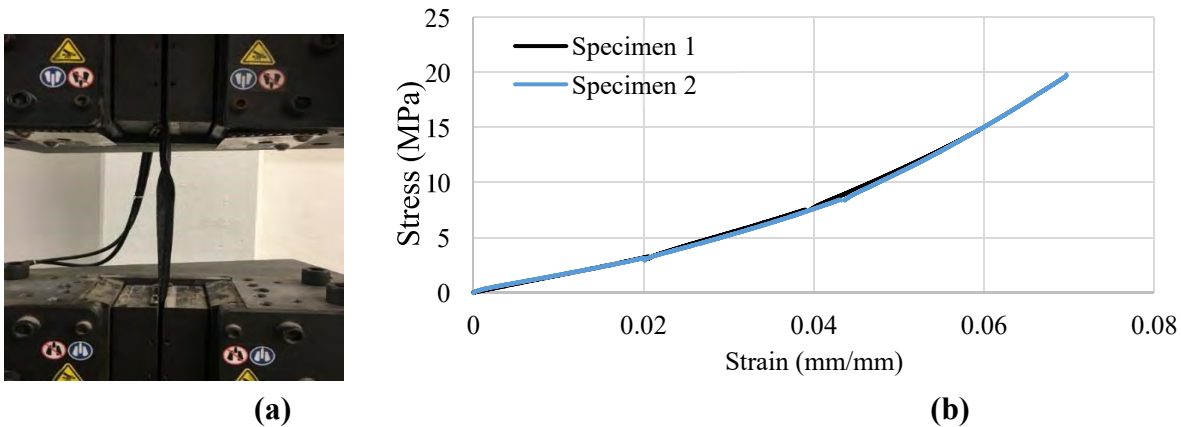


FIGURE 3 Tensile testing of TDGC specimens: (a) TDGC strip in torsion due to applied loading and (b) stress versus strain graph for tensile testing TDGC specimens.

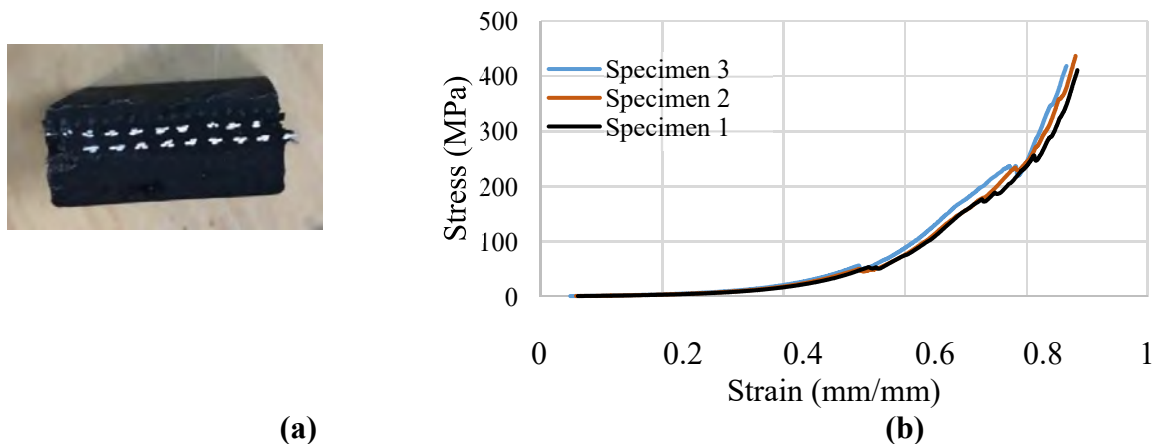


FIGURE 4 Compression testing TDGC specimens: (a) cross section of a compression specimen and (b) stress versus strain graph for compression testing TDGC specimens.

(0.075 in.), 2.54 mm (0.1 in.), and 3.175 mm (0.125 in.). Small bin testing was composed of static and fatigue testing of TDGC configurations. A displacement limit of 3.81 mm (0.15 in.) was used for each of those tests. Three tests were conducted per loading plate size per specimen. The third cycle was used for analytical purposes. Loading plate diameters of 375 mm (15 in.) and 300 mm (12 in.) were utilized during small bin static testing. Fatigue testing in the small bin consisted of loading a 300 mm (12 in.) plate at a frequency of 2 Hz. This test frequency was based on conclusions from previous research by Gillespie and Sayers (1981). They concluded that resonance of basic automobile and commercial vehicles can span from 1 to 3 Hz. By utilizing 2 Hz, this research was able to observe the fatigue loading effects of basic automobile tires and commercial vehicles on a roadway section. Two separate fatigue loading conditions were applied to the specimens, a low load range (2,224 to 4,448 N) and a high load (4,448 to 17,793 N) range configuration. Fatigue testing was conducted up to 3 million cycles. Maximum displacement setting was limited to 19 mm during the fatigue tests.

STATIC TEST DATA

Large Bin Static Testing

Large bin testing was performed on weak subgrade conditions by statically loading the 375 and 600 mm plates to displacement limits of 0.635 mm increments. The TDGCs were laid adjacent to one another and filled with AASHTO #57 aggregate to create a reinforcement mattress also known as mechanical concrete. Three TDGC configurations were evaluated and compared (Figure 5).

Within the displacement limit of 3.175 mm, increases of 113.9% and 18.3% in base–subbase moduli with respect to two different loading plate sizes (380 and 600 mm) were observed. Figure 6 shows the base–subbase moduli for the three configurations at different displacement levels.

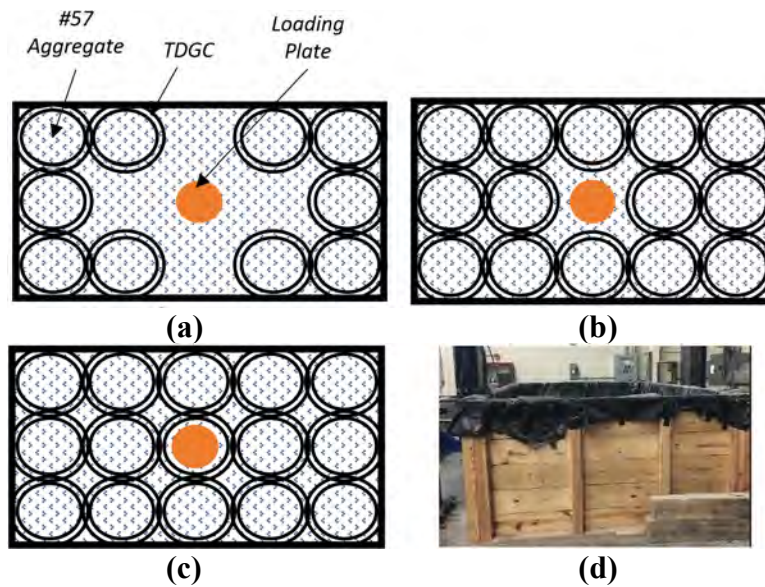


FIGURE 5 Large test bin TDGC configurations: (a) five TDGCs removed; (b) one central TDGC removed; (c) all TDGCs present; and (d) large test bin.

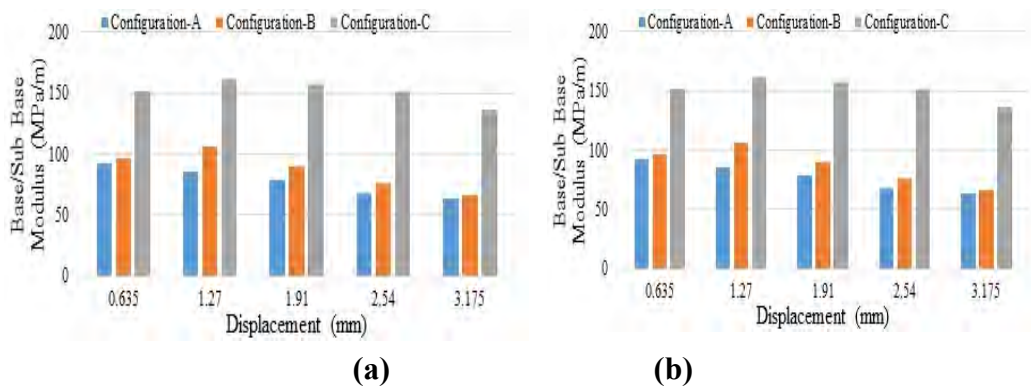


FIGURE 6 Large bin static results for (a) 375-mm load plate and (b) 600-mm load plate.

Small Bin Static Testing

Static tests were conducted on the pavement representative section in small test bin with respect to the configurations shown in Figure 7.

Base–subbase moduli enhancement with the use of a single TDGC configuration was noted to be 34% to 35% for the two plate sizes of 300- and 375-mm diameter. The overall performance of different configurations is shown in Figure 8. From the test data, the TDGC with a diameter of (650 mm) provided better confinement effect as compared to TDGC-S configurations with smaller diameter (450 mm) and without the steel belt reinforcement. Better performance by the TDGC is attributed to the presence of steel reinforcement along the circumference of the tire that increases the stiffness, as compared to TDGC-S that were not steel reinforced.

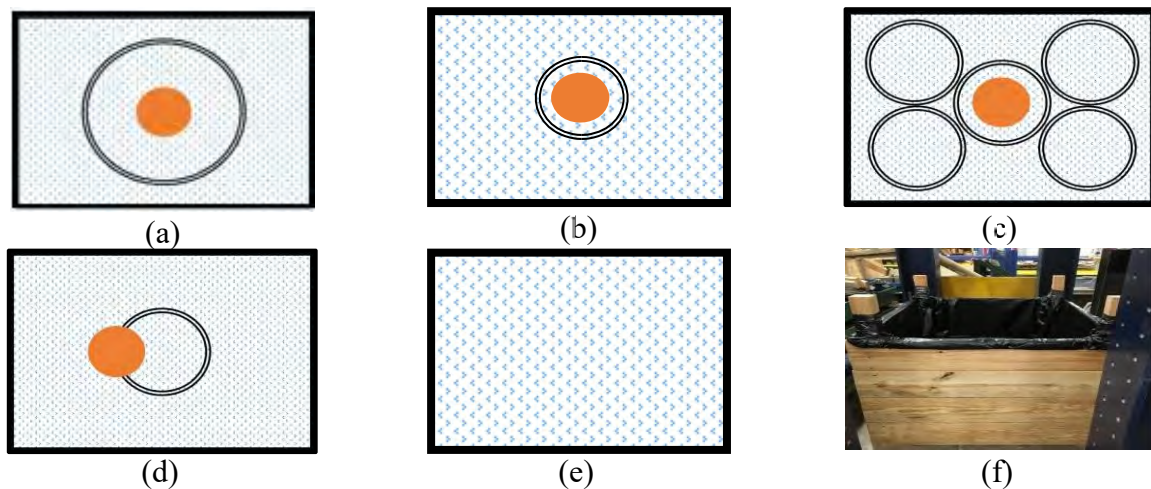


FIGURE 7 Small test bin TDGC layouts: (a) TDGC; (b) small TDGC or TDGC-S; (c) five small TDGC-S or five TDGC-S; (d) offset; (e) base condition; and (f) small test bin.

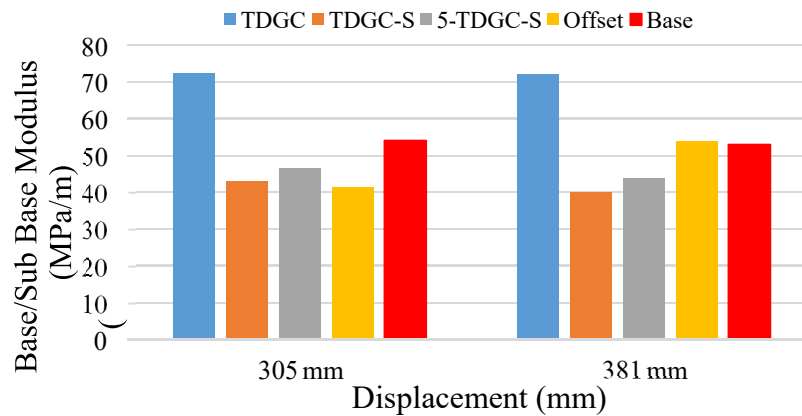


FIGURE 8 Base–subbase moduli with respect to tire-derived geocylinder (mechanical concrete) configurations and plate sizes. (Note: 1 in. = 25.4 mm; 1 psi = 0.00689 MPa.)

FATIGUE TEST DATA (SMALL BIN)

Dynamic fatigue tests were conducted to obtain load versus displacement data for different reinforcement (tire) configurations of pavement sections. Fatigue tests were conducted on base case without the presence of tires and also with the presence of a regular automobile tire (AT) referred to as TDGC. Based on the deflection values (displacement), after 1 million cycles, mechanical concrete (presence of tires) contributed to 67% reduction in the displacement values. Displacement in nonreinforced pavement configuration was 207% higher (3.35 mm versus 1.1 mm) than the one with mechanical concrete having a regular recycled auto tire (Table 2).

Wheel loads cause shear punching in weaker subgrades and confinement systems such as mechanical concrete provides considerable lateral resistance in addition to reduced local settlements (Kief, 2008). Figure 9 shows displacement (settlement) values with up to 1 million load cycles for pavement sections with and without the presence of mechanical concrete. Most of the settlement occurs within the first 10,000 to 100,000 cycles for each test configuration. Trend lines were plotted after 100,000 cycles of loading to observe the rate of increase in displacement by ignoring initial settlement. Trends reveal a significant decrease in displacement rate for both 4,448 N – 17,793 N and 2,224 N – 4,448 N load ranges in TDGC reinforced aggregate base sections.

For fatigue loading with a lower load range of 2,224 – 4,448 N, the settlements were about 2.5 times higher in pavement sections without mechanical concrete (0.000001440x versus 0.000000592x). For fatigue loading with a higher load range of 4,448 N – 17,793 N, the

TABLE 2 Displacement at 1,000,000 Cycles for Specimens Exposed to the Lower and Higher Load Range

Higher Load Range (N)	Plate Diameter (mm)	Displacement at 1,000,000 Cycles (mm)		Percent Difference in Displacement for Base to TDGC Configuration
		TDGC	Base	
2,224 – 4,448	305	1.1	3.35	67
4,448 – 17,793	305	9.0	19.0	54.0

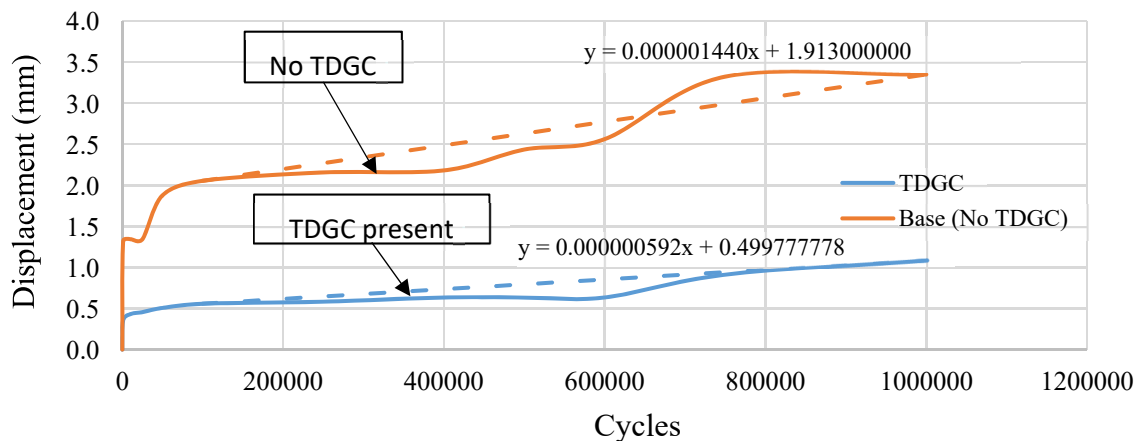


FIGURE 9 Displacements for pavement section with and without mechanical concrete exposed to the lower load range (2,224 N – 4,448 N). (Note: 1 in. = 25.4 mm.)

pavement section with mechanical concrete provided a 54% increase in performance during the first 100,000 cycles. However, this value would have been much higher as can be observed from Figure 10, but the test was stopped after reaching a preset limit of 19.0-mm settlement for the test case with soil only (displacement in base condition).

The TDGC was tested again with a fatigue load range of 4,448 – 17,793 N at 2 Hz for 3 million cycles to further explore the TDGC at increased cycle ranges. The 305-mm load plate was utilized for testing. The test data is plotted in Figure 11.

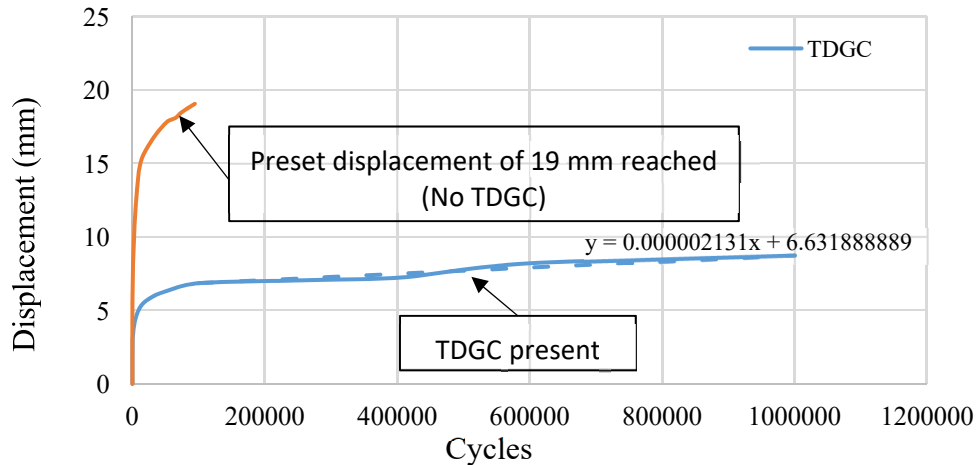


FIGURE 10 Displacements for pavement sections with and without mechanical concrete subjected to the higher load range (4,448 – 17,793 N). (Note: 1 in. = 25.4 mm; AT-mechanical concrete with recycled automotive tire.)

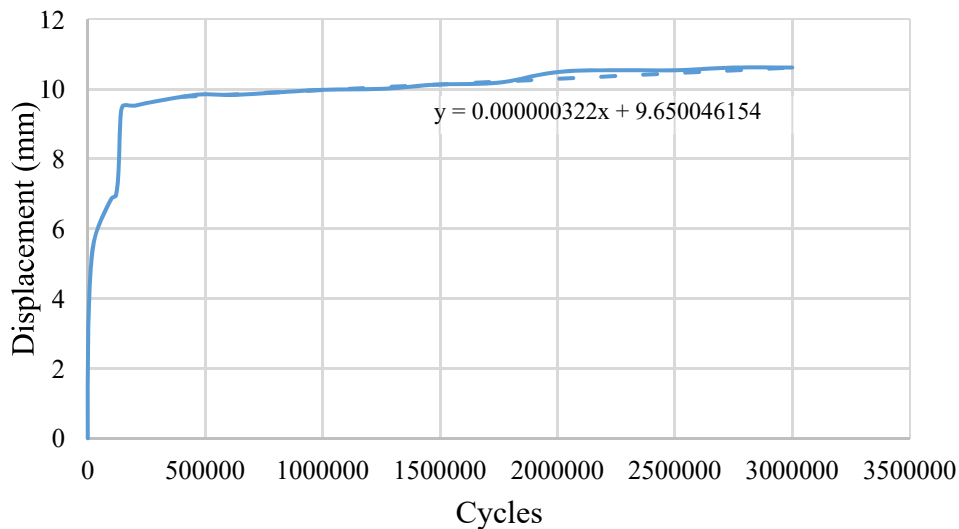


FIGURE 11 Displacement for pavement sections with mechanical concrete exposed to the higher load range (4,448 – 17,793 N) for 3 million cycles.

Following 3 million fatigue cycles, static tests utilizing the fatigue testing setup were conducted at a displacement limit of 3.81 mm (0.15 in.) to observe the post-fatigue behavior of the aggregate within the TDGC. The data showed that the higher load was needed to reach the displacement limit of 3.81 mm (0.15 in.). TDGCs facilitate additional compaction and displace more efficiently over time. The TDGC obtained an increase base–subbase modulus (combined) of 144 MPa/m after exposure to the higher fatigue loads for 3 million cycles than compared to the initial static testing value of 72.5 MPa/m (Figure 12).

CONCLUSIONS

Pavement strengthening using recycled TDGCs also referred to as mechanical concrete can significantly enhance the pavement life and reduce deterioration problems for low volume roads. In addition to strength enhancement, use of mechanical concrete will help mitigate some of the pavement rutting and pumping issues based on their ability to provide confinement effects along with the ability to offer water drainage. Tires with steel reinforcement provided better subgrade moduli improvements as compared to those without the presence of internal steel reinforcement. Use of recycled tire derived cylinders (mechanical concrete) is a green solution and helps address the landfill issues and emissions from burning of tires. Further research is being conducted on the field application and performance evaluation of mechanical concrete for flexible and rigid pavements.

ACKNOWLEDGMENTS

The funding provided by the West Virginia Department of Highways for this research is sincerely acknowledged. Authors are also grateful to Sam Bonasso for the material support and valuable suggestions during this research. The laboratory help provided by Piyush Soti and Krishna Tulasi Gadde are appreciated. We also thank laboratory technician Jerry Nestor at the West Virginia University Structures Laboratory for his help during testing.

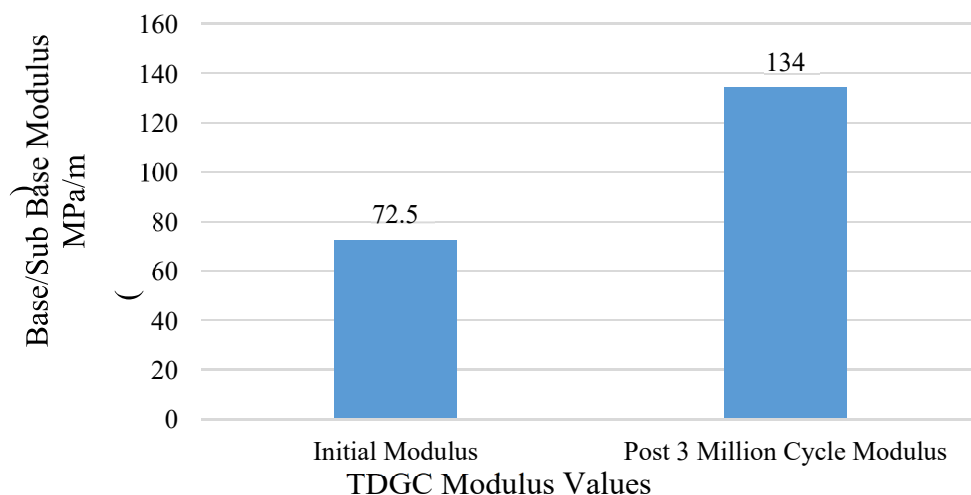


FIGURE 12 Post 3 million fatigue cycle static test results.

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Drainage and Stream Crossings

Superficial Drainage Control

Facts That Affected the Construction of Loboguerrero Divided Highway Project

SARA JIMENA MOSQUERA GONZÁLEZ

MARÍA CAMILA OROZCO MOSQUERA

DANIELA TRIGUEROS SABOGAL

DIEGO DARÍO PÉREZ RUÍZ

MARÍA FERNANDA SERRANO GUZMÁN

Pontificia Universidad Javeriana

The development of a country is associated with multiple factors that affects its economic growth. Among them, the availability and quality of transportation and road infrastructure are key factors to increase the productivity, reduce production costs and improve the standards of living. Particularly in Colombia, large investments in road construction projects have been made to improve connectivity between the regions. One of these projects is the Loboguerrero Road that has been designed to improve road safety while reducing the travel time. Going over a mountainous region the construction of this project includes large excavations on earth and rock, building embankments, and compacting in-place materials, among other activities. The disturbed rock and soil of slopes generated by cutting the natural ground surface and the conformation of embankments are very sensitive to weathering and erosion. Engineering properties of soil and rock mass may degrade and change during and right after construction due to weathering and erosion and the stability of new slopes can be reduced increasing the risk of failure if proper actions aren't considered.

Multiple construction problems have occurred on the Loboguerrero Road, landslides being the most important due to the risk of life and its direct effect on construction delays and the need of closing the road, sometimes with no possibility of detour. This study indicates that the main cause of slope failure is the lack of adequate drainage management. In addition, it presents solutions that could be used in projects with similar characteristics. A review of landslide records included on the reports of the construction project as well as local newspapers were used to identify the sectors considered critical due to landslides occurrence and magnitude. The information suggests that some landslides could have been prevented by proper surface drainage maintenance, significantly reducing the delays in the project execution and the increase in transportation costs generated by the numerous road restrictions and closures. It is important to note that, even on engineered slopes, inadequate surface drainage and maintenance could exacerbate slope stability problems leading to sizeable landslides.

INTRODUCTION

To lower the cost of economic activity and improve economic performance, the Colombian government is urged to increase the investment in public works construction such as highways, hospitals, schools, housing, sewer systems, waste water treatment systems, water supply systems, among many others (1, 2). Transportation infrastructure are key to ensuring the development of

economic and social activities and the cohesion of populations (3), however, the design and construction of these projects face difficult challenges. One of these challenges is the transformation of the ground surface and the alteration of soil and rocks masses that will be exposed to weathering and erosion causing reduction in slope stability. Landslides occur when gravitational forces exceed the strength of the soil that constitute the slope. Gravitational forces can be increased by increasing the water content of soil or changes in groundwater, usually due to rainfall or snowmelt (4). During heavy rainfall, inadequate drainage managements in slope areas allow water infiltration, increasing the risk of landslide (5).

The approaches for sustainable drainage emerged in North America and Europe to mimic the hydrologic cycle as closely as possible (6). One of the main causes of costly pavement maintenance is the presence of moisture within a roadway system. This problem is usually aggravated by the poor or inadequate drainage management. The advent of geosynthetics and its use in a wide variety of civil engineering applications have proven to be a versatile and cost-effective solution. Their use has expanded rapidly leading to new advances in geosynthetic manufacturing, including the development of geotextiles with enhanced lateral drainage (ELD) (7). The drainage capabilities of ELD makes this geomaterial particularly promising for slope stability control applications (Figure 1) as it is engineered to prevent increase in pore water pressure, among them, drainage galleries, horizontal or vertical drains, and drainage trenches, many of them applied in Colombia, reducing the road infrastructure maintenance cost.

Many are the factors involved in the analysis and design of natural and engineered slopes. Some of them are gravitational forces, seepage of water, pore water pressure, erosion of the

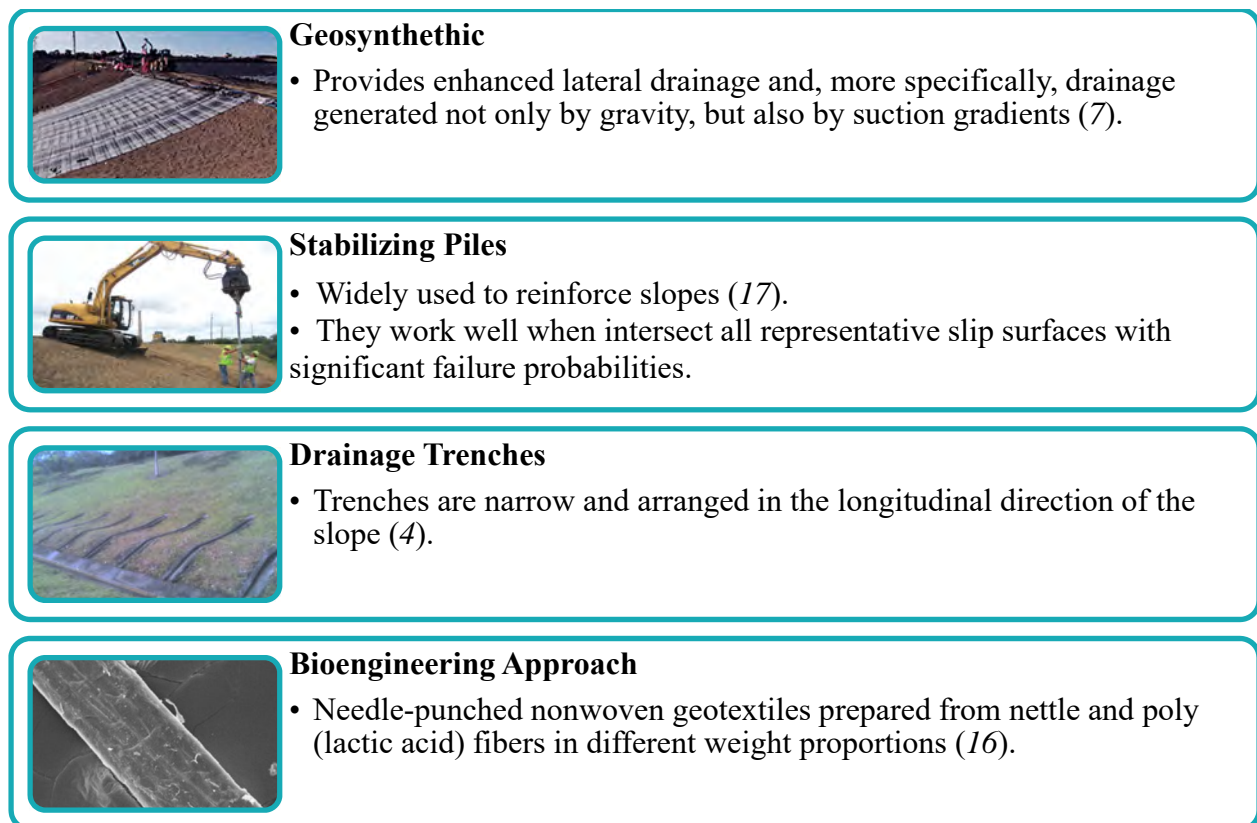


FIGURE 1 Some slope control alternatives.

surface, earthquake, drainage components, current and future tributary areas, land cover, and more (8). Slopes in the roadway systems are exposed to weathering and erosion generated by climate conditions. The main goal of this work is to reveal the effect of the absence of maintenance of slope control, particularly in the drainage system of Loboguerrero Road, and the negative impact for landsliding; and also showing some possible technical mistakes that increased the cost of this megaproject.

PROBLEM STATEMENT

The Loboguerrero Road, connecting the Buenaventura seaport with Buga city in Colombia (Figure 2) has been affected by avalanches, landslides, and traffic congestion, causing excessive freight prices generated by poor road infrastructure, low safety standards, and a high number of accidents. A new construction project designed to improve the road safety and reducing the travel time is involves building a divided highway between the municipalities of Buga and Buenaventura (Figure 3) and Mulaló–Loboguerrero (Figure 4). Large structures such as bridges, viaducts, and tunnels are being built as part of the road system financed by the National Roads Institute (INVIAS), and are expected to reduce the freight price and increase the revenue for more than 70% of the country's export and import trading that moves through the corridor, in addition to prioritizing integration of the region and updating the country's road infrastructure.



FIGURE 2 General location of the project area in Colombia (18).



FIGURE 3 Divided highway project Buga–Buenaventura (9).



FIGURE 4 Divided highway project Mulaló–Loboguerrero (9).

Unfortunately, the Loboguerrero–Buga roadway is an unstable corridor in which numerous landslides have occurred recently, allowing to identify at least 10 critical points (10). At these points there are clay slopes exposed to excessive water infiltration during high rainfall events, probably due to the absence of adequate surface drainage systems. This condition makes slopes susceptible to landslides or soil mass movement, especially on cut slopes (11). The cuts of the slopes reach 77 m (252.60 ft) and the frequency of landslides reveals the presence of unstable terrains that requires adequate design and construction to reduce the risk of slope failure and avoiding disasters with human and material losses (12).

FINDINGS

The construction of Loboguerrero project was divided into four sections: Citronela–Altos de Zaragoza, Altos de Zaragoza–Triana, Triana–Cisneros, and Cisneros–Loboguerrero, where slopes have been engineered to provide stability. The most important cases of slope stability include the following locations (13):

- Tres Chorros Slope. In the sector of Alto de Zaragoza–Triana–Cisneros, analyses and designs were made for the slope located between the abscissa K6 + 215 and K6-505 in Section 3 (Triana–Cisneros). This slope was engineered to provide second traffic corridor to fit a two-lane, one-way roads. The slope has maximum cuts of about 30 m (98.43 ft).
- Paraguas Slope. Also located in Section 3 (Triana–Cisneros) between K6 + 800 to K7 + 000, it has an approximate length of 540 m (1771.65 ft) and maximum cuts of up to 45 m (147.64 ft).
- Cementerio Slope. It is located between K6 + 800 to K7 + 000 in Section 3 (between Triana and Cisneros), the slope has maximum cuts of 22 m (72.17 ft) and a length of 200 m (656.17 ft).

Other slopes that have been stabilized are those located at the entrance and exit of the road tunnels which suffers delays due to the sliding of the slopes affected during construction.

Several slopes have been built without surface water drainage, particularly, for the Alto de Zaragoza–Citronela sector, which is characterized by the presence of silt and clay residual soils with low bearing capacity, deposited on siltstone or mudstone. In this zone it is recommended to build retaining structures on pile foundations (14). On the other hand, deforestation and the installation of water supply pipe lines on soft soils have accelerated the soil mass displacement phenomena causing massive landslides destroying the road near to Bendiciones town and surrounding areas; some similar effect is having the mining activity in Zaragoza area, where underground excavations is triggering the of collapse to the road (14).

Visual inspection of the road allowed to corroborate that in a large part of the road corridor, the main factor that generates the instability in hillsides and slopes is lack of superficial drainage management. This problem has been neglected and although concrete structures such as gravity walls, cantilever walls, or gabions have been constructed to stabilize the slopes, no hydraulic structures have been provided for the management of surface and subsurface water, promoting the infiltration of water in the backfill material with the consequent increase in pore water pressure and the risk of foundation scouring. In addition, many engineered slopes were built, but no revegetation was carried out and no drainage system was provided making natural development of vegetation more difficult.

DISCUSSION

Infrastructure is the foundation for economic growth and it is a measure of global competitiveness. Poor transportation systems reduce development opportunities and industrial competitiveness by raising the cost of freight and making services commercially unsustainable (3). Concerning transportation system, Colombia is behind several developing countries regarding density, specifications, and maintenance of roadway systems, causing lost revenues and affecting people's quality of life. Construction of new roads and the maintenance, improvement, and rehabilitation of existing roads is a key factor for the development of a region (15) and allows connection with urban centers as a strategic approach to influence regional development and diminish the social gap generated by economic inequality by giving access to education, medical assistance, and fair trade opportunities at all levels. Therefore, road safety and comfort are now variables to be included in the road serviceability definition, beyond traffic conditions and travel time. Slope stabilization is a key factor to guarantee road serviceability. There are

different methods for stabilization of slopes such as concrete retaining walls, soil nailing, gabions, mechanically stabilized earth walls, sheet pile walls, revegetation, etc. (16). Because of rain infiltration or rising of water table, slopes and back fill materials on retaining walls may become saturated and thus increasing gravitational forces and the earth pressure exerted on the retaining structure, it is necessary to include drainage management in the desing and stabilization of slopes. Among the lessons learned during the construction of loboguerrero Road is the need of including drainage management and erosion control as part of all natural and engineered slopes. Geotextiles with ELD, for example, allow drainage even under saturated conditions (7) and could be useful as a solution to be applied for Colombia's roads. Other alternative could be vertical drainage that have good performance when intersect all representative slip surfaces with significant failure probabilities (17). Another solution could be nettle and poly (lactic acid) fibers that shown remarkable improvement in fertility of soil (16).

CONCLUSION

The connectivity of rural and urban centers is a key stimulus for regional development and economic growth that can help to achieve social equality. Therefore, design and building of roads combines design and construction into one coordinated task that should be accomplished with the quality and safe conditions required by governamental agencies and users, speeding up project delivery and using the resuorces efficiently through adequate engineering designs, innovation and right construction. In the case of Colombia, road infrastructure is fundamental for the economic and social growth. In this work, some problems affecting construction process and the delivery time of the Logoguerrero Road have been described, showing that it is necessary to improve the road designs in future projects and drainage management must be one of the main concerns especially in areas with topographic and weather condicions that make them prone to landslides.

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DRAINAGE AND STREAM CROSSINGS

Understanding Ditch Maintenance Decisions of Local Highway Agencies for Improved Water Resources Across New York State

DAVID P. ORR
REBECCA SCHNEIDER
ANTHONY JOHNSON
Cornell University

Networks of roadside ditches crisscross the landscape, and have played a significant but previously unrecognized role in flooding and water pollution. This study surveyed town and county highway professionals across New York State (NYS) to determine their ditch management practices. There was a 41% response rate from the 999 highway staff surveyed, representing 54 of the 57 counties statewide. Of the responses, 36.8% of the agencies reported using full scraping or reshaping without reseeding as their primary method of ditch management and half scraped their ditches on average once every 1 to 4 years. It is estimated that one-third to one-half of the roadside ditches across upstate NYS are therefore in fair to poor condition. This translates to thousands of miles of exposed substrate vulnerable to storms, acting as a source of sediment and pollution. Limited resources including time, labor, equipment, and money were the primary reasons given for the practices used. Additional challenges identified included interactions with landowners over rights-of-way, farm–field drainage, and increasing frequency of downpours. A comprehensive, statewide program will be necessary to actualize ditch improvement. It will require a complete toolbox of strategies, from financial support and training to regulatory mandates and penalties, and needs to include a ditch inventory system. Incentives in the form of grants and shared services should be offered by state agencies working collaboratively with local governments. Valuing highway department managers as water stewards and supporting the improved management of roadside ditches can provide an important new mechanism for protecting NYS’s water resources.

To view this paper in its entirety, please visit:
https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

Retrofitting the Rural Roadside Ditch Network to Treat Nitrogen from Agricultural Runoff Using Woodchip Bioreactors

ERIC H. CHASE

Penn State University

REBECCA L. SCHNEIDER

NATHAN J. BAKER

STEVEN J. DUNN

Cornell University

INTRODUCTION

Nitrogen movement across the landscape has been indicated as a key contributor to the growth of the hypoxic zone in marine estuaries and is a concern on the federal level for watersheds like the Chesapeake Bay. Rural roadside ditches play a very significant role in nitrogen movement because they efficiently transport agricultural runoff to streams and rivers. Multiple well-tested best management practices (BMPs) (e.g., bio swales, woodchip bioreactor basins) for reducing nutrients already exist, but these have been primarily tested and used in large agricultural ditches or in situations where there is a single point of discharge such as a tile drain. In many agricultural settings, the topography and farm sizes do not lend themselves to treatment of agricultural runoff in large on-field ditches or as point sources. To address these issues, this study evaluated the potential of installing scaled-down woodchip bioreactors into roadside ditches for filtering dissolved nitrogen from agricultural activities.

Although many different iterations and permutations of the woodchip bioreactor technology exist, all rely on heterotrophic denitrification to convert nitrate to nitrogen gas. By supplying a carbon source in a bioreactor, the nitrogen is effectively removed from the hydrologic system. This technology has been in use for over two decades with a considerable amount of research having been done on the topic over the past 10 years. The rate of removal varies tremendously depending on the influent concentrations, the size of the reactor, the contributing area, the type of land serviced, and the carbon source used. Additionally, contact time with the bioreactor media is an important variable in removal efficiencies, especially at low dissolved oxygen levels. A review found that in agricultural applications, large subsurface bioreactors saw removal rates of between 12% and 76% of the total load (1). The National Resource Conservation Service (NRCS) in Iowa has even issued a Conservation Practice Standard for the design of denitrifying bioreactors. More recently, testing of woodchip bioreactors in roadside ditches by Rebecca Schneider (co-author) showed promise with a nitrogen removal efficiency of 40% during the growing season.

This study located in Bradford County, Pennsylvania, was undertaken by Pennsylvania State University Center for Dirt and Gravel Road Studies (the Center) and Cornell University to further study the potential for utilizing the rural roadside ditch networks as the basis for a low-cost agricultural lands filtration system. This project set out to monitor the treatment

effectiveness and efficiency for removal of dissolved nitrogen contaminants using existing road ditches retrofitted with woodchip bioreactor socks. This study seeks to evaluate the effectiveness of the in-ditch woodchip bioreactor to remove nitrogen from agricultural runoff, and determine temporal changes in effectiveness and the environmental conditions such as temperature and flows that influence it. The results will be used to determine the total nitrogen removed and cost per pound of nitrogen reduced through the use of in-ditch woodchip bioreactors to support improvement of conservation practice standards.

METHODOLOGY

Prior to the installation of the woodchip bioreactors, roadwork—consisting of additional cross pipe installations, farm field access upgrades, and road surfacing with the Center’s Driving Surface Aggregate (DSA)—was conducted to reduce sediment from the road entering the ditches (2). Additionally, a local spring was disconnected from the road ditch network to reduce overall flow volume entering the bioreactors. This preparatory work was performed in order to increase the overall efficiency and lifespan of the woodchip bioreactors. Monitoring of the road drainage and surface improvements included visual inspection of the practices to ensure that all drainage features were functioning properly. Additionally, a more rigorous surface condition index including measurements of wash boarding, raveling, surface rutting, and pothole depths was conducted similar to Woll et al. (3).

In 2016 a team from Cornell University led by Schneider began testing designs of the in-ditch woodchip bioreactor “sock” to reduce nitrogen entering streams (Figure 1). In May of 2018, using a similar design to earlier work done by Schneider, two bioreactor socks were constructed in sequence for this study. A single sheet of ¼-in. polyester debris netting was laid in the ditch bottom, extending totally across the ditch, filled with wood chips and the edges were rolled and locked with cable ties. The bioreactor media consists of coarse hardwood woodchips (~2 to 3 cm in length), aged in the open air for several months. Denitrifying bacteria are very



FIGURE 1 Woodchip bioreactor sock installed in ditch with ISCO sampler housing and rain gage in the background.

widespread and no inoculation of bioreactor material was necessary prior to installation. The bioreactors are held in place by six rebars hammered through the sock and into the substrate. The completed bioreactor socks each measured approximately 5 m in length by 30 cm in depth by 1.5 m in width. The system can be scaled up as needed depending on the nitrogen loads and field hydrology by simply adding more woodchip bioreactor reactor socks in line with each other.

After the installation of the bioreactors, two ISCO automated water samplers were installed to collect samples above, between and below the two woodchip bioreactors. In addition, grab samples were collected and both the ISCO and grab samples were tested for pH, electrical conductivity, nitrate + nitrite, and total phosphorus. Ditch flow was measured at the inlet of the bioreactors to create rating curves and water depth was continuously logged using capacitance style data loggers to create a continuous flow record.

FINDINGS

Sampling during 2018 consisted of collecting grab samples on 11 different dates and the two ISCO automated samplers captured seven complete or partial storm events. Overall, 2018 has been a difficult monitoring season due to record breaking rainfall across Pennsylvania (> 100 cm in 5 months) which led to extended periods of the bioreactors being overtopped. Over the entire monitoring season removal efficiencies varied from 0% to 100% and were highly dependent on ditch flow. Figure 2 shows that the bioreactors reactors performed well in the spring and early summer when ditch flows were not overtopping the bioreactors; however, in the later summer and fall high flows severely impacted removal efficiencies.

Visual monitoring of the road surface condition and other road drainage improvements indicate that limited suspended sediment is reaching the bioreactors. A quantitative assessment

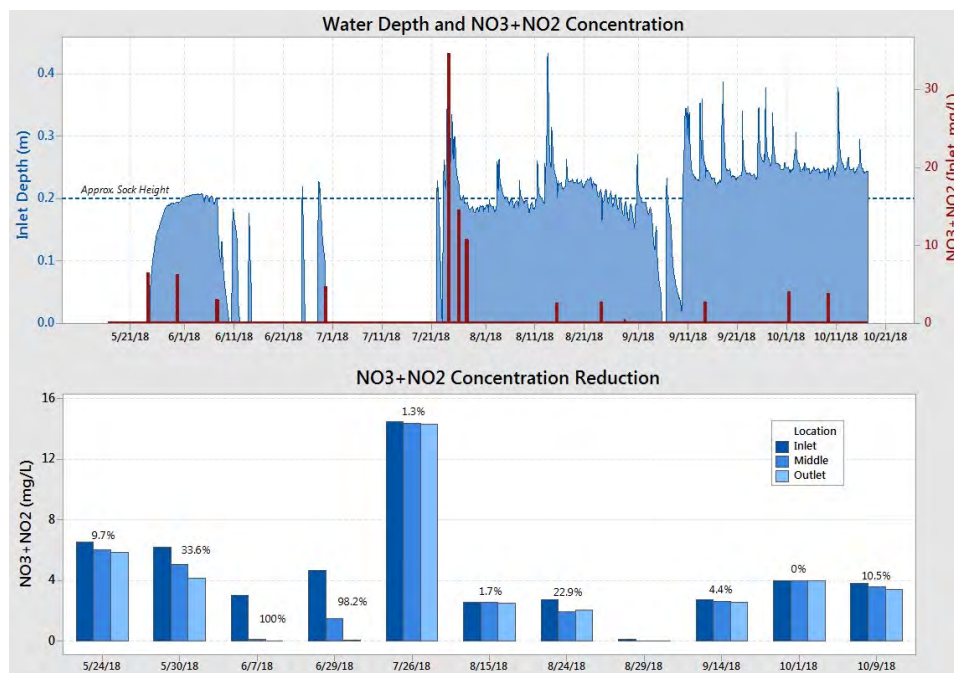


FIGURE 2 Nitrogen removal efficiencies as water passes through the bioreactors.

of the new DSA surface was completed on October 26, 2018. The results indicate that despite significant summer rains and heavy agricultural traffic, the road surface was in good condition (Table 1).

The results to date indicate in-ditch woodchip bioreactors are a viable option to treat agricultural field runoff for nitrogen. Overall, the first year results confirm earlier findings that performance of the bioreactors is enhanced at lower ditch flows (more contact time). This is an ongoing project with additional sampling planned for spring and summer 2019.

CONCLUSIONS

The data collected to date demonstrate that the use of the existing road ditch infrastructure that exists across Pennsylvania and the entire country can be used more extensively as a watershed-wide nutrient removal system. Additional data will be collected in 2019 to further quantify the efficiency of nitrogen removal and temporal changes in effectiveness of nitrogen removal and the environmental conditions influencing those changes. Specifically, the influence of temperature on nitrogen removal efficiency will be a focus during the early and late growing seasons in 2019. The larger dataset will also be used to quantify a total nitrogen reduction and cost per pound of nitrogen removed using in-ditch woodchip bioreactors.

This information will be compiled to determine the feasibility and usefulness of in-ditch woodchip bioreactors for widespread adoption and possible enhancement of an accepted NRCS conservation practice (Denitrifying Bioreactor Standard 605). There is also the potential to improve performance of an accepted NRCS conservation practice (Access Road Standard 560) by managing the access road ditch network as a system to reduce nutrients through woodchip bioreactors. Additionally, the project team hopes to demonstrate the potential for roadside ditches to be used as a watershed-wide nutrient reduction system incorporating multiple nutrient removal BMPs.

Extending the finding of this study, the team envisions the project going beyond just nitrogen reduction and leading to the utilization of the ditch networks as the basis for a low-cost, watershed-wide filtration system to help reduce pollution in our critical water resources. The bioreactors as installed are inexpensive (<\$500 each), need little maintenance other than seasonal inspection to ensure water is not bypassing the bioreactor and that they are not choked with sediment, and should last 5 to 10 years depending on environmental conditions. Outside of the agricultural community, this technology has the potential to economically and environmentally

TABLE 1 Road Surface Condition Evaluation. All Ratings on a Scale of 0 to 10, with 10 Being the Best Condition

Section	Wash Boarding		Raveling		Rutting		Potholes		Overall Rating
	Mean Depth (mm)	Rating	Mean Depth (mm)	Rating	Mean Depth (mm)	Rating	Mean Depth (mm)	Rating	
1	<5	9	7	8	0	10	0	10	9.25
2	<5	9	5	8	2	9	0	10	9
3	<5	9	2	9	7	8	0	10	9
4	<5	9	5	8	2	9	0	10	9

benefit municipalities that need to reduce pollution under their Municipal Separate Storm Sewer Systems (MS4). The potential application of denitrifying storm water in the roadside ditch network should be of interest to not only agriculture, but both road and wastewater managers as well.

ACKNOWLEDGMENTS

The authors thank the U.S. Department of Agriculture's Conservation Innovation Grant Program, the Pennsylvania State Conservation Commission, and the Bradford County Conservation District for supporting the project. Additionally we want to thank Canton Township for allowing us use of their roads for this study.

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DRAINAGE AND STREAM CROSSINGS

Optimization of Low-Volume Road Design in Flooding and Debris Control Activities in Selected Caribbean Areas Including Dominica *Consideration of Local Climate and Development Changes*

JACOB GREENSTEIN
USAID–Contractor

INTRODUCTION

Dominica’s coastal areas have experienced severe flooding during recent heavy rainfall and tropical storms events due to the rapid movement of water from the upland areas to its coastal plains and lack of adequate floodwater storage areas. Fundamentally, as part of an integrated stormwater management approach, the related main engineering issues and interrelated challenge includes rapid surface water flow associated with severe rock and mud debris that have produced significant damages to local roads, facilities, and endangered offshore protected fauna and flora.

Cost-effective and affordable engineering solutions were implemented in Dominica and other Caribbean countries to mitigate negative environmental impacts of low-volume road (LVR) investments, improve operation and maintenance activities, and minimize coastal flooding. Solutions included improving stormwater flow conveyance systems and incorporating flow retention and debris–rock trap facilities. Near-shore water quality preservation activities included controlling surface water flow through specific retention measures and source treatment in terms of modification and improvements of land use practices.

METHODOLOGY

Sediment Impacts and Environmental Mitigation Activities Related to Design of LVR Drainage Systems in Selected Caribbean Coastal Areas

During flood events, fine and coarse sediments erode and flow into the gullies and into overland water flow. Much of this sediment comes from the upstream watersheds, although human activity in the watersheds increase sediment loads above natural levels, primarily through erosion from agricultural land and from construction sites. When this sediment is discharged from land to the sea it affects near-shore habitat. Changes to coastal discharge and sediment loading are key consideration for the design of LVR drainage systems in coastal areas and must follow U.S. Endangered Species Act (ESA) legislation. Figure 1 and Figure 2 summarize the impact of ongoing climate changes and related heavy rainfall and flooding events on roads, structures, and local marine endangered species in the Caribbean region. The four photos given in Figure 1 present Dominica’s typical erosion and related landslide failures and flooding attributed mainly to weakened vegetation resulting from rising ambient temperature and prolonged drought periods.

The three photos given Figure 2 present three protected Caribbean marine species that are protected, among other protected marine fauna and flora, under the ESA. The related lessons

learned from implementing the LVR drainage design requirement into the fragile ecosystem of the Caribbean coastal areas of Dominica and Barbados, include

- The ongoing local climate changes characteristics in terms of rainfall and high-speed wind intensities, durations, and frequencies, and the related volume of erosion debris and the associated accumulation of contaminated sediments.
- Other sources of contaminants, including industrial sources and poorly developed and operated water treatment practices, to ensure the preservation of the health of all the marine species, as specified in the ESA and in the related local environmental preservation legislations and regulations.

Considering the ESA and best practices of LVR investments, financial institutions such as the Inter-American Development Bank and World and Caribbean Development Banks, and the Caribbean governments are required to



FIGURE 1 Impact of ongoing climate changes and related heavy rainfall and flooding events on roads, structures, and local endangered species. The four photos present Dominica's typical erosion, landslide failures, and flooding attributed to weakened vegetation as a result of rising ambient temperature and prolonged drought periods.

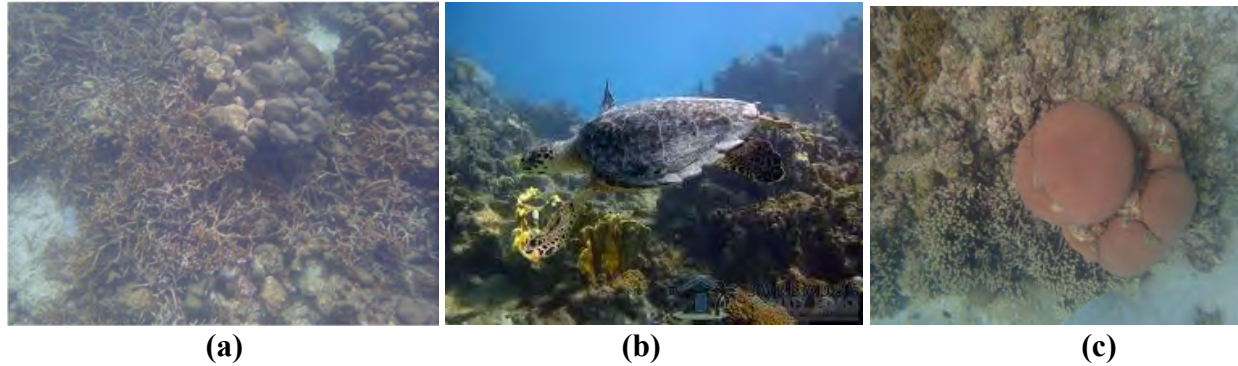


FIGURE 2 Three threatened species at a typical Caribbean marine reserve: (a) *Acropora cervicornis* (staghorn coral); (b) *Montastrea Annularis* (Boulder star coral); and (c) Massive starlet and finger corals.

- Improve the ecosystem integrity and functionality of the watersheds of the coastal areas in order to promote effective drainage in an environmentally sound manner.
- Carry out a benefit–cost analysis (BCA) considering the:
 - Investment–construction and annual routine and periodic maintenance costs of the drainage improvement works;
 - Mitigation and remediation works and related costs of the on-shore and offshore activities needed to avert the polluting of the marine reserve species; and
 - Drainage and stormwater management improvement.

Other related benefits are associated with the reduction of the flooding damages to roads, structures, and water supply systems. Figure 5 shows typical damages to roads and structure in Dominica. Also, a recent BCA, given in the section on Best Practices for Optimizing LVR Design and Operation and taken from Greenstein (1–3), indicates that drainage investments and stormwater management improvements, including affordable maintenance works and related environmental mitigation and remediation works needed to comply with the ESA, has resulted an adequate economic return of BCA of 1.1 and internal rate of return (IRR) of 15%.

Stormwater Management: Selected Cost-Effective Drainage Solutions for Coastal Low-Volume Roads

Figure 3, Figure 4, and Figure 5 present cost-effective and affordable LVR drainage and related sediment control solutions—such as debris trap elements—implemented successfully in Dominica’s Mero community to avert the coastal damages caused by tropical storms such as Erika in August 2015. Photos of these damaged areas are presented in Figure 5.

Figure 4 presents the actual drainage and debris control facilities implemented in Dominica to mitigate damages caused by tropical storms, such as Erika that produced the significant damages to unprotected coastal areas of Dominica, given in Figure 5.



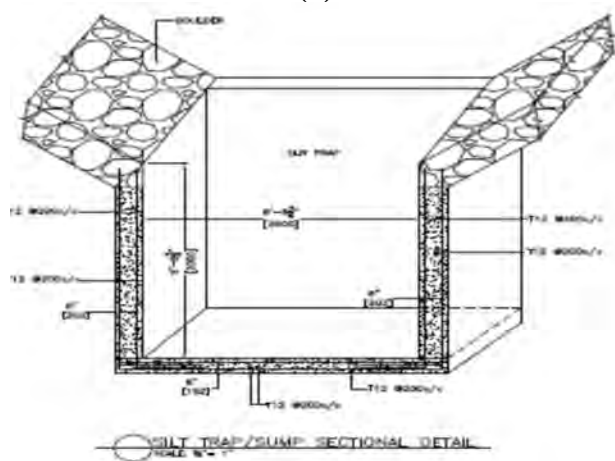
FIGURE 3 Cost-effective stormwater management tools, including debris control elements given in the two-upper-center photos and other stormwater management and sediment control and retention facilities used to avert environmental degradation of threatened marine species in the Caribbean coastal areas.



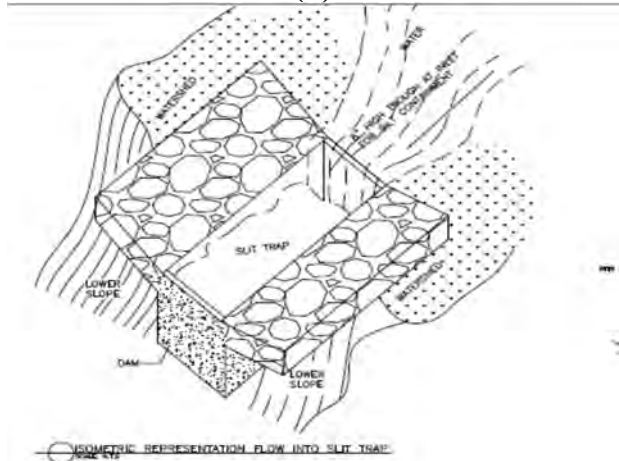
(a)



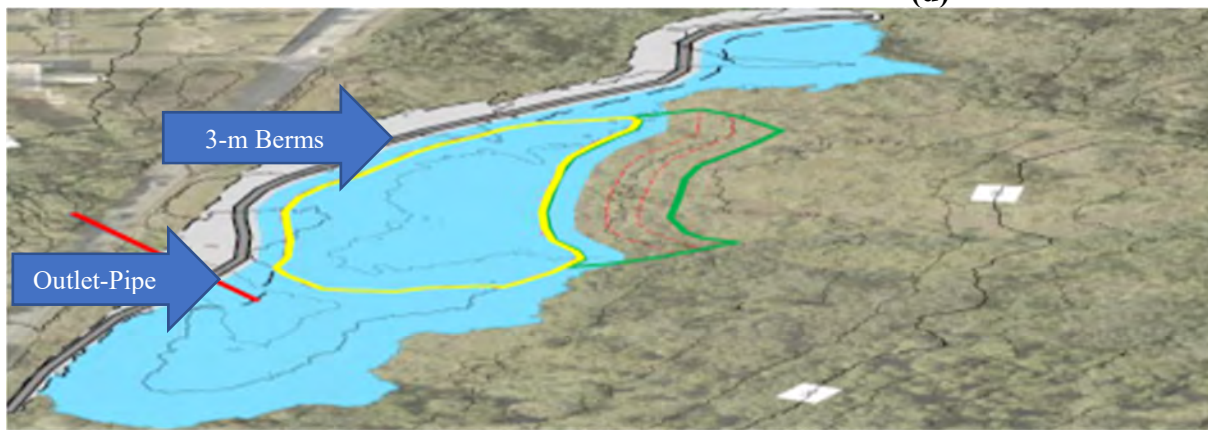
(b)



(c)



(d)



(e)

FIGURE 4 Implementation of the (a) through (d) debris traps and (e) the stormwater retention facility used to collect debris and other contaminated sediments at the Dominica’s Mero community and thus averting the coastal damages given in Figure 2 and caused by the tropical storm Erika in other coastal areas of Dominica in August 2015.



FIGURE 5 Representative damages caused by the tropical storm Erika in Dominica. The damages occurred in nonmitigated coastal areas. In comparison, mitigated areas with debris–silt traps and other stormwater retention facilities were not damaged by Erika.

Best Practices for Optimizing LVR Design and Operation in Terms of How Best to Manage Flooding and Debris Control Activities Administered by Local Community

In the coastal community of Mero in Dominica, the USAID engineering team supported the design and implementation of an effective and transparent gender approach to optimize the design and construction of cost-effective infrastructure for disaster risk reduction and climate change adaptation. Dominica has been facing higher ambient temperatures and prolonged draught periods that have decreased the ability of vegetation to maintain the stability of slopes on the island’s mountainous and hilly terrains. In consultation with male and female community leaders, a system to trap debris and avert clogging of the drainage system was upgraded to address rainfall characteristics and road safety issues. Community participation included an effective and transparent gender approach, in which local women leaders and engineers played key roles in planning, construction, and maintenance. During Tropical Storm Erika in August 2015, this infrastructure demonstrated its resilience in the face of heavy rains with Mero avoiding the losses of life and property experienced in other parts of Dominica, as described in the above Figure 5.

BCA of the Dominica drainage improvement works justified this investment in terms of producing a benefit–cost ratio of 1.1, IRR of 15% and a net present value of \$377,738, considering

- a. Coastal investments of \$3.0 million;
- b. Periodic–preventive maintenance works of \$0.6 million every 5 years; and

c. Routine watershed drainage and environmental maintenance, including revegetation at an affordable annual cost of \$142,000.

The BCA total annual benefits, including infrastructure, tourism and environmental benefits generated from the drainage improvement works was estimated at \$600,000 per year.

CONCLUSION

Optimization of LVR design, construction and operation–maintenance activities financed by international donors in the Caribbean region requires to (a) optimize the design of the flooding and debris control elements and especially the debris traps facilities given in Figure 1, considering ongoing local climate and development changes; (b) Address the ESA and the related local environmental preservation legislations and regulations; and (c) carry out a quantitative BCA needed to justify the LVR investment and maintenance costs in terms of producing an economic IRR of over 12.

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DRAINAGE AND STREAM CROSSINGS

Improving Road Stream Crossings for Storm Resiliency and Aquatic Organism Passage

Pennsylvania Case Study

STEVEN BLOSER
Pennsylvania State University

INTRODUCTION

Pennsylvania is home to more than 86,000 mi of streams and rivers, second in the United States only to Alaska (1). With over 120,000 mi of nonfederal public roads, 75,000 mi of which are owned by 2,500 local municipalities, Pennsylvania has an extensive network of stream culverts and bridges owned and maintained by a wide variety of entities (2). Unfortunately, a large percentage of these stream crossings, especially in rural areas, are inadequately sized for the streams they attempt to convey. Undersized crossings pose a host of problems for both the road and stream including channel erosion, road erosion, gravel aggradation, blockage of aquatic organism passage, road flooding, and even loss of structure.

Pennsylvania's Dirt, Gravel, and Low-Volume Road Maintenance Program (Program), administered through the Pennsylvania State Conservation Commission, provides \$28 million annually in funding to local municipalities to implement environmental improvements on unpaved and low-volume (<500 average daily traffic) paved roads. Starting with a funding increase in 2014, the Program has increased its focus on stream crossing replacements, now funding about 100 annually. This abstract summarizes the efforts of the Program from 2014 to 2018 to focus funding on road–stream crossing replacements that provide an environmental benefit, and to ensure new crossings are adequately sized and installed properly. The experiences and lessons learned in Pennsylvania can benefit other entities as concerns about both aquatic organism passage and flood resilience in a changing climate gain traction across the United States and beyond.

METHODOLOGY

Focusing on Undersized Structures

The Program has always tried to focus its limited funding on locations where the greatest environmental improvements can be achieved. Not wanting to become simply “supplemental bridge replacement funding,” the Program implemented policy to limit the amount and type of structures eligible for replacement and focus funding where environmental improvements could be achieved. The Program enacted policy that in order to be eligible for replacement, the existing structure (pipe, bridge, etc.) must be significantly undersized compared to the bankfull channel width, and show signs of erosion or aggradation due to the structure. The “bankfull” channel width is defined as the width of the active channel at a bankfull discharge, typically associated

with imminent access to the floodplain and a roughly 1.5-year flow recurrence interval (3). The existing structure opening width to bankfull channel width ratio to be eligible for replacement with Program funds was originally set at $\leq 50\%$, but has since been increased to $\leq 75\%$ to allow more structures to be eligible.

Defining New Structures

While smooth round pipes are discouraged, the Program does not specify the type of structure to be installed (pipe, arch pipe, bridge). The Program has implemented a policy requiring new structures to have a single opening that at least spans the bankfull channel width ($\geq 100\%$ new structure opening to bankfull channel width). The policy also requires consideration for aquatic organism passage and proper structure alignment where possible.

Installation Assistance and Education

While adequate structure sizing is likely the most important step in replacing stream crossings, there are a wide variety of other complex issues that impact the overall quality of the installation such as grade controls, establishing substrate through the crossing, creating a stable slope, and dealing with legacy sediment wedges and plunge pools caused by undersized structures. The Program has partnered with Trout Unlimited and the Penn State University Center for Dirt and Gravel Road Studies on an education and technical assistance initiative to improve the quality of stream crossing installations. These entities provide several field trainings each year, and spend significant amounts of time providing technical assistance on the design and implementation of specific projects around the state.

FINDINGS

Focusing on Undersized Structures

While the determination of bankfull channel width is somewhat subjective, it has provided the necessary quantification to be able to focus funding on undersized structures.

While the structure opening width to bankfull channel width ratio for replacement eligibility was initially set at $\leq 50\%$, the Program found that there were many structures in the 50% to 75% range that were significantly impacting streams, but fell outside the criteria for replacement. Approximately 3 years after initial policy implementation, this eligibility threshold was increased to $\leq 75\%$ to allow more structures to be replaced, while keeping the focus on undersized structures. While it has taken a large educational effort, the use of a structure opening width to bankfull channel width ratio limitation seems to be working well in focusing Program funds at sites that will provide environmental improvements.

Defining New Structures

By requiring existing structures to be undersized to the bankfull channel by $\leq 75\%$ to be eligible for replacement, and requiring new structures to be $\geq 100\%$ of the bankfull channel, the Program is installing structure that are, at a minimum, 1.33 times the width of the existing structure. While

this has caused some pushback from local municipalities and engineers, mostly in regards to added cost, it has generally been accepted. Since the Program is funding the crossing replacements, adequate sizing to meet policy is a requirement for a project to proceed. Road owners typically report a reduction in maintenance such as washouts and gravel deposition associated with the new larger structures. Realignment of new structures with the stream channel is encouraged, but is often not allowed due to permitting issues in many parts of the state.

Evaluation of Effectiveness

With 100 stream crossing replacements completed annually at the local level, assessing the effectiveness of policy, training, and project implementation is a large task. Trout Unlimited is conducting a comprehensive survey in the winter of 2018–2019 to evaluate 100 randomly chosen stream crossings across Pennsylvania. This survey will provide valuable feedback to direct both policy and educational discussions for future Program stream crossing projects (results will be incorporated into any presentations related to this submission). Anecdotal evidence to date suggests that more design and planning assistance is needed to ensure structures are placed at the proper slope and with proper grade control to maintain substrate within the structure. This is especially true in situations where significant channel aggradation and degradation have been caused by the existing old structure (Figure 1 and Figure 2).



FIGURE 1 Before: twin 7-ft “tanker car pipes” bankfull-width channel created a barrier in a reproducing brook trout stream, and because it often clogged, flooded the road and required maintenance. (Photo: Tioga County Conservation District.)



FIGURE 2 After: 32-ft geosynthetically in a 30-ft reinforced soil bridge used to span bankfull in a channel width. (Photo: Tioga County Conservation District.)

CONCLUSIONS

The policies and educational efforts implemented by the Program have gone a long way to focus funding on stream crossing replacements with environmental benefits and encourage the installation of structures that better serve both the road and the stream. The use of bankfull as a quantification threshold, while slightly subjective, has been a great benefit in both determining crossing eligibility for replacement, and sizing new structures.

Challenges and lessons for other entities:

- Acceptance and understanding of the policy and the need for larger structures at the local level can be challenging. On Program-funded projects, policy requirements must be met in order to receive funding, but similar policy efforts not tied to funding may be met with more resistance.
- Pennsylvania has a highly fractured road ownership system with over 2,500 municipalities owning public roads. This type of system requires a significant amount of education, outreach, and technical assistance to implement these changes to such a large and diverse set of road owners.
- While requiring new structures to be $\geq 100\%$ of the bankfull channel width is a great start, many entities have determined that a larger structure is needed to better ensure continued aquatic organism passage. For example, the U.S. Army Corp of Engineers and several New England states require structures to be size at $\geq 120\%$ the width of the bankfull channel (4, 5).
- A great deal of education is required in proper design and implementation of these crossings, including determining the appropriate slope, grade control, and substrate in the structure. This is especially true where undersized structures have caused significant gravel

deposition upstream and scour erosion downstream of the road crossing, requiring extensive stream rehabilitation.

- It is important to develop working relationships with pipe and bridge manufacturers and suppliers in order to facilitate choosing the structure best suited to each situation. Note that open fords may be an option for some entities, but are rarely permitted in Pennsylvania.
- Defining the lower boundary of “what is a stream,” is important in headwater areas to make sure small perennial and ephemeral channels are treated as streams and follow bankfull policy. If such channels are deemed “not a stream” by local entities, undersized smooth plastic pipes are often used.
- Since they are cheap and readily available, many entities prefer smooth plastic pipe for stream crossing installations of 6 ft or less. The use of these pipe should be discouraged since it is difficult to maintain substrate through these pipes.

While there have been some implementation issues occasionally resulting in less than desirable projects, the policy and educational efforts of the Pennsylvania Dirt, Gravel, and Low-Volume Road Maintenance Program have been largely successful. The Program will continue to evolve policies and educational efforts based on feedback from road owners, advisory workgroups, and studies as they are completed.

ACKNOWLEDGMENT

Made possible through the Pennsylvania Dirt, Gravel and Low-Volume Road Maintenance Program and the support of the Pennsylvania State Conservation Commission.

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Options for Aging and Deficient Low-Volume Road Bridges

GORDON REX KELLER

Genesee Geotechnical

INTRODUCTION

The condition of many low-volume road bridges in rural United States, as well as the world, are in poor condition yet are still being used. A recent U.S. Government Accountability Office report on bridges in the United States shows that of the 610,000 existing bridges, nearly 25% are deficient, with 10% categorized as structurally deficient and 14% categorized as functionally obsolete (1). A structurally deficient bridge means that there is significant deterioration of the bridge's major components. Thus, over 60,000 bridges are structurally deficient, obsolete, or in poor condition, and some are scour-critical. Many were built more than 50 years ago; many have not received needed maintenance and climate change, with its larger storm events, has contributed to the deterioration of the structures, making them only marginally sound or potentially dangerous today. Additionally, vehicles used in agriculture and alternative energy development today impose additional and heavier loads than those encountered in the past on many rural bridges.

Considering this situation, agencies have been looking at the many creative ways to either replace bridges, close bridges, or find alternative structures or routes. The main options include either 1) ways to replace bridges with relatively inexpensive new structures or 2) ways to find alternatives to the use of the conventional bridge. This includes ways to reuse old bridge parts, reducing bridge width, reduced load capacity, using temporary and alternative structures such as low-water crossings, or closing bridges where reasonable detours or alternative routes exist.

METHODOLOGY

Through networking, a number of federal, state, and local agencies, as well as universities and industry personnel, were queried to determine how they are dealing with their aging bridges. Agency road funds are typically limited. One new bridge can cost a significant part of the annual operating or road maintenance budget of the local agency, unit, or forest. Thus, agencies are in a bind regarding the need for bridge improvements or replacement and the lack of funds to accomplish this needed work. Yet continued use of structurally deficient bridges could risk public safety.

Agencies have been quite resourceful in dealing with their old bridges, though the backlog of needed replacements persists. Finding less-expensive ways to replace bridges, reusing parts of old bridges, using alternative structures or alternative routes, or reducing design loads have all been alternatives used. These alternatives are discussed below. Additionally, this information has been presented in Transportation Research Board-sponsored activities such as workshops, webinars, and published reports that help disseminate the concerns and information on solutions to this issue (2, 3).

FINDINGS

Relatively inexpensive new construction bridge replacement options include the following:

- Using accelerated bridge construction (ABC) and modular construction techniques with prefabricated units to reduce the time and cost of bridge construction;
- Building geosynthetic-reinforced soil (GRS) bridge abutments and placing the superstructure on those abutments, thus saving time and costs; and
- Building long-span “buried bridges” (large concrete or corrugated metal pipe structures) as an alternative to conventional bridges, with spans of over 15 m (50 ft).

Other alternatives, which are not as ideal as new bridge replacement, but can provide some reasonable level-of-service for traffic on the bridge, or an alternative to typical bridge structures, include:

- Using salvaged, suitable parts from old bridges to construct a new bridge, and strengthening existing components;
- Replacing a double-lane bridge with a single-lane bridge on low-traffic roads;
- Replacing the old structure with a relatively inexpensive temporary bridge, such as a Bailey bridge or a used railroad flat car;
- Reducing the allowable load capability of the existing bridge with a reduced weight limit;
- Replacing a bridge with a less-expensive low-water crossing if the route is noncritical and some traffic delays are acceptable; and
- Doing traffic studies, route planning, and examining alternative routes to make decisions as to the feasibility of closing certain bridges and rerouting the traffic.

Accelerated Bridge Construction

Methods to increase the speed and efficiency of bridge construction have been developed by various universities and agencies. The concept of ABC is to use modular components to simplify and speed up construction of a bridge and reduce costs. Much of this work has been pioneered by the Bridge Engineering Center at Iowa State University and the Iowa Department of Transportation (DOT) (4), as well as the FHWA (5). The use of ABC can significantly decrease traffic delays during construction.

Commonly used modular units include precast piers and columns, abutment units, and deck panels. Examples of pre-cast elements include the homemade backwall and pre-cast caps and pre-cast superstructure elements, most of which are constructed locally.

Geosynthetic Reinforced Soil Abutment Bridges

The use of GRS bridge abutments to support the superstructure of a bridge has been pioneered by agencies such as the (6) and the U.S. Forest Service (7). Several hundred of these structures have been built across America to-date, with most showing a significant savings of both time and cost of construction compared to traditional bridge design and construction. GRS structures can be constructed quickly with common equipment and by maintenance personnel. A principal

advantage of GRS bridge abutments is that the design can eliminate the infamous “bridge bump” caused by differential settlement between the abutment foundation and the approach fill.

The soil mass of the abutment system is constructed like a reinforced soil or mechanically stabilized earth-retaining structure, using reinforcing layers of a polymeric geosynthetic material such as geotextiles and geogrids or a welded wire. The bridge girders then rest upon this reinforced soil mass. Various “flexible” facing materials have been used, including timbers, concrete block (Figure 1), gabions, or rigid-facing material such as concrete can be used, depending on the design and site conditions.

Buried Bridges

Buried bridges involve use of large multi-plate corrugated metal pipe (CMP) or precast concrete culverts to support the roadway embankment and surface as a cost-effective alternative to bridges in many applications (3, 8). CMP culverts are commonly used to replace small low-volume road bridges and the cost of culverts are typically much less (Figure 2). Because of the



FIGURE 1 A GRS bridge abutment constructed in Pennsylvania using modular concrete blocks for the abutment facing.



FIGURE 2 A CMP-buried bridge with a 56-ft span built on a rural road (photo courtesy of Joel Hahm).

high loads on large structures, specific designs are required for the deep corrugated structure plate structures. Spans of over 15 m (50 ft) are common, and some have exceeded 25 m (80 ft).

Use of Reused and Reinforced Bridge Elements

Some county engineers have become quite resourceful at reusing existing bridge components to replace an existing structure (Personal Communication: Brian Keierleber, Buchanan County Engineer, Iowa). Work has involved reusing bridge girders on new pile foundations or abutments, using existing bridge foundations in good condition and replacing a deteriorated superstructure, or reinforcing an existing foundation before placing a new superstructure (9). Other measures have included splicing broken piles, reinforcing or encasing existing piers and abutments in concrete; using sheet piles to shore up failing abutment walls; adding new piles to augment existing piles; or using ultra high-performance concrete. Most of this work has been done by local agency crews, again to reduce costs.

Temporary Bridges

Temporary or modular bridges, which are often kept in service for years, include structures such as Bailey bridges, Big R, Acker, Hamilton EZ, and other modular bridges, and used railroad cars. Both 20.7- and 27-m (68- and 89-ft) long railroad flat cars are available and can be used as a low-cost alternative to replace deficient bridges (10). Railroad flatcars have a very high load capacity, typically exceeding highway loadings. However, some load rating, analysis, or reinforcement may be desired. Many temporary bridges were designed to originally accommodate emergency or temporary repairs, but have been left in service for many years.

Single-Lane Bridges and Reduced Load Limits

On low-volume roads, many bridges are found that are only a single-lane wide, with about a 5-m (16-ft) wide deck. An existing two-lane bridge can be replaced with a similar span, single-lane structure. Cost savings can be significant but traffic and safety concerns must be addressed. Adequate signs are needed to warn of a single-lane structure.

Also, marginally strong structures can be load rated and signed for a reduced load capacity. Load-limit signs are legally required to indicate some load capacity less than the standard legal load limit, such as HS20-44 or HL-93 loadings.

Alternative Low-Water Crossing Structures

In some instances, a replacement bridge may not be necessary. For low-traffic volumes, noncritical routes, and roads that can tolerate traffic delays, a simple ford or vented low-water crossing may be adequate. It will typically cost less than half or a third of the cost of a common bridge. Low-water crossings are ideal and cost-effective on noncritical rural routes with low traffic volume, particularly where stream flows are highly variable or the channel moves a large amount of debris (11). Their use is limited on many roads because of occasional traffic delays and concerns for traffic safety, but they are also found on many rural roads.

Bridge Closures and Use of Alternative Routes

In some circumstances, a replacement bridge may not be needed if an acceptable and cost-effective alternative route can be used. The University of Kansas (12) conducted an economic evaluation studying the acceptable distance a detour could be used in the state where traffic volume is low and many road systems are laid out in a systematic grid pattern. In that study, detours or alternative routes have been considered acceptable for up to several miles where the traffic volume is very low.

CONCLUSION

The condition of many low-volume road bridges in America, as well as worldwide, is poor. More than 60,000 rural bridges in America are old, structurally deficient, or functionally obsolete. Many have not received needed maintenance, and climate change, with its associated larger and more-intense storm events, has contributed to an increased risk of failure for many marginally sound structures. As a result, agencies have been looking at many creative ways to either replace bridges, close bridges, or find alternative structures or routes.

Where funds are not available, bridge alternatives or traffic management solutions have been adopted. Many of these alternatives are undesirable to some agencies, or may not be allowed by policy, but reality dictates that some old, unsafe bridges either have to be closed or some reasonable alternative adopted to keep a road open. Fortunately, options, described herein, do exist for the construction or replacement of conventional bridge structures.

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Proposed AASHTO-Ready Specifications for Load Rating and Redundancy Evaluation of Railroad Flatcars Used as Highway Bridges on Low-Volume Roads

ROBERT J. CONNOR
KADIR SENER
Purdue University

TERESA L. RATHBRUN
Michael Baker Jr., Inc.

INTRODUCTION

Railroad flatcars (RRFCs) are an attractive option to replace existing deteriorating bridge structures on low-volume roads. They are typically used as the bridge superstructure by placing two or more flatcars side-by-side to achieve the desired roadway width. Utilizing RRFCs as a bridge allows for rapid construction by normal highway maintenance personnel using readily available equipment compared to traditional practices (Provines et al., 2014a). These benefits make them an attractive solution for rural communities.

The unique superstructure of RRFCs creates a challenge when attempting to load rate these types of bridges. Unfortunately, there is no guidance in existing American Association of State Highway and Transportation Officials (AASHTO) specifications on load rating RRFC bridges, often resulting in overly conservative load postings. Utilizing the results of the previous work from other studies (Wipf et al., 1999; Wipf et al., 2007a; Wipf et al., 2007b; Provines et al., 2014b), proposed load rating guidelines were developed.

The research project resulted in the development of AASHTO-ready specifications for rating RRFCs. Further, a proposed methodology to evaluate the redundancy of RRFC, in event one of the main longitudinal load-carrying member fails was also developed.

METHODOLOGY AND FINDINGS

This document will summarize the results of a multi-year research program focused on:

1. The development of AASHTO-ready specifications for load rating RRFCs of various geometry and configuration;
2. Investigating the system redundancy of RRFCs bridges as they are commonly classified as fracture critical members (FCMs) since there are typically two RRFC used in a given cross section; and
3. Developing AASHTO-ready specifications to perform system analysis of a bridge built RRFCs to establish if the members are system redundancy members or FCMs.

Load Rating Provisions

The primary objective of this portion of the study was to develop load rating guidelines for RRFC bridges based on measured data collected during field instrumentation and controlled load testing on a representative sample of RRFC bridges. The guidelines were aimed at establishing the maximum positive live load stress due to bending when load rating RRFC bridges. The live load bending stress then can be used in conjunction with the load rating method presented in the *AASHTO Manual for Bridge Evaluation*. The load rating guidelines are intended to provide a simple, yet accurate alternative to load rating methods developed in previous research studies. Furthermore, the guidelines were specifically prepared to be similar to AASHTO specifications to provide engineers with a degree of familiarity when performing a load rating on RRFC bridges. To this end, specification ready language and commentary was developed as part of the study should there be interest to adopt the rating procedure.

Fracture Critical System Analysis

Although the research by Provines et al. (2014b) resulted in reasonable rating procedures, some uncertainty remained regarding the response under higher loads than could be easily and safely achieved in the field with test trucks and the effects of a fully composite concrete deck. In addition, the rating procedures developed by Provines et al. (2014b) did not directly include provisions for calculating ratings for shear. During the research described herein, laboratory testing of a RRFC bridge with two flatcars placed side-by-side allowed for experimental testing under higher loads, as well as increased amounts of instrumentation to better understand the behavior of the RRFCs. The two RRFCs that were acquired for testing are classified as “typical” RRFCs. A typical flatcar consists of one main box girder, two exterior channel girders on either side of the main girder, and three to four stringers between the main girder and each exterior girder. Following the experimental program, a detailed finite element (FE) modeling was developed and benchmarked using the experimental data. Once benchmarked, a comprehensive parametric study on the behavior of the RRFC bridge with a composite deck was performed by varying the spacing between flatcars, load position, and member relative stiffness.

Further, a full-scale bridge was constructed with two RRFCs placed sided by side in the laboratory with a fully composite concrete deck (Figure 1). While the laboratory research provided additional data to enhance the rating specifications, this phase of the work focused on the level of system redundancy in a RRFC bridge after failure of one of the two main box girders. Based on the lab testing and a large analytical parametric study (Figure 2), additional procedures were developed to estimate whether the remaining longitudinal members provide sufficient available capacity to carry traffic loads when one of the main box girders is assumed to have failed.

The analytical models for conducting the parametric studies were benchmarked to experimental tests that were conducted on a previous research project presented in the report by Washeleski et al. (2013). The experimental research evaluated the behavior of both intact and fractured flatcar bridges subjected to high loads to develop load rating guidelines. The experimental research focused on the load distribution and level of system redundancy in the tested RRFC bridge before and after failure of one or both main girders. This analytical studies included several parameters which were varied including the (i) spacing between flatcars, (ii) load position, and (iii) member relative stiffness. The results were used to improve previously



FIGURE 1 Photograph of laboratory test specimen.

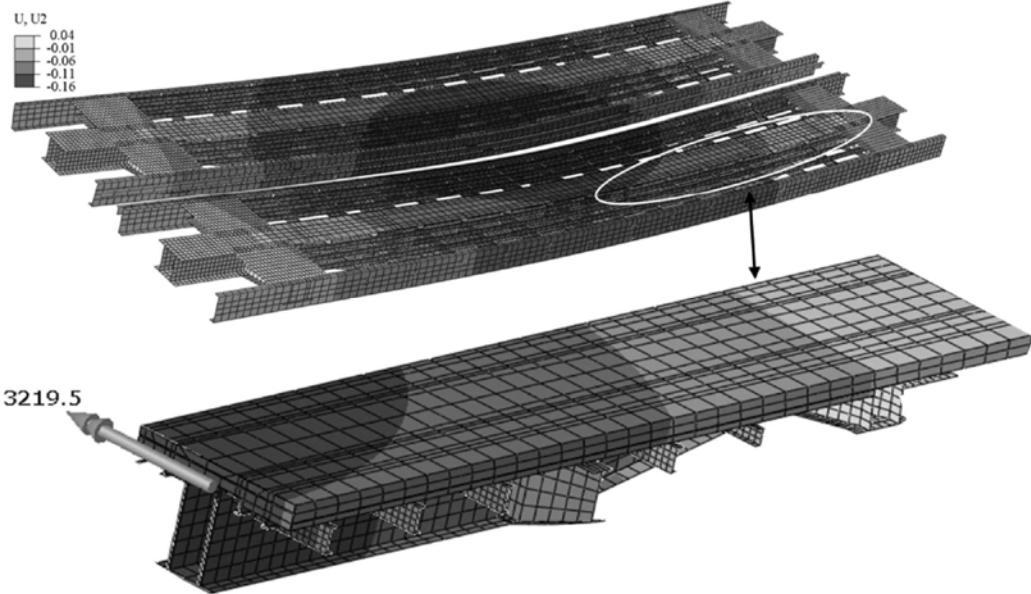


FIGURE 2 Illustration showing detailed FE analysis of RRFCs with composite concrete deck.

proposed load rating guidelines developed for RRFC bridges constructed with composite concrete decks.

CONCLUSIONS

The research demonstrated that RRFC bridges can be load rated using simple producers consistent with existing AASHTO methodologies for fatigue. Further, the system analysis showed that the addition of a composite concrete deck provides significant reserve strength in the even one of the main load carry members were to fail. A cost analysis suggests that a concrete deck is actually cheaper than may be expected when compared to the traditional oak timber type deck commonly used.

This paper will present the results of the multi-year research project and provide an overview of the proposed AASHTO guide specifications and the simplified procedures to evaluate the redundancy of RRFCs with composite concrete decks. Copies of the proposed guide specifications will also be made available to the audience for download via web-based distribution.

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Flexible Buried Bridges for Low-Volume Road Crossings

JOEL HAHM
Big R Bridge

INTRODUCTION

Structural plate buried structures have been in use for over 80 years. These types of structures started out as a deeper corrugation profile option than traditional corrugated steel pipe (CSP) for use in hydraulic and minor crossings where CSP could not meet flow and size requirements or where bottomless structures were needed. These structures have traditionally consisted of shallow corrugated structural plate (6- x 2-in. corrugation profile) and were limited to drainage applications.

Since the development of deep corrugation structures (>5-in. corrugation depth) about 40 years ago there has been a significant increase in the use of structural plate as bridge length structures in larger hydraulic crossings and grade separation applications where traditional bridges and precast concrete structures have historically been used. There have not been studies published to document the percentage of buried bridges in use compared to short to medium span traditional bridges, but buried bridges are increasingly being specified or allowed as alternatives to traditional bridges for many transportation projects by various agencies.

Structural plate structures are flexible buried structures where the structure consists of steel plates that have been corrugated, shaped to a specific curvature, hot-dipped galvanized, and then bolted together to construct an arch- or box-shaped structure. After the structure has been assembled it is backfilled using granular backfill to complete the bridge crossing. These structures work through soil-structure interaction, where the structure and surrounding engineered backfill work together to support the design loads. Buried bridges are structural plate structures with a span of greater than 20 ft.



Bridge built by tribal forces near Craig, Arkansas.

It used to be the case that structural plate buried bridges were only used in smaller culvert type crossings (<20-ft span), but until recently they were not being widely considered for short to medium bridge length crossings. The increased consideration been made possible by advancements in design and analysis tools, manufacturing capabilities, materials, and development of deeper corrugation profiles to allow for longer spans, heavier loads, and higher cover. These types of structures can be used for a wide variety of applications and have become more common in low-volume road bridge applications in recent years because of economy, construction advantages, ease of transportation, environmental benefits, low maintenance costs, resilience, and other factors. This extended abstract summarizes many of the advantages of using buried bridges for low-volume road crossings.

METHODOLOGY

Concepts presented in this paper are based on experience by the author gained through design and construction of hundreds of buried bridges used in a variety of locations and applications. Many case studies illustrating the findings below and how buried bridges can be used on low-volume roads are available.

FINDINGS

On most low-volume road bridge projects there is a desire to construct the bridge as quickly as possible, minimize costs, and avoid the need to hire a specialized contractor. Buried bridge structures fit well into the accelerated bridge construction model in terms of quicker lead times for materials, fewer construction steps with no lag time between steps, no need for specialized labor or heavy equipment (many counties and general contractors prefer to use their own equipment and forces whenever possible), elimination of bridge abutments, and often a smaller construction footprint. In some cases it is possible to complete a buried bridge project without delivering concrete to the site. This is especially beneficial in remote areas where it may be difficult or cost prohibitive to mobilize large equipment. Individual plates used to form the



Forest Service bridge in Unity, Oregon.

bridge structure typically weigh between about 300 and 1,200 lb each and are easily stacked in bundles on a flatbed truck or in a shipping container (when being shipped by rail or ship). In remote areas these bundles can be broken down and plates carried individually with light equipment, making them the only viable bridge option.

Inspection and maintenance of buried bridges do not require special training or specialized personnel and have a much lower cost compared to traditional bridges (e.g., pavement maintenance versus deck repairs or replacement, no “bump” at the end of the bridge). Inspection of the underside of buried bridges can be performed by almost anyone with minimal training.

There has been an increased focus in recent years on

- Restoration of aquatic organism habitat,
- Limiting areas of disturbance,
- Protecting wildlife,
- Natural stream beds for fish passage, and
- Various sustainability initiatives.

As a result, owners have been faced with the challenge of finding “greener” solutions on infrastructure projects. Buried bridges can be an effective way of meeting those requirements when compared to traditional bridges. Environmental advantages include the ability to provide wide enough spans allow natural stream beds and be outside of the limits of disturbance, wildlife crossing separations, ~ 75% + recycled content, ability to easily transport to remote areas and install with light equipment and little manpower, and a more natural look to fit into the surroundings—especially when on-site materials can be incorporated into end treatments.



Residential development in Houston, Texas.

Resilience is emerging as a critical topic in the transportation industry. In some ways, the ability to maintain functionality after extreme events can be more critical in areas served by low-volume roads than in more-populated areas. Buried bridges can provide added resilience compared to traditional bridge structures through redundant structural systems, flexibility, and high tolerance for differential settlement, ability to remain in service after seismic events, and the engineered backfill acting as a buffer against impact or explosions.

When proper design and site considerations are made, structures are properly constructed with appropriate engineered backfill materials, structures are properly maintained by the owner, and structures are used for their intended purpose, a service life of 75 years or greater can be achieved. This topic was addressed in a Transportation Research Board webinar in September 2018.

Buried bridges are not proprietary and information needed for anyone to design, manufacture, and construct buried bridges is available in current AASHTO specifications:

- Sections 12.8 and 12.9 of the current AASHTO *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*,
- Section 26 of the current AASHTO *LRFD Bridge Construction Specifications*, and
- Standard M167 in the AASHTO *Materials Specifications*.

Although structural plate structures have been in use for a long time and their use as buried bridges has been incorporated into AASHTO LRFD specifications, many engineers are not aware of industry advances or the benefits of using them as an alternative to traditional bridges. Most manufacturers will include the structure design along with the bridge materials supplied for a buried bridge project and are available as resource to agencies and consultants as they consider and develop buried bridge projects.

Many low-volume road bridges are located in areas with significant vehicle loads such as mining vehicles (haul trucks, shovels, and drag lines with loads ranging between about 400,000 and 8 million lbs), agricultural equipment (grain trucks, fertilizers, and other vehicles that are not legal loads on U.S. highways), and equipment associated with oil and gas production (large trucks, drill rigs, and cranes). Design and construction of traditional bridges to carry these loads for a relatively short period of time is often cost prohibitive or not feasible with some site conditions. In



Grade separation—county road over dual track rail crossing near LaCygne, Kansas.



Replacement for timber bridge on farm road in Buchanan County, Iowa.

addition to being resilient, buried bridges are able to safely and economically carry heavy vehicle loads by optimizing structure shapes and taking full advantage of soil–structure interaction. Wheel loads are distributed through granular fill, lightening their impact on the underlying structure. Structure shapes can be easily customized to optimize the design efficiency and strength of the structure.

Buried bridge spans can vary widely based on the application and site requirements and limitations. Grade separation structures (usually with traffic passing through the structure) will typically have spans on the order of 50 ft or greater while structures used for hydraulic crossings can range from about 20 ft to over 60 ft depending on the stream width and other hydraulic requirements. The largest spans to date are in the 75 to 80 ft range in North America, with recent structures constructed with a span of 92 ft in Poland and 107 ft in the United Arab Emirates.

CONCLUSION

Low-volume road bridges need to provide the same basic functions and have the same design standards and loading requirements as highway bridges with higher traffic volumes, but agencies building low-volume road bridges typically don't have the same resources to design and construct them. Low-volume road bridges need to be economical, easy to build, and in many cases capable of supporting higher vehicle loads than bridges in more populated areas. Flexible buried bridges are able to meet these needs in many cases and are a viable alternative to consider in low to medium span bridge applications.

Climate Change Resiliency

CLIMATE CHANGE RESILIENCY

Lessons Learned and Recommendations from Embedding Climate Change Adaptation into the Roads Sector

From Policy to Practice

ALIZE LE ROUX

JOHAN MARITZ

KATHRYN ARNOLD

BENOIT VERHAEGHE

MICHAEL ROUX

Council for Scientific and Industrial Research

INTRODUCTION

African countries tend to be particularly vulnerable to the effects of climate variability and in the past four decades more than 1,400 recorded weather related disasters (meteorological, hydrological and climatological) have caused havoc on countries' economies and for rural communities in particular, on their access and sustainable livelihoods. Communities in Africa are projected to be some of the worst affected by climate change, in part due to their high socio-economic vulnerability, growing rural populations, high dependency on natural resources and low adaptive capacity, but also due to the relatively strong climate change signal consistently being projected for most of the continent (Le Roux et al., 2016).

For African countries, the lack of adequate road infrastructure and the long distances to markets and essential services have been a major development hurdle and continue to make rural communities especially susceptible to the impacts of climate variability. The African continent is facing an enormous economic cost to repair and maintain roads damaged from temperature and precipitation changes directly related to projected climate change. There is thus a pressing and urgent need to mobilize resources to address the continent's current limitations to deal with climate events and to deal with future climate change.

A series of more targeted adaptation interventions are required and it is crucial that decision-makers in all sectors across all governmental spheres factor climate change into all their long term strategic decision making processes. Although many of the countries in sub-Saharan Africa have developed policies to deal with climate change, these policies have not always been translated into tangible actions in the relevant sectors and the transport sector specifically (especially rural roads) has not featured strongly on the climate change adaptation agenda.

There is thus a critical need for the prioritization of climate adaptation mainstreaming and embedment thereof into policies, systems, thinking and practice. Regional guidance is also needed on the development of climate-resilient road infrastructure to ensure that sustainable and inclusive development materializes in vulnerable rural areas most at risk to the devastating impacts of a changing climate (Head et al., 2018b).

METHODOLOGY

As part of this plea the Africa Community Access Partnership (AfCAP), a research program funded by UKAid, commissioned a multidisciplinary project to produce regional guidance on the development of climate-resilient rural access in Africa with the aim of assisting the development of a climate-resilient road network that reaches fully into and between rural communities.

The focus of this multidisciplinary multiyear project was on enhancing the capacity of national road authorities in Africa to reduce the current and future impacts of climate change on their rural roads sector. Through research and knowledge sharing, the objective was to compile pragmatic cost-beneficial engineering and non-engineering procedures that could be used to guide road-sector authorities to address climate threats more efficiently and effectively.

Through the project, a five-stage methodology for carrying out climate adaptation assessments for rural roads was developed and packaged into a handbook that provides a range of interventions from informing national policies, through regional and district investment planning, down to practical guidance on adaptation delivery at project level (Head et al., 2018b). The handbook has been produced to provide relevant information on adaptation procedures for both new and existing rural road access.

The handbook, which illustrates the fundamental principles, processes and steps required for climate resilience is accompanied by a set of guideline documents. Details regarding actual adaptation measures and vulnerability assessment methodologies are included in the supporting guidelines documents covering

1. Climate risk and vulnerability assessment (Le Roux et al., 2019);
2. Change management (Head et al., 2018b); and
3. Engineering guidelines (Page-Green et al., 2018).

Each one of these guideline documents has been tested and trialed in three AfCAP countries, namely Mozambique, Ethiopia, and Ghana to ensure that effective capacity enhancement, as well as the embedment and uptake of climate adaptation in the road sector is realized.

FINDINGS

The process of trialing the embedment of climate change adaptation into the three respective national transport authorities brought to light a number of challenges and constraints that had to be addressed. Obstacles that were faced by the team included policy gaps that fail to adequately address climate change in the transport sector, the lack of in-country collaboration between sectors when dealing with climate change science, the challenge of working with multiple actors from different decision-making environments, the scarcity of data when conducting analysis in resource-stricken countries, the inconsistency between African countries in the quality and quantity of available data, and restrictive institutional policies (e.g., data-sharing protocols) regarding ownership and custodianship of data sources.

Policy (as it relates to the roads sector) is integral for supporting change management as it sets the scope and content of strategic planning for programs and plans which, when implemented, will support the creation of more sustainable rural access (Figure 1). Considering the transport policies of the three participating countries, it is only recently that new policies are

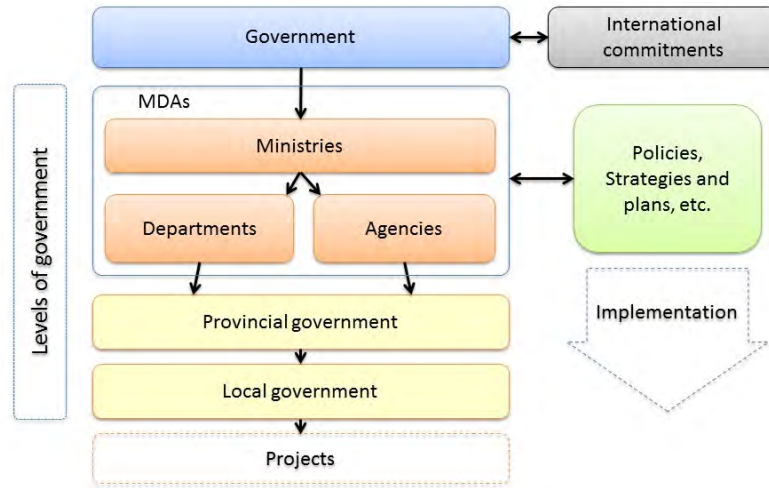


FIGURE 1 Generalized government structure, with high-level policy to project-level implementation links.

featuring climate change more prominently, but in all cases, the scope of these policy do not extend to rural roads, and in most cases have not yet been translated into action. Policies related to incorporating climate change screening into road asset management systems are also severely lacking. These findings affected the practical inclusion of climate change content into respective design guidelines and road asset management systems. It was also found that in the three countries, no person, department, or institution was tasked with the responsibility to actively address the issue of climate change as it relates to the transport sector. A related gap was therefore the need to identify staff responsible for driving climate change implementation within the respective institutions (Maritz et al., Submitted).

Key to addressing many of these challenges were multisector engagements, specifically early engagements in the developmental phases and constant innovation in moving forward. Overcoming many of the restrictive policies and legislations relating to the right of access to information included the formulation of innovative methodologies and assessment frameworks that bridged the data sharing gaps. Overcoming constraints with regards to the quality, quantity and availability of data involved (e.g., sourcing data from reputable open data platforms and tools). Continuous engagement with a wide range of participants enabled effective and efficient stakeholder communication, collaboration and involvement during the work processes. Stakeholders included ministries, departments, authorities, institutions, and research organizations as well as nongovernmental organizations.

CONCLUSION

The presentation will briefly highlight the need for adaptation planning in the rural roads sector and will then progress to explain how this need has been addressed through the development and delivery of a five-stage adaptation handbook. The presentation will reflect on the multidisciplinary project team's experience the past 2.5 years and share lessons learned with the audience on some of the challenges, obstacles, and victories in developing, trialing and

embedding climate change adaptation into the rural roads sector. The presentation is meant to stimulate a debate on the way in which we think, act, plan, and design climate-resilient rural access roads. The presentation will conclude with a series of recommendations for the embedment of climate change adaptation that spans across the entire sector, from high-level policy intervention all the way to proactive actions at project level.

ACKNOWLEDGMENTS

This paper is the result of a research program that was commissioned by the AfCAP, a research program funded by UKAid. The authors also express their gratitude to the roads authorities in Ghana, Ethiopia, and Mozambique for their inputs into the refinement of the guidelines and handbook.

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Stabilization

STABILIZATION

Stabilizing Sand Roads Using Pulp and Paper Mill Boiler Ash

MARK L. RUSSELL

United States Army Corps of Engineers, Los Angeles District

Gravel is the most common surfacing material for unpaved roads and the most common material for stabilizing unpaved roads. However, suitable gravel is often unavailable or expensive in some areas because of a lack of natural gravel deposits or for other reasons.

In such cases, alternate road stabilization materials are needed. Boiler ash from energy generation at pulp and paper mills is a wood-based industrial byproduct that is common in forested regions that often have thriving timber and paper industries. Beneficial re-use of industrial byproducts including boiler ash, is environmentally desirable. Benefits of using pulp and paper mill boiler ash for road stabilization include reduced gravel mining and hauling and reduced solid waste going to landfills.

A demonstration project used various applications of paper mill boiler ash to stabilize two low-volume roads with different soil types. The road subgrade properties were tested in-place and in the lab before and after road stabilization. This paper presents the construction procedures, test results, information on the performance of the two treated roads, the environmental concerns, and a comparison of costs with similar gravel surfacing road projects. Pulp and paper mill boiler ash appears to be effective for road stabilization especially in very sandy soils, and may be cost-competitive with other stabilization techniques in many situations.

INTRODUCTION

There are millions of miles of low-volume roads throughout the world. The United States Forest Service (USFS) alone maintains over 380,000 mi (611,550 km) of forest roads, and the majority of those are neither paved nor surfaced with gravel. Gravel and mineral aggregates are the most common surfacing material for unpaved roads and the most common material for stabilizing subgrades for such roads. However, such materials are often unavailable or expensive in some areas because of a lack of natural gravel deposits (e.g., regions of widespread loose sandy soil, or soft clay soils) or for other reasons.

Boiler ash is a potential alternate road stabilization material. Boiler ash from energy generation at pulp and paper mills is an industrial byproduct that is common in forested regions that often have thriving timber, pulp, and paper industries. It is a waste or byproduct of pulp and paper manufacturing that usually is discarded or disposed of in sanitary landfills. Its disposal is a cost to the mill so re-use of the material rather than disposal is a financial benefit to the mill and is commonly encouraged by states as an environmentally sound re-use of material to reduce the volume of solid wastes going to landfills. Since pulp and paper mill boiler ash comes primarily from wood materials, the recycling aspect of boiler ash re-use is particularly evident (essentially returning wood materials to their place of origin). Beneficial re-use of industrial byproducts in-

cluding boiler ash is a major goal for both industry and environmental regulators. Benefits of using pulp and paper mill boiler ash, for road stabilization include reduced gravel mining and hauling, and reduced solid waste going to landfills.

The USFS conducted a demonstration project using various application rates of pulp and paper mill boiler ash to stabilize two low-volume roads with different soil types. Road subgrades were tested before and after road stabilization. This paper presents the construction procedures used, the results of field and laboratory tests, and description of treated road performance, summarizes the environmental concerns and environmental advantages, and compares the construction costs with those of typical gravel roads.

WHAT IS BOILER ASH?

The waste materials from energy production at pulp and paper mills or municipal power plants using wood materials as fuel are known as “boiler ash” or “furnace ash residuals.” The dry ash solids resulting from combustion of wood-based fuels (biomass) for energy generation is primarily wood ash. Over 4 million dry tons of wood ash are generated annually in the United States. Forest management residues (slash) and lumber mill waste (bark, sawdust, and wood chips) provide “clean” sources of woody biomass for combustion.

Boiler ash consists of both “bottom ash” (heavier material deposited at the bottom of the furnace) and “fly ash” (the lighter particulate materials captured from exhaust gases). Bottom ash occurs naturally as a solid waste that must be handled and disposed of or otherwise used. Fly ash, on the other hand, is collected as an air pollution control measure. Bottom ash has qualities similar to a mineral or gravel–sand mixture while fly ash is a powder more closely resembling portland cement. Fly ash is finer and more reactive than bottom ash. The term fly ash is most commonly associated with combustion from coal-fired power plants, but is used here in its more general sense of any combustion ash, including wood ash, captured from exhaust gases. Boiler ash from a wood-burning plant is typically high in unburned carbon, magnesium, and calcium and has a high pH (1). As an example, data from the Flambeau River Papers (formerly Fraser Papers) plant in Wisconsin are summarized in Table 1.

The physical properties of wood ash (both bottom ash and fly ash) can be compared to those for coal fly ash, which is commonly used in portland cement as a pozzolanic admixture. One study tested 12 wood ash samples from various sources for properties that are used to evaluate the quality of fly ash for portland cement concrete (2). Table 2 shows results of those tests and their comparison to requirements for coal fly ash in concrete given by ASTM D618. As part of this study, samples of fly ash and bottom ash from the Flambeau River Papers were examined and tested them for strength activity index as summarized in Table 3.

Note that neither the wood fly ash or bottom ash materials meet most of the ASTM C618 requirements for coal fly ash used in concrete. Nevertheless, they show beneficial qualities for road stabilization as discussed below.

PREVIOUS EXPERIENCE USING BOILER ASH AS A ROAD STABILIZER

Wood ash from energy generation (boiler ash) has been shown to be effective for stabilizing soil subgrades and road base materials in northern Europe (1, 3–12). Most of those uses involved

TABLE 1 Composition of Fly Ash and Bottom Ash from Flambeau River Papers Mill

Ingredient	Formula	Fly Ash	Bottom Ash
Aluminosilicate, glass-ceramic	(Al, Si, Ca, Fe, Mg, K, Na)	60%–70%	95%–100%
Unburned carbon and wood fiber	C, (C ₆ H ₁₀ O ₅)	<20%	<2%
Crystalline silica	SiO ₂	<5%	0%
Calcium compounds	CaCO ₃ , Ca(OH) ₂ , others	<20%	0%
Calcium sulfate, potassium sulfate	CaSO ₄ , KSO ₄	<10%	<3%
Crystalline silica	SiO ₂	<5%	0%
Iron mineral	Fe ₂ O ₃ , Fe ₃ O ₄	<2%	<3%

TABLE 2 Results of Physical Property Tests on Wood Ashes and Comparison to Coal Fly Ash Specification (ASTM D618) Requirements

Parameter	Wood Fly Ash Samples (2)		Wood Bottom Ash Samples (2)		ASTM D618 Requirement
	Range	Average	Range	Average	
Retained on No. 325	3.6–98.3	46	—	—	<34
28-day strength activity index with cement (% of control)	48.9–123.8	73	—	—	>75
Water requirement (% of control)	103–155	118	—	—	<105–<115
Autoclave extension (%)	0.01–0.63	0.10	—	—	+0.8
SiO ₂ + Al ₂ O ₃ + Fe ₂ O ₃ (%)	10.0–72.2	40.7	56.9–93.4	73.8	>50.0–>70.0
Sulfate, SO ₃ (%)	0.1–15.3	4.8	0.1–0.7	0.4	<5.0
Loss on ignition (%)	6.7–58.1	23.4	1.4–33.2	9.9	<6.0
Available alkaline, Na ₂ O equivalent (%)	0.4–20.4	3.3	—	—	<1.5

TABLE 3 Strength Activity Index Test Results for Wood Ashes (28-day SAI, % of Control)

Fly Ash Samples (2)	Fly Ash Sample ^a	Combination of 40% Bottom Ash with 60% Fly Ash ^a	ASTM C618 Requirement
48.9–123.8 73 (average)	68	63	> 75

^a From Flambeau River Papers, 2016.

wood fly ash with only occasional mentions of bottom ash, and most of them involved ash from pulp and paper mills although wood ash is sometimes obtained from municipal power-generating plants as well. Some of these studies refer to the ash as “bio fly ash” or “biofuel fly ash.”

Detailed chemical and geotechnical field and lab testing on demonstration projects involving low-volume roads in Sweden, Finland, and Austria and have been conducted using wood ash as a road stabilizer. One study in Sweden involved mostly fine-grained soils and road test

sections stabilized with gravel and with 30% wood fly ash, compared to an untreated control section, and monitored for 2 to 3 years (13). Another study in Sweden involved wood fly ash from a pulp–paper mill blended with “road base material” (presumably gravel) at a rate of 20% to 30% (3). The study in Austria involved both fly ash and bottom ash from woody biomass fuels and focused on using the ash as a substitute for lime for road stabilization in clay and silt soils (10). The study in Finland (7, 8) involved 10 unpaved forest roads stabilized with fly ash from wood and peat and monitored for 3 years. Each of these studies indicated generally favorable results from the use of wood ash, although they also point out the variables that can affect road performance including weather, soil moisture, variability in ash–soil blending, degree of compaction, and other issues.

Laboratory research at the University of Guelph (Ontario, Canada) demonstrated that wood fly ash [with high loss on ignition (unburned carbon content = 21%) and low calcium (Ca = 250 mg/kg)] could improve the strength and stiffness of soil (14). Field testing confirmed the lab results for that study by treating a landfill haul road having clayey soil, which resulted in reduced rutting. The pulp mill involved in that research subsequently began routinely treating forest haul roads with the fly ash, and the treated roads have an increased allowable traffic load.

ENVIRONMENTAL CONSIDERATIONS

Regional and local governments typically regulate land application of pulp and paper mill boiler ash, although it is also potentially subject to national statutes. Because pulp and paper mill boiler ash is not defined as a hazardous waste, it is not regulated under the Resource Conservation and Recovery Act in the United States. Compared to the uncombusted wood fuel, both fly and bottom ash contain concentrated chemical elements resulting from the removal of volatile components and the reduction of carbon mass. Fly ash tends to have higher concentrations of some heavy metals than bottom ash including zinc, lead, and cadmium, but bottom ash tends to have higher concentrations of copper, nickel, and chromium (10). As with any other industrial byproduct, the pH, toxicity, and leachability of trace constituents (such as certain organics and heavy metals) from pulp and paper mill boiler ash must be assessed prior to any beneficial use application.

Industrial wastes that have a beneficial use—even if they cannot be sold commercially—are generally considered industrial byproducts and are regulated differently than wastes that are disposed of in landfills or are otherwise disposed of. Regulators often permit, and sometimes encourage, re-use of industrial byproducts that are nonhazardous and can be used safely.

For example, the state of Wisconsin—which has significant timber, pulp, and paper industries—established a policy that addresses beneficial use of industrial byproducts (15). The Wisconsin Department of Natural Resources has allowed land application of each of these materials (sludge, ash, lignosulfonates, and lime grit), subject to certain limitations and monitoring (16).

Florida, another state with significant timber and paper industries, also has a policy regarding beneficial use of industrial byproducts (17). The Florida Department of Environmental Protection appears to have more-stringent regulations than Wisconsin at this time but has allowed use of some boiler ash (petroleum coke and coal ash from power plants) blended with lime for road stabilization (18, 19) as well as other land applications of ash byproducts.

Wood ash (boiler ash) from energy generation at pulp and paper mills, in general, is benign and is essentially the same as naturally occurring ash resulting from forest fires. When a plant uses coal or other fuels in combination with wood wastes, however, constituents of concern can be present in the ash. Most metals concentrations in wood-fired boiler ash are similar to those of many natural soils, municipal treatment plant biosolids, and agricultural limes, but much lower than those from coal fly ashes (20).

Boiler ash from the Flambeau River Papers mill was evaluated as a typical material for this study. That ash is analyzed periodically for various chemical constituents. Testing for polycyclic aromatic hydrocarbons (PAHs) in 2015 showed no detectable concentrations of any PAH in bottom ash, and no detectable concentrations of any PAH except naphthalene in fly ash. Naphthalene was detected at 260 micrograms per kilogram (ug/kg) in the fly ash sample (16). Naphthalene is a commonly occurring substance found naturally in crude oil and tar and is used to make certain products such as mothballs. Naphthalene levels in soil are not regulated at the federal level but the California 2013 Tier 1 Environmental Screening Level (used here as a convenient reference standard) is 1.2 mg/kg which is well above the concentration found in the boiler ash from the Flambeau River Papers mill. Chemical analysis of bottom ash and fly ash from 2011 and 2014 showed concentrations of metals generally within ranges common to natural soils and none exceeding the Wisconsin Department of Natural Resources (WDNR) standard or the U.S. federal total threshold limit concentration value. The ash samples were also subject to solubility testing. None of the measured concentrations of metals exceeded the WDNR standard or the U.S. federal soluble threshold limit concentration value. Flambeau Papers obtained an open-end approval from WDNR for land application of their high-volume industrial wastes, including boiler ash. They also obtained specific approval for use of boiler ash on the two demonstration roads described here.

FIELD DEMONSTRATION PROJECTS

The USFS conducted projects using wood boiler ash between 2013 and 2016 to investigate the effectiveness of the material for low-volume road stabilization. The USFS selected the Chequamegon–Nicolet National Forest in northern Wisconsin as a study area because of a range of soil types including unstable road subgrade soils such as cohesionless “sugar sands” and because there is an active paper mill (Flambeau River Papers) there that uses biomass as fuel and generates boiler ash. Two low-volume unpaved forest roads were selected: FR685 containing poorly graded loose sand sugar sand (SP soil type per the Unified Soil Classification System) and FR1155 containing silty fine- to medium-grained sand (SM soil type). The following steps were taken at each of the two roads:

1. Conducted falling weight deflectometer (FWD) testing on the untreated road grades at approximately 0.1-mi (0.2-km) intervals.
2. Cleared vegetation and graded and shaped the road prism.
3. Conducted dynamic cone penetrometer (DCP) testing on the cleared road grade at approximately 0.1-mi (0.2-km) intervals.
4. Tested the maximum density optimum moisture and California bearing ratio (CBR) for the untreated road grade soils.

5. Spread boiler ash on the road grade to be blended with the upper 3 in. (7.6 cm) of soil at the following proportions:
 - a. 3 in. (7.6 cm) thick (50%–50% ash–soil mix) on first 1/3-mi (0.5-km) road segment;
 - b. 2 in. (5.1 cm) thick (40%–60% ash–soil mix) on second 1/3-mi (0.5-km) road segment; and
 - c. 1 in. (2.5 cm) thick (25%–75% ash–soil mix) on last 1/3-mi (0.5-km) road segment.
6. Applied some water at selected locations to adjust soil moisture content.
7. Blended the ash with the soil (five to six round-trips, 10 to 12 passes) to achieve an approximately 6-in. thick (15-cm) ash–soil treated zone.
8. Tested the maximum density optimum moisture and CBR of the blended ash–soil mixture.
9. Compacted the blended ash–soil and test compacted grade to verify at least 90% of maximum dry density (approximately four passes with vibration and four passes in static mode).
10. Conducted DCP testing and FWD testing on the treated and compacted road grades at 0.1-mi (0.2-km) intervals.
11. Surveyed road cross sections at 0.1-mi (0.2-km) intervals to provide a baseline for future rutting or settlement of the road sections.
12. Recorded traffic volumes on the roads using automatic traffic counters.
13. After 1 year of service, conducted DCP testing and FWD testing and re-surveyed cross sections on the treated and compacted road grades at 0.1-mi (0.2-km) intervals.

To keep costs low and to demonstrate the kind of construction that would actually be expected on typical low-volume forest roads, only conventional readily available construction equipment was used. Boiler ash was delivered by end-dump trucks; belly-dump trucks would have been preferred to facilitate spreading, but were not available at the time of the demonstration. The boiler ash was spread using a front-end loader and a motor grader. It was mixed into the soil using an agricultural disk pulled by a tractor, and water was added using a trailer-mounted water tank. The blended material was compacted using a smooth-wheeled vibratory compactor.

TEST RESULTS

FWD test results from the siltier of the two roads (FR 1155) showed improved stiffness immediately after treatment and still more improvement 1 year later. Figure 1 shows the measured deflections along the length of the test section of FR1155. Figure 2 shows the back-calculated modulus for the same test events on that road. FWD test results on the road with the sugar sand soil (FR 685) indicated a softer road subgrade for the test event shortly after boiler ash treatment (September 2015) than the test event before initial grading (July 2015), but then stiffer than initial conditions a year later (August 2016). This temporary loss of stiffness may be more a function of soil moisture from rain events than a function of the road grading, ash treatment, and compaction. Or it may be that scarification and treatment with boiler ash actually loosened the soil somewhat compared to the initial condition of the roads in spite of compaction after applying ash. The subsequent subgrade stiffening shown in the 2016 data a year later may reflect curing of the cementitious elements in the boiler ash and settlement or compaction of the soil from traffic and weather.

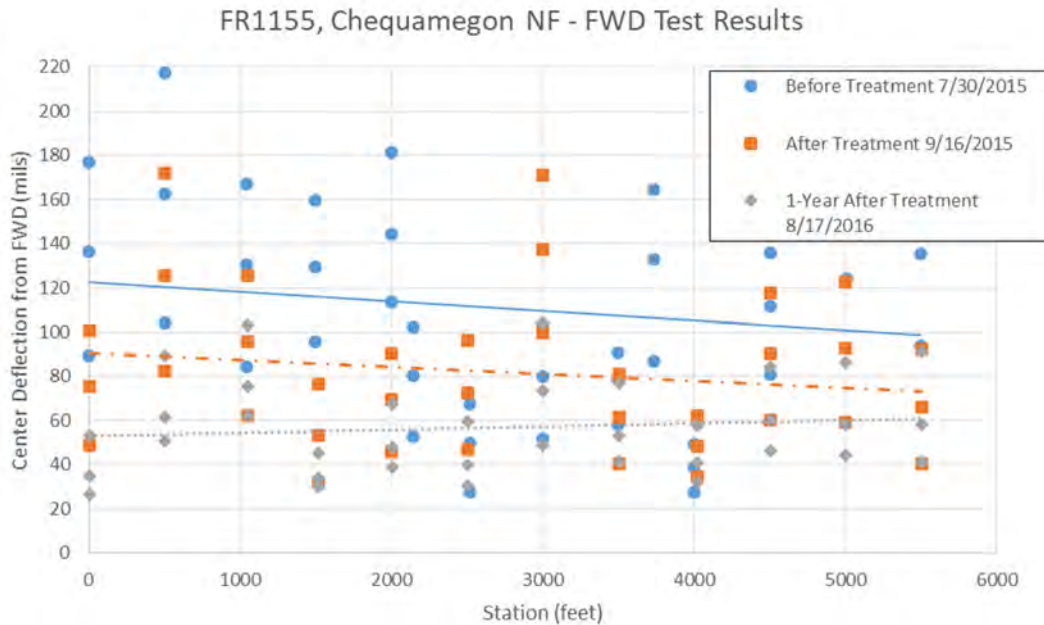


FIGURE 1 Typical FWD test results, deflection.

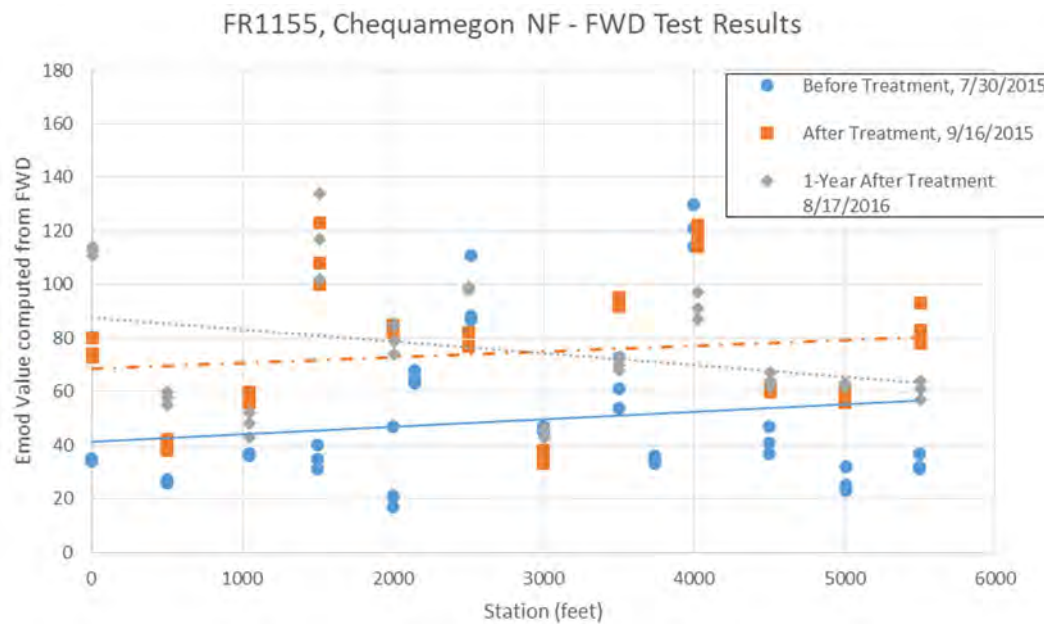
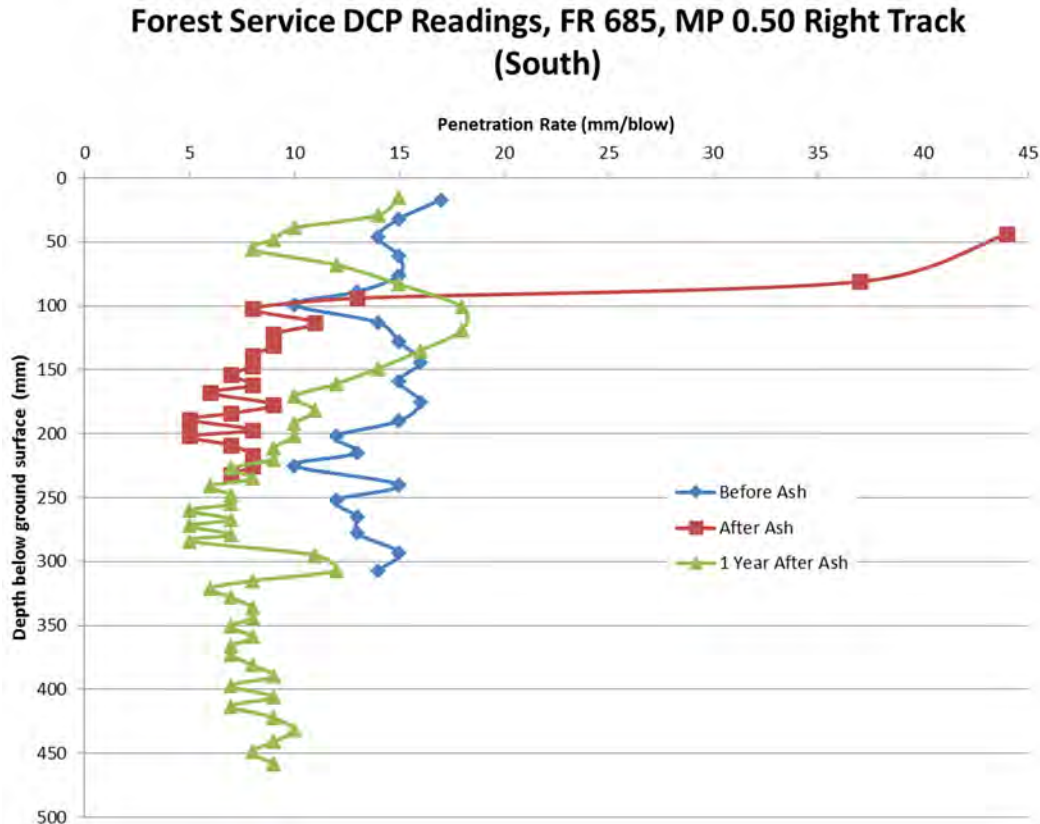


FIGURE 2 Typical FWD test results, modulus.

As with the FWD testing, DCP testing was conducted after the initial road grading but before placement of boiler ash, and then again after ash placement and compaction. Figure 3 shows a typical DCP test at one location along the sugar sand road (FR 685). The DCP tests also showed results that varied with time. In the test shown on Figure 3, that results indicate improvement in stiffness (lower penetration rate) for the test event shortly after boiler ash treatment (September 2015) compared to the baseline event (July 2015), but then somewhat softer (higher



penetration rate) 1 year later (August 2016), although the 1-year average results were still stiffer than the baseline. As with the temporal variations in FWD results, this may be a function of soil moisture variations due to weather, curing of cementitious elements in the ash, or soil compaction due to traffic. Although there was much variability, the average DCP penetration rates throughout both roads 1 year after treatment were lower (stiffer soil) than the average untreated baseline DCP penetration rates.

The field testing suggests that boiler ash treatment generally improves subgrade stiffness compared to untreated subgrades for the soil types tested. The reasons for the temporal variation in apparent subgrade stiffness is not clear; further study would be needed to evaluate it.

Geotechnical laboratory tests compared the basic engineering properties of untreated soils with those of various soil-boiler ash blends. The testing focused on parameters related to road surface stability and included grain-size distribution (gradation), compaction (Proctor test), strength (unconfined compressive strength), and bearing (CBR).

As shown on Figure 4, the sugar sands from FR 685 were poorly graded and clean (relatively free of silt and clay) but their blends with boiler ash are more well graded. Blending with ash provides fines that tend to act as binder to help stabilize clean sands.

As shown on Figure 5, which presents the Proctor compaction test results, the addition of boiler ash to the soils reduced the maximum density and shifted the optimum moisture content to the right (wetter). This does not necessarily indicate a more stable material but is to be expected with given the higher fines content.

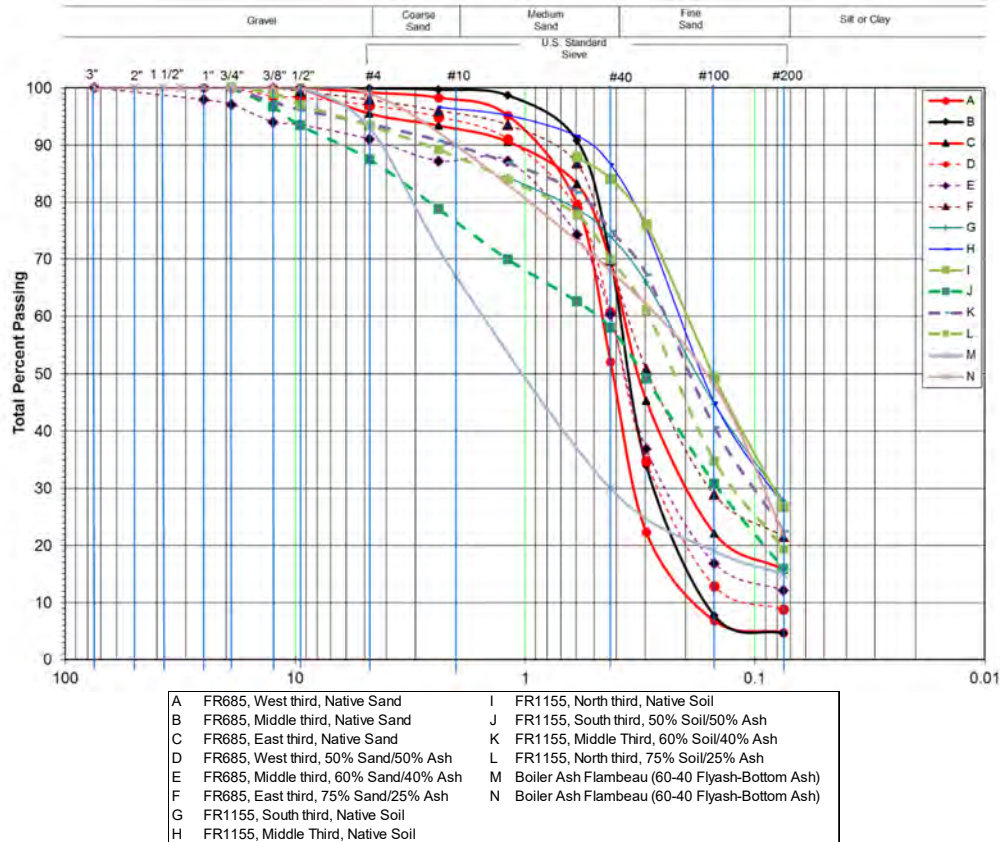


FIGURE 4 Particle size distribution of various materials tested.

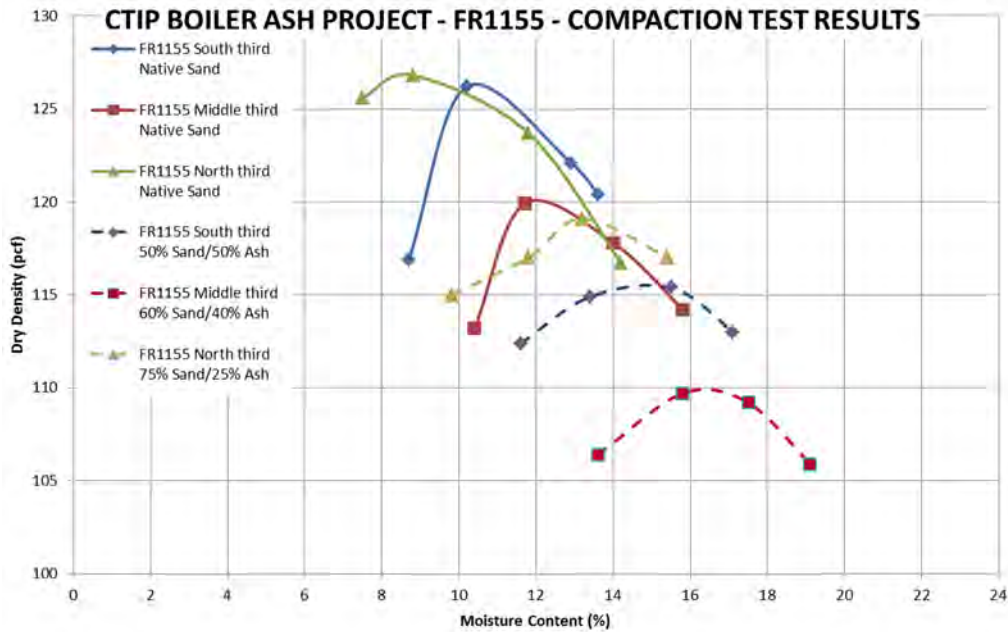


FIGURE 5 Maximum density versus optimum moisture relationship for native soil and ash-blended soil.

The unconfined compressive strength of compacted 50–50 blends of ash and soil show some strength gain with time suggesting that the combined ash has cementitious properties (see Figure 6).

Figure 7 shows CBR test results for treated and untreated soil samples from both roads. The addition of boiler ash clearly improved the subgrade stability for both roads, although the amount of improvement in CBR value did not correlate clearly with the percent of ash in the blend.

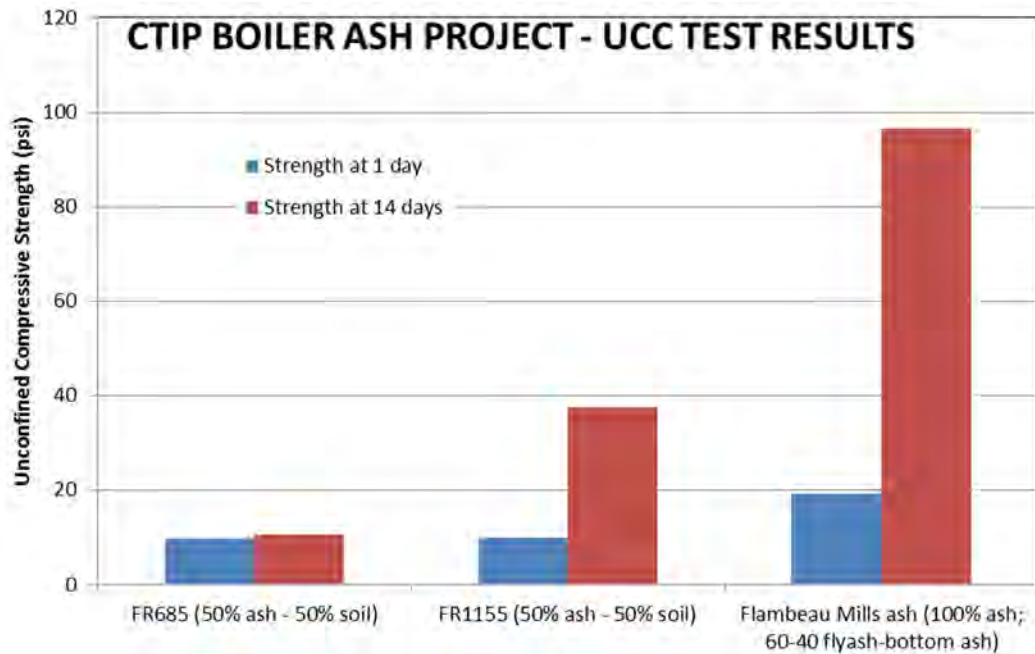


FIGURE 6 Unconfined compressive strength results.

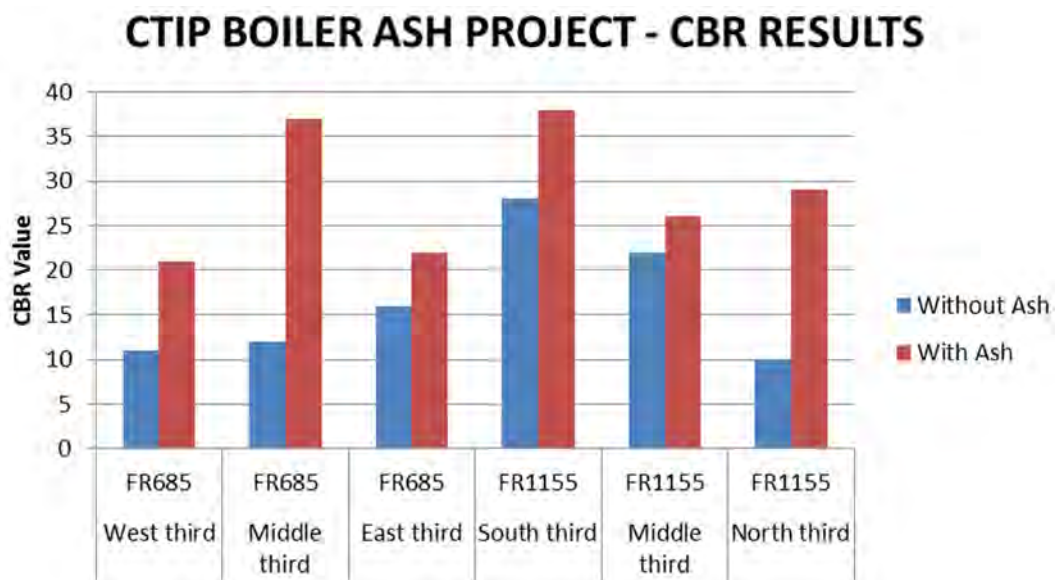


FIGURE 7 CBR for various soils and boiler ash amendment blends.

TRAFFIC IMPACT

Both roads are considered “very low-volume roads” with generally light seasonal traffic (Table 4). Based on visual observations and the results of surveys, both roads performed better after treatment. Of the two demonstration roads, the sugar sand road, FR 685, benefited most from ash stabilization. The extreme rutting observed before treatment was virtually eliminated and the road kept its shape throughout the 1-year test period that included the full cycle of seasonal ground conditions and the seasonal (mostly summer) traffic. Surveys showed very little change (mostly less than 0.2 ft, 6 cm) in road surface elevation over the year.

FR 1155, with silty sand or sandy loam subgrade, did not have the extreme rutting or traction problems to start with that the sandy road (FR 685) experienced prior to treatment. After the 1-year monitoring period, the ash-treated road FR 685 appeared to be in roughly the same condition as it was after the initial clearing and shaping, but prior to ash stabilization. Cross sections showed the road generally holding its shape over the year but with a few ruts of half a foot (15 cm) or more.

COST COMPARISON

Boiler ash for the demonstration project was stockpiled by the Flambeau River Papers mill prior to construction and provided free of charge to the USFS. Approximately 400 yd³, CY (300 m³) of boiler ash was applied to each of the two roads. Table 5 summarizes the actual project costs for the two boiler ash road stabilization projects and presents the hypothetical approximate cost if the same roads had been surfaced with six inches of gravel. A cost savings of approximately 57% was estimated for both roads.

SUMMARY AND CONCLUSIONS

Unpaved low-volume roads often need stabilization when native soils are overly soft, loose, wet, or otherwise unstable. Where gravel and aggregate are not available or not economical, alternate stabilizing materials are necessary. Commercially available products such as portland cement, coal flyash, lignosulfonates, and proprietary stabilizers can be used as alternatives to stabilize unpaved road grades but are expensive.

Boiler ash, a pulp and paper mill byproduct, was considered as a road-stabilizing alternative. Boiler ash is often readily available for little or no cost in areas with a pulp or paper industry. It is a combination of bottom ash and fly ash from wood combustion for energy production

TABLE 4 Traffic Data

Road	Period	Traffic Type	Total Traffic Count	ADT
FR685	5/25/2016 to 9/27/2016	Passenger cars and pickup trucks	783	6.3
FR1155	5/9/2016 to 10/4/2016	Light vehicles and heavy trucks	814	5.5

NOTE: ADT = average daily traffic.

TABLE 5 Project Cost Summary

Item	FR1155 Boiler Ash	FR1155 Gravel Surfacing	FR685 Boiler Ash	FR685 Gravel Surfacing
	Actual Costs (400 CY Ash)	Hypothetical Costs (1500 CY Gravel)	Actual Costs (400 CY Ash)	Hypothetical Costs (1500 CY Gravel)
Purchase material	\$0	\$15,000	\$0	\$15,000
Haul/deliver to site	\$15,000	\$8,000	\$9,720	\$23,000
Shape and prep road bed	\$6,100	\$6,100	\$6,100	\$6,100
Spread and compact material (includes blending for boiler ash)	\$12,100	\$7,500	\$12,100	\$7,500
Total cost per lane mile	\$33,200	\$36,600	\$27,920	\$51,600
Cost per lane mile less haul costs	\$18,200	\$28,600	\$18,200	\$28,600
Ash-gravel cost difference	57%		57%	

NOTE: Costs are in 2016 U.S. dollars.

and has mechanical properties resembling those of coarse to fine silty sand as well as some cementitious properties.

Boiler ash was used on two demonstration roads at three different application rates. Engineering laboratory and field tests on the road subgrades and samples of soil and boiler-ash-amended soil indicated that boiler ash can be an effective road stabilizer. The two demonstration roads were monitored for a 1-year period and the road with a poorly graded sugar sand subgrade appeared to benefit significantly from ash stabilization, while the road with a silty sand-loam subgrade performed adequately but did not appear to benefit significantly from the ash treatment.

Road managers must consider both the environmental impact and benefit of road stabilization projects. Environmental concerns for any given road stabilization project should be considered based on applicable regulations. Pulp and paper mill wood boiler ash is generally environmentally benign, meeting regulatory requirements for land application and many regulators encourage beneficial re-use of industrial byproducts such as pulp and paper mill boiler ash. By stabilizing roads with boiler ash, the ash need not be disposed in a landfill and the gravel that otherwise would have been used for aggregate road surfacing need not be mined or hauled.

Although more expensive exotic techniques such as pulverizers and pugmills can produce highly refined, uniform, subgrades of ash-treated or cement-treated soil, the construction methods for road stabilization with boiler ash can be very simple and conventional. The cost of boiler ash road stabilization compares reasonably with more traditional techniques such as gravel. The relative advantages and disadvantages of road stabilization using boiler ash versus gravel or some alternative material such as portland cement depend primarily on the purchase price of the material and the hauling cost. Where boiler ash is available free of charge close to a project site, its cost advantages are apparent.

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STABILIZATION

Mechanical Stabilization of Black Cotton Soil Using Recycled Concrete Aggregates

C. KIRAN

N. K. NABEEL MUHAMED

R. S. JAYA

RASTA-Centre for Road Technology

The improvement of in-situ soil properties to act as subgrade layer is known soil stabilization. After stabilization, strength and durability of the in-situ soil is assessed which qualifies it to be suitable as a subgrade material. Improved sub grade material not only reduces the pavement thickness but also the cost of replacing weak soils with good borrow soil. Black cotton (BC) soil is one among the major soil deposits in Indian subcontinental belt, which has poor engineering properties owing to its organic and clay content.

The objective of this study is to improve the existing BC soil properties using constructional waste material in the form of recycled concrete aggregate (RCA). RCA is processed from construction and demolition (C&D) wastes. According to Indian Road Congress (IRC) guidelines (1) C&D waste makes up about 25% of the municipal solid waste generated annually. This poses a threat to the environment in terms of finding suitable landfills to dispose it. Conventionally naturally occurring graded aggregates are used for mechanically stabilizing the poor soil subgrades. An additional 750 million m³ of aggregates would be required to achieve the targets of the highway sector (1). The Rural Road Manual of India (IRC 20-2002), has specified a brief procedure of mechanically stabilizing weak soil with granular material (aggregates) and thus improving its suitability as a road material (2). The same concept of mechanical stabilization has been used in the present study, with the use of RCA instead of virgin aggregates. Trial ratios of soil:RCA ratio: 1:3, 1:2, and 1:1 have been mixed and tested for improvement in engineering properties. Based on the results obtained, all the three proportions have shown considerable strength improvement to use it as a sub grade material and can be recommended as an effective option for stabilization.

INTRODUCTION

When an unsuitable soil for subgrade layer is encountered in a highway construction project, ideally the contractor has four options:

1. Find a new construction site.
2. Redesign the structure so that it can be constructed in the poor soil.
3. Remove the poor soil and replace it with good soil.
4. Improve the engineering properties of site soil.

In general, options 1 and 2 tend to be impractical today, while in the past; option 3 has been used most commonly. However due to improvement in technology coupled with increased

transportation costs, option 4 is being explored more often and is expected to dramatically increase, probably resulting in an inevitable solution, in the near future.

Improving on-site or in-situ soil's engineering properties is referred to as either "soil modification" or "soil stabilization." The term "modification" implies a minor change in the properties of the soil, while stabilization means that the engineering properties of the soil have been changed enough to allow field construction to take place.

In the case of highway or road construction, subgrade soil is an important component in the pavement structure as it acts as a supporting structure or foundation for the upper pavement layers and helps in taking the load coming from the traffic above the pavement. The subgrade is constructed using superior soils brought from the selected borrow pits for a specified thickness, compacted to a specified dry density.

The cost of road construction using conventional materials has been increasing, year after year by leaps and bounds and with limited finances in developing countries like India; the biggest challenge is to provide a complete network of road system with minimum cost possible. Therefore there is a need to resort to one of the suitable low-cost road construction method by effectively utilizing the locally available materials and adopting soil stabilization techniques.

OBJECTIVES OF THE PRESENT STUDY

1. To characterize the locally available soil taken for the study, i.e., black cotton soil and to ascertain its use as a subgrade material.
2. To characterize the obtained RCA and assess its suitability as a material for blending with native soil, by method of mechanical stabilization.
3. To study the improvements of the basic properties and strength characteristics of the native soil by mechanical stabilization using RCA.
4. To effect economy in the initial construction cost of lower layers of the pavement by using RCA for mechanical stabilization.

MATERIALS USED

Black Cotton Soil

In the present study, BC soil obtained from the project site near Raichur, Karnataka State, India (at two different chainages) has been used (Table 1, Figure 1). The soils were air dried and pulverized in the laboratory. On visual observation the BC soil was seen to form hard lumps which was pulverized to obtain the actual gradation and soil characteristics. The organic matter was found to be 19.50%. The soil was classified as inorganic clays of high plasticity (CH).

Recycled Concrete Aggregates

Conventionally naturally occurring graded aggregates are used for mechanically stabilizing the poor soil subgrades. However, in India projections for construction material requirement by the infrastructure sector indicate a shortage of about 55,000 million m³ aggregates. A large amount

TABLE 1 Gradation of BC Soil

Soil Fractions	Particle Size Distribution of BC Soil
Gravel (%)	1.01
Sand (%)	14.14
Silt and clay (%)	84.85
Soil Fraction	Particle Size Distribution by Hydrometer Analysis
Silt (%)	66.34
Clay (%)	20.8

**FIGURE 1 BC soil sample.**

of C&D waste are generated owing to rapid urbanization in cities. In India the total C&D waste generated just by buildings in 1 year (2013) amounts to a humungous 530 million tones, 44 times higher than the official estimate (*1*). RCA which is used in the present study is processed from C&D wastes, obtained from a local supplier who processes C&D waste into various sizes for different application of the construction industry. The RCA obtained for the present study has been characterized for its basic properties as mentioned in Table 2.

It can be seen that the RCA is slightly falling short of the requirements specified for natural aggregates and can be tried as partial replacements in soil–aggregate mixes.

TABLE 2 Physical Properties of RCA

Sl. No.	Property	Result	Specifications for Natural Aggregates as per MORT&H 5th Revision
1	Specific Gravity	2.44	Maximum 2.5%
2	Water Absorption	2.69%	Maximum 2%
3	Aggregate Impact Value	33%	Maximum 30%
4	Aggregate Crushing Value	30.55%	Maximum 30%
5	Los Angeles Abrasion Value	32%	Maximum 35%

METHODOLOGY

The *Rural Road Manual* (IRC SP: 20-2002) of India, has specified a brief procedure of mechanically stabilizing weak soil with granular material (aggregates) and thus improving its suitability as a road material. Accordingly the process of combining two materials based on sieve analysis as per IRC: SP 20-2002 is done and various ratios of mixing RCA with the native soil for obtaining the desired gradation has been worked out. The proportions of the mix—i.e., native soil: RCA—was obtained to be 1:3 as per the guidelines. In order to check the suitability of decreasing RCA proportion in the mix, two more trial ratios of 1:2 and 1:1 was envisaged. It was ensured that the gradation of the trial ratios was also satisfying the gradation requirements as per IRC SP: 20.

As seen from the gradation curves shown in Figure 2, BC soil showed gradation having maximum fines; but when stabilized with RCA, the gradation seems to have improved drastically, by way of increasing the amount of coarser fraction, which would impact the density and strength criteria.

The basic characteristics of the stabilized soil such as gradation, Atterberg's limits, free swell, specific gravity, maximum dry density and optimum moisture, according the standard specifications, were studied in the laboratory and inferred. The observations are tabulated in Table 3.

The free swell index test was conducted to determine the percentage of swelling of the soil on absorbing water (Figure 3). This is an important test which also indicates the presence of high amount of clay whose swelling potential is high in the presence of water. The native soil without stabilization was found to swell 65% which reduced to 35%, 30%, and 25% for 1:1, 1:2, and 1:3 combinations.

The variation of the dry density with moisture content for the samples compacted with modified proctor compaction for the various combinations are shown in Figure 4.

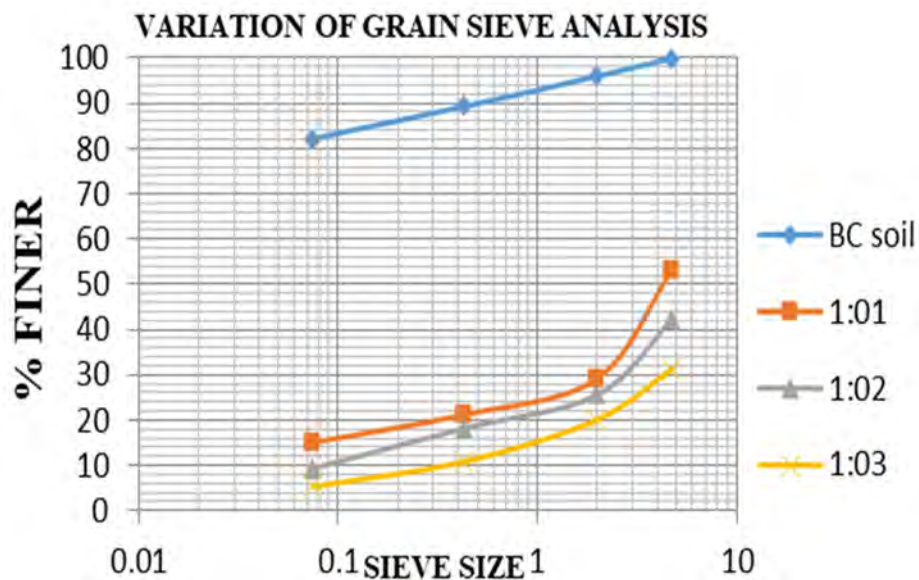


FIGURE 2 Variation of gradation of stabilized and unstabilized soil.

TABLE 3 Abstract of Stabilized Soil Test Results

Sl. No.	Test	Black Cotton Soil Unstabilized	(1:1)	(1:2)	(1:3)	
1	Wet sieve analysis	Gravel	1.01	47.53	60.74	72.38
		Sand	14.14	37.37	29.91	22.26
		Fines	84.85	15.10	9.35	5.36
2	Atterberg's limits	Liquid limit (%)	80.10	48.6	34.50	24.50
		Plastic limit (%)	32.51	34.15	25.85	18.60
		Plasticity index	47.59	14.45	8.65	5.90
3	Free swell index	65	35	30	25	
4	Specific gravity	2.46	2.46	2.44	2.43	
5	Compaction	Max. dry density (g/cc)	1.62	1.80	1.91	1.98
		Opt. moisture content (%)	20.7	13.6	14.0	14.2

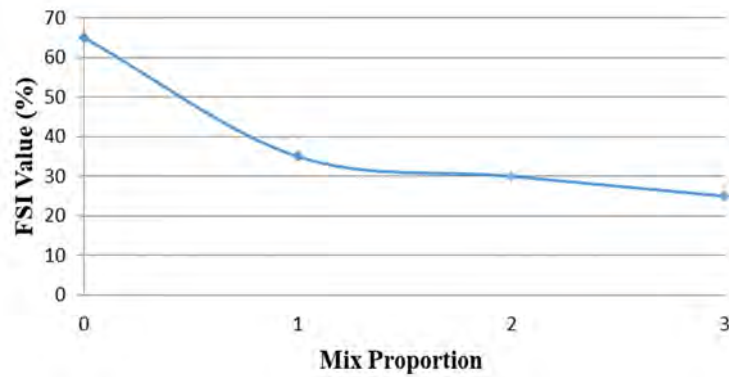


FIGURE 3 Variation of free swell index with RCA proportion.

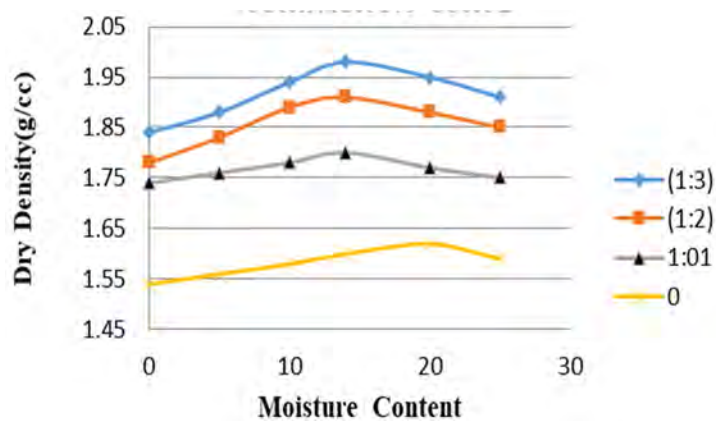


FIGURE 4 Compaction curves for native soil and stabilized soil with RCA.

It can be observed that the maximum density achieved with addition of RCA in native soil in the ratio of 1:1, 1:2, and 1:3 increases with a substantial decrease in OMC as shown in Figure 4.

The strength of the soil was evaluated based on CBR test which is a common test for characterizing strength of soil for pavements and the penetration resistance offered at 2.5-mm penetration was noted as the CBR value. The increase in the CBR value for various combinations is shown in Figure 5.

The CBR value is seen to increase drastically with the use of RCA, which is very encouraging.

The unconfined compressive strength was carried out to find the compressive strength of the soil as well as the shear parameters when a cylindrical specimen is subjected to an axial stress till it fails by shear in which the lateral pressure is zero (Figure 6).

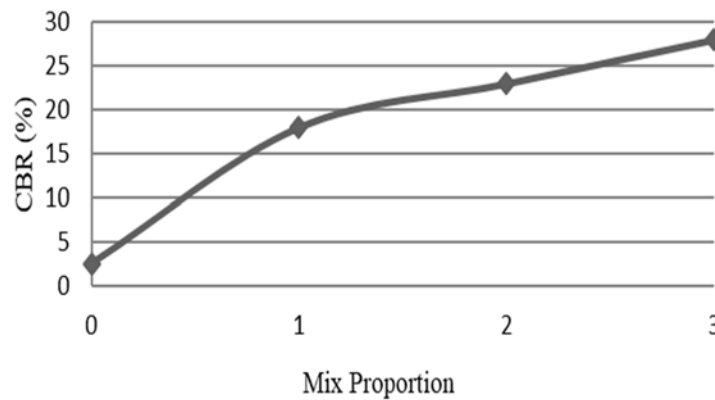


FIGURE 5 Variation of CBR strength with RCA proportion.



FIGURE 6 UCS testing of BC soil sample after testing.

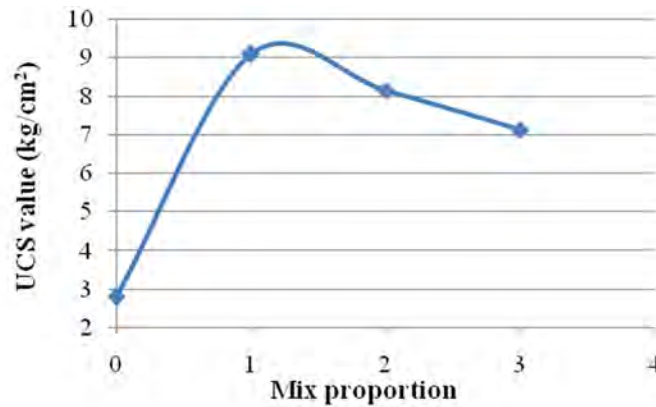


FIGURE 7 UCS curves of RCA stabilized added to normal BC soil.

It can be observed from the results that though the UCS value is showing a decreasing trend with increase in RCA in the mix, its value is substantially higher than the UCS of soil which is unmodified (Figure 7). This needs to be given due consideration.

To understand the behavior of the stabilized soil under repeated load, an attempt was made to subject the stabilized soil to repeated load application by the Fatigue test and observe the number of load repetition cycles it could endure. The test is conducted under the following circumstances.

- i) Stress Level =20%, Frequency =1GHz, Rest Period =0.5 s
- ii) Stress Level =40%, Frequency =1GHz, Rest Period =0.5 s

It is inferred that for both 20% and 40% stress levels, with increase in soil to aggregate proportion, the number of cycles and also deformation is observed to be decreasing.

Durability Test

The durability test was conducted as per IS: 4332 (Part 4) with alternate wetting and drying of soil specimen prepared with stabilized soil. The mould of size 5-cm diameter and 10-cm height is used. The number of cycles taken by the specimen decides the durability of the soil.

Though the durability property does not show encouraging results one should understand that this property is more crucial for chemical stabilization than mechanically stabilized soil. In the present study of RCA used for mechanically stabilized soil shows increase in strength property which gives us an indication of enhancement of engineering properties (Figure 8).

The findings of all the above tests conducted have been tabulated in the Table 4.

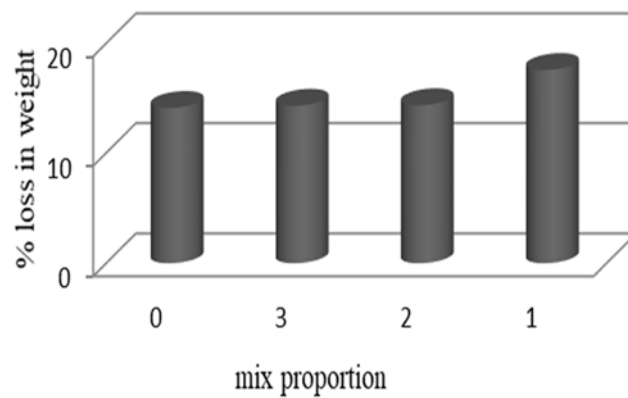


FIGURE 8 Durability curves of RCA stabilizer added to normal BC soil.

TABLE 4 Results of laboratory tests on unmodified and RCA modified soil

Sl. No.	Test	Black Cotton Soil Unstabilized	(1:1)	(1:2)	(1:3)	
1	CBR (%)	2.57	18	23	28	
2	UCS (kg/cm ²)	2.81	9.10	8.16	7.13	
3	Repeated load test (no. of load repetitions taken)	20% stress level	Failed	4,000	3,000	2,500
		40% stress level	Failed	2,000	2,000	1,500
4	Durability (no. of cycles)	1	4	3	2	

COST ANALYSIS

Cost analysis for this present study is done to determine the variations in cost of the pavement structure upon addition of suitable dosage of stabilizer because the cost of the project is the main factor in determining the choice of soil improving method to be adopted. Accordingly design of the pavement is done in order to find the thickness of each layer to be considered, for both stabilized and un-stabilized soils. The minimum thickness of subgrade for rural roads is specified as 300mm. Table 5 gives the details of the pavement design:

- Traffic considered: 10 MSA; CBR for the unstabilized subgrade soil (BC): 6%.
- CBR for the mechanically stabilized subgrade soil (BC) with RCA: 20%.
- Ratio of soil: RCA= 1:1.

TABLE 5 Pavement Design According to IRC SP: 20-2002

Pavement Layers	Pavement Thickness (mm)	
	Unstabilized	Stabilized
Pre-mix chip carpet	20	20
Water bound macadam (WMM)	150	150
Granular subbase	225	30
Subgrade	300	300

Calculation of Mechanical Stabilizer Quantity

$$\begin{aligned}
 \text{MDD of the BC soil} &= 1.80 \text{ g/cc} &= & 1.80 \times 10^6 \text{ g/cum} \\
 \text{Volume of the sub-grade soil} &= &= & \text{length} \times \text{width} \times \text{depth} \\
 &= &= & 1000 \times 3.75 \times 0.3 \\
 &= &= & 1125 \text{ cum} \\
 \text{Weight of the subgrade soil} &= &= & 1125 \times 1.80 \times 10^6 \\
 &= &= & 2025000 \text{ kg} \\
 \text{Mechanical stabilizer ratio} &= &= & 50\% \\
 \text{Mechanical stabilizer quantity} &= &= & 0.5 \times 2025000 \\
 &= &= & 1012500 \text{ Kg}
 \end{aligned}$$

Cost Calculations for Conventional Pavement

The schedule of rates for the calculations is referred from Schedule of Rates (11).

TABLE 6 Cost Calculations of Conventional Pavement

Particulars	Rates (Rs./cum)	Length (m)	Width(m)	Depth(m)	Quantity (cum)	Amount (Rs)
Excavation and removal	157	1,000	3.75	0.3	1,125	176,625
Embankment	331	1,000	3.75	0.3	1,125	372,375
Subgrade	331	1,000	3.75	0.3	1,125	372,375
Granular subbase	1,408	1,000	3.75	0.225	844	1,188,352
Water bound macadam	1,638	1,000	3.75	0.150	563	922,194
Prime coat	26	1,000	3.75	—	3,750	97,500
Tack coat	18	1,000	3.75	—	3,750	67,500
Pre-mix chip carpet	114/m ²	1,000	3.75	—	3,750	427,500
Seal coat	36/m ²	1,000	3.75	—	3,750	135,000
Total amount, INR						3,759,421

TABLE 7 Cost Calculations for Pavement with Stabilized Subgrade

Particulars	Rates (Rs./cum)	Length (m)	Width (m)	Depth (m)	Quantity (cum)	Amount (Rs)
Mechanical Stabilizer cost for 1012500 kg at the rate of 0.50 Rs/Kg						506,250
Excavation	68	1,000	3.75	0.3	1125	76,500
Embankment	150	1,000	3.75	0.3	1125	168,750
Sub-grade	150	1,000	3.75	0.3	1125	168,750
Granular Sub-base	1,408	1,000	3.75	0.03	112	157,696
Water Bound Macadam	1,638	1,000	3.75	0.15	563	922,194
Prime coat	26	1,000	3.75	—	3750	97,500
Tack coat	18	1,000	3.75	—	3750	67,500
Pre-Mix Chip Carpet	114/m ²	1,000	3.75	—	3750	427,500
Seal coat	36/m ²	1,000	3.75	—	3750	135,000
Total amount, INR						2,727,640

From the above tables (Table 6 and Table 7), it is evident that the cost of 1 km of road with conventional materials along with BC as subgrade soil is found to be INR 37,59,421/- where as the cost of 1-km pavement with stabilized BC soil as subgrade material is found to be INR 27,27,640/-. The overall savings in the cost after adapting soil stabilization is found to be INR 10,31,000/- and similarly the mechanically stabilized at different ratio like (1:2) and (1:3) cost of 1-km pavement with stabilized with different proportion BC soil as subgrade material is found to be INR 29,41,221/- (1:3) for INR 30,56,702/-. The overall savings in the cost after adapting different proportion of soil stabilization is found to be for (1:2) is INR 8,18,000/- and (1:3) for INR 7,02,000/-. Hence the soil stabilization using mechanical soil stabilizer is found to be economically viable and should be preferred to replacing the existing weak soil by new borrow soil.

DISCUSSIONS

1. As seen from the gradation curves in Figure 2, BC soil showed gradation having maximum fines; but when stabilized with RCA, the gradation seems to have improved drastically, by way of increasing the amount of coarser fraction, which would impact the density and strength criteria.

2. On observing the results of free swell index in Figure 3, it is understood that the addition of RCA contributes to reduction of FSI, i.e., from 65% to an average of 30% for BC soil. In the case of BC soil, the FSI has reduced below 50% which is the acceptable limit according to MORTH specifications. The liquid limit and plasticity index of soil having RCA in varying proportions, has reduced after the addition of RCA imparting mechanical stabilization to the normal soil. This indicates that the RCA stabilized soil reduces the water-holding capacity of the soil due to the increase in coarser fraction which results in lesser PI values.

3. It can be observed from the compaction curves plotted in Figure 4, that the maximum density achieved with addition of RCA in native soil in the ratio of 1:1, 1:2, and 1:3 increases with a substantial decrease in OMC. The MDD of BC soil has increased from a value of 1.62 g/cc for unstabilized soil to a value of 1.98 g/cc. The OMC of normal BC soil is 20.70% whereas the OMC's of RCA stabilized added BC soil samples are 13.6% ,14.00%, 14.20% at(1:1),(1:2) and (1:3) ratio respectively. There is a large variation in between OMC's of normal and RCA stabilized soils.

4. As seen from the Figure 5, the CBR of normal BC soil is 2.57% and the CBR of the RCA-stabilized soil has increased substantially to 28% for mechanically stabilized BC soil. This would result in reduction in the pavement thickness and therefore proves to be economical. Also from Figure 7, it can be seen that there is an increase in the UCS of stabilized soil compared to that of native BC soil. However a decrease in UCS value, with the increase in RCA was observed, which may be attributed to the lack of cohesion between soil particles and hence decrease in shear strength. This may be enhanced by using additives such as lime which provides for future scope of studies.

5. Though the durability property does not show encouraging results, as shown in Figure 8, one should consider that this property is more crucial for chemical stabilization than mechanically stabilized soil. In the present study of RCA used for mechanical stabilized soil shows increase in strength property which gives us an indication of enhancement of engineering properties.

6. For both 20% and 40% stress levels, mechanically stabilized BC soil has endured 4,000 cycles of repeated load; when compared to native BC soil, which failed instantaneously.

CONCLUSIONS

- The mechanically stabilized BC soil using RCA aggregate has shown improvement in strength properties and hence recommended for use as alternative to “removal and replacement” with borrow soil. Also according to the present study it has been found that 1:1 (ratio of soil: RCA) has present to be an effective and economical option for mechanical soil stabilization.
- An average savings of 22.63% per km is achieved by adding RCA to BC soil in the ratio 1:1 (soil: RCA); which is substantial.
- In the present alarming scenario of (a) over use of naturally occurring aggregate and also (b) requirement of huge land fill sites to dump C&D waste, the present study suggests a sustainable solution to tackle both the above problems and hence may be adopted as a promising alternative especially for rural–low-volume road construction.

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STABILIZATION

Rapid Rehabilitation of Highly Distressed Low-Volume Roads in Papua, New Guinea, Using In Situ Soil Recycling with Hydraulic Soil Stabilization

ALEX CAMPBELL

AEC Transportation Consulting Inc.

INTRODUCTION

Papua New Guinea (PNG) is a culturally diverse country that lies north of Australia. It is also one of the most rural countries in the world with only 15% of inhabitants living in an urban center (World Bank, 2018). The country, consisting of the eastern half of the island New Guinea and several offshore islands, lacks interconnectivity, especially between the mountainous Highlands Region and the capital city of Port Moresby where there is no road link. The lack of good-quality roads and connectivity between regions are major obstacles to economic development (Gibson and Rozelle, 2003). The majority of roads throughout PNG are low-volume roads with the condition ranging from poor to very poor. Highly distressed roads are commonplace. Poor drainage, inadequate design, lack of maintenance, and poor road-building materials are typical. However, rehabilitation of the existing fragmented road network is a national objective and an aggressive platform has been adopted (Faiz, 2012).

Highly weathered tropical soils are commonplace throughout PNG, which typically have high plasticity and low load-bearing capacity. High rainfall and poor drainage conditions also makes road construction and rehabilitation challenging. A typical pavement design or rehabilitation intervention would assess the subgrade soil conditions and, in most cases, remove the weak soil and replace it with a suitable fill or subbase material. Yet, good-quality accessible road-building material is uncommon in remote areas and often requires lengthy transportation. The lack of quality road-building materials and presence of plastic subgrades, makes road building and rehabilitation interventions expensive and slow.

Soil stabilization of marginal materials (in-situ and imported) using hydraulic products presents an ideal process to convert limited use materials (i.e., plastic subgrades, marginal subbase, and base materials) into useable pavement materials. In the case of PNG, to be able to use the subgrade soil and upgrade its engineering properties presents a valuable tool to decrease costs, decrease environmental impacts, and accelerate construction processes to meet the national objective of improved interconnectivity and quality of the low-volume road network.

To illustrate the ability of hydraulic soil stabilization to accelerate project timelines and create a sound technical solution, a case study is presented. A pavement rehabilitation project in PNG highlights how using hydraulic soil stabilization can be used to create a new bound base course in a project with significant pavement material quality issues. The successful project highlights the technical advantages of the solution when compared to the traditional pavement approach of exchanging materials. Since the successful implementation of the rehabilitation project using soil stabilization, soil stabilization is now an integral practice adopted in PNG by the Department of Works for construction of new and rehabilitation of distressed pavements.

CASE STUDY: WABAG TO WAPENAMANDA ROAD

The Wabag to Wapenamanda Road (W-W) is located in the Northern Highlands region in Enga Province, PNG. The 30-km road was upgraded between 1995 and 2001 to a double chip seal pavement, however signs of distress and premature failure were observed within months of completion. Observed distresses included stripping of the chip seal and potholes (Figure 1).

Despite the fact that the W-W road showed earlier signs of distress and failure, little had been done to rectify the problem. Spot maintenance in the form of pothole repair had been undertaken and impassable sections had been remedied by grading. There was an urgent need to rehabilitate the road to bring it to a functional classification, as it was impeding economic growth due to the poor riding condition. The objective of the project was to undertake a site and soil survey to assess the extent by which a solution incorporating AnyWay's blended cementitious soil stabilizer (cement, slag, lime, and rate governing additives) could produce the desired outcome, and rehabilitate the road using full-depth recycling.



FIGURE 1 Distresses observed during the pavement investigation included stripping of the wearing course, edge-break, and potholing in both the sealed and exposed pavement.

The W-W road had a very low level of serviceability due to failure throughout the entire length. The following were found to be the principal issues associated with the road:

- **Variability of material.** The variability of material over the project was substantial. Alluvial gravel having poor gradation as a base material was used. The presence of rounded aggregates is problematic for load-bearing capacity and resistance to repetitive loading. The use of different materials outlines a complete lack of quality control at the borrow pit.
- **Variability in design.** The prescribed design was a double chip seal over a 150-mm base course over a 200-mm subbase material. Yet, over the test pit locations there was no single layer that met the base requirement for thickness. Further compounding the inappropriate thickness and material of the base layer was the variance in the subbase layer. The subbase not only had variability in material type and thickness, but in some instances, was not present at all.
- **Drainage.** The drainage over the course of the road was very poor with the least serviceable sections of the road being those that had poor drainage (Figure 2). Where drainage was well established, it was rendered ineffective due to debris either impeding drainage within the culvert or channel and/or material blocking access to the drainage system. In some cases, the camber of the road was sloped away from the drainage channel.
- **Poor seal quality.** The chip seal lacked specification in terms of the nature of the chips used. The chips originated from an alluvial source and did not have any formal processing. As a result, the chips were rounded.

From the site survey and test pits, it was concluded that there was no single mechanism of failure. Rather, compounding factors contributed to the rapid decrease in pavement performance. The following principle factors were identified:

- Alluvial gravel should not be used as a base material without crushing the material to give the material some angularity. The smooth characteristics of the alluvial aggregates are not



FIGURE 2 Distresses were more prevalent where poor drainage was encountered. Vegetated growth was the principal contributor to poor drainage.

desirable as they do not have the ability to withstand repetitive loading. Compounding the poor base material selection was a complete lack of adherence to required thicknesses of both base and subbase layers. Varying thicknesses of base or subbase layers significantly compromised the pavements performance;

- Completely different materials were found to be present in both the base and subbase layers, again contributing to the decreased ability of the pavement to resist the required loading on the road; and,
- The alluvial aggregate used for the base was the wrong material to have been used. The layer thickness variability coupled with the choice of aggregate significantly compromised the ability of that layer to resist the effects of traffic. Further complicating the issue of failure was the variability in base and subbase material. Therefore, the type of material used in the base cannot be the sole contributing factor to premature failure. It was an inappropriate material, but if it has been constructed to the correct thicknesses (and likely densities) it should have not shown early distresses and failure. It was the quality of materials, variability of materials and variability in thicknesses that was the pavement undoing and cause of premature failure.

The inability of the base to withstand repetitive loading needed to be addressed. Geotechnical reports compiled by the Department of Works and dynamic cone penetrometer (DCP) results showed that the California bearing ratio (CBR) strength of the base was not only variable, but also that it was below the required CBR requirement of 80. Therefore, in an effort to rehabilitate the pavement to a serviceable level and to minimize the cost, improvement of the upper layer of the road needed to be addressed. This was achieved through cementitious stabilization.

Using stabilization can increase load-bearing capacity of the pavement layer to a level that exceeds the CBR requirement of 80. Given the broad range in material quality, a stabilizer was needed that could achieve a minimum strength. Through laboratory trials, a blended soil stabilizer consisting of cement, lime, pozzolans and rate governing additives was chosen. The rounded nature of the aggregates used in the base also needed to be addressed using the rehabilitation intervention (Figure 3). As such, a recycler was employed in the stabilization construction process (the first in PNG). The soil recycler allowed for milling of the upper pavement layers to break up the rounded aggregate and provide angularity. In total, two (2) passes with the recycler was made; one do process the road and another to further process the stabilized base to OMC requirements. An added benefit to the recycler is that the depth of mixing and moisture content can be accurately controlled, which produces a uniform rehabilitated base layer along the project.

The design that was applied to the rehabilitation project was to mill to a depth of 175 mm to ensure that a significant thickness of stabilized base material was provided to compensate for any variability in subbase strength. The recycled base course was stabilized with 3% stabilizer at OMC and uniform depth over the entire project length. The road was sealed with a double chip seal with a 1 9/13-mm stone chip size.

FINDINGS

To evaluate the pavements performance since rehabilitation (2012), visual pavement evaluations and assessments have been employed along with nondestructive testing (DCP) at biannual intervals.



FIGURE 3 A soil recycler was used to blend the various materials and layer thicknesses, to help break down some of the rounded nature of the alluvial stones that had been used in the original pavement design, as well as to achieve a homogenous mix, stabilizer distribution, and optimum moisture content (OMC).

The pavement was found to have very consistent DCP through the stabilized layer. This consistent penetration is indicative of consistent strength of the stabilized layer. CBR strengths of the layer using the conversion coefficient produced by Kleyn et al. (1983) resulted in higher CBR results when compared to the coefficient developed by Chen et al. (2001). The average CBR of the chip seal section was 139 and was indicative of a bound base-course material.

The completed 30-km pavement has been in service for 6 years and there has been no major maintenance required. The issues associated with the failed road (i.e., potholing, stripping of the wearing course, rutting) have been eradicated through the combination of full-depth recycling and the use of appropriate soil stabilization technology.

CONCLUSION

The quality and extent of pavement infrastructure in PNG is variable. Not only does the rural road network require investment and upgrading, especially to reach the Millennium Development Goals, but the sealed road network needs upgrading and rehabilitation to keep the economy of PNG moving forward. However, variable pavement designs, pavement layer thicknesses and pavement material in existing roads have been shown to have variability over the length of a single projects. As such, a methodology is required that can upgrade these variable materials to create enhanced structural pavements without compromising the structural integrity of the road, without impacting the environment through accessing borrow pits for virgin material, and using existing budgets.

Through the use of a case study of an in-service project, a full-depth recycling and stabilizing method has been shown to provide excellent long-term results. By recycling the existing distressed pavement, with an appropriate hydraulic stabilizer, the pavement has been shown to provide excellent DCP strength. Through the use of a soil recycler, the depth and

mixing conditions can be controlled, thereby further improving the quality and pavement longevity of the applied solution.

Pavement recycling using soil stabilization is now being used to upgrade highly distressed pavements in PNG as a result of the successes achieved in the W-W Project. Soil stabilization is being used to enhance rural and urban mobility.

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STABILIZATION

Cement-Stabilized Fly Ash for Application in Structural Layers of Low-Volume Road Pavements

ANUSREE BHOWMIK

Larsen and Toubro, TIIC

BRUNDABAN BERIHA

UMESH CHANDRA SAHOO

IIT Bhubaneswar

Fly ash, with an estimated production of about 200 million tons per year in India and maximum usage of 60% is demanding large disposal areas and also posing a potential threat to water pollution by leaching of heavy metals present in it. In the present study, an attempt has been made to use fly ash (FA) and stone dust (SD) mixture (70:30 ratio) and stabilized with cement, as a structural layer material for use in rural road pavements. To address the brittleness of the stabilized material, polypropylene fibers were added as reinforcing material. Mechanical properties of the stabilized material, in terms of unconfined compressive strength (UCS), flexural strength (FS), flexural modulus (FM), and Poisson's ratio were evaluated to assess the suitability of the material for use in pavement layers. Fatigue characteristic of the stabilized material was also evaluated as part of this study. Durability tests indicated adequate retained strength of the stabilized mixture. The scope for contamination of groundwater was evaluated through leaching of heavy metals from stabilized FA. The encouraging results found from this study indicate that, cement-stabilized FA–SD mix can be suitably used for the construction of subbase and base layers of rural road pavements.

INTRODUCTION

In India, thermal power plants are generating 200 million tons of FA every year, out of which only 60% is used for gainful purposes. It can be seen from Figure 1 that the bulk usage is mainly confined to the cement industry in producing portland pozzolana cement. The second big sector is the road and embankment, where FA is used as embankment fill material and improvement of the weak soil subgrades. The rest unused FA is dumped on wastelands and leachates generated from it is an environmental concern. Therefore attention is needed to explore suitable methods for bulk usage of the FA.

A huge length of rural roads are being constructed in India under the most ambitious rural connectivity program called Pradhan Mantri Gram Sadak Yojana, but sustainable growth of this development has been affected by the scarcity of good construction materials near to project locations. The pozzolanic property of FA makes it a very good material for fill and embankment—particularly at subgrades which are prone to settlement—and as a soil stabilizer, but studies on its use in bulk quantity for construction of structural layers (i.e., subbase and base) are very limited. Therefore considering these two issues, the potential use of FA in bulk quantity for construction of structural layers for rural road pavements need to be explored.

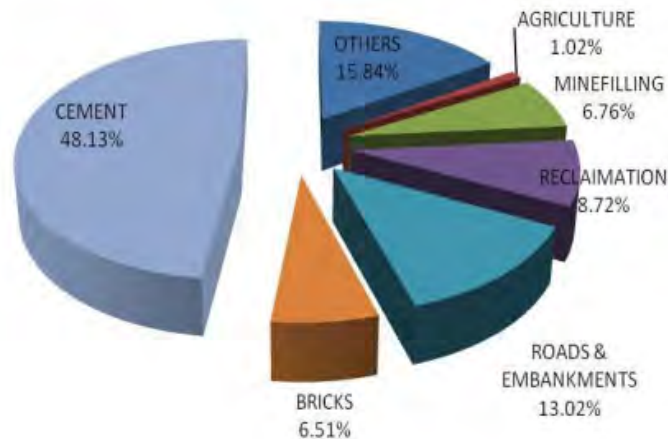


FIGURE 1 Usage scenario of FA in India (1).

Laboratory studies conducted by different researchers (2–4) suggest that stabilized FA has great potential for use in the base and subbase layers of a pavement. Predominantly, cement (2, 4) and lime (3) are used as a stabilizer for improving the strength and stiffness of FA. As aggregate is not used in the mixture, relatively high stabilizer quantity is required to achieve the desired strength for application in the road pavements (2). Study even shows that the addition of a small quantity of gypsum to lime stabilized FA significantly enhances the shear strength and cohesion of the mixture (3). Addition of stabilizer increases the strength and stiffness of FA, but it makes it brittle. Therefore researchers add fibers to the mixture that changes its behavior from brittle to ductile (5, 6).

One of the concern in using industrial wastes like FA in the road construction is the leachate generated from it, which could contaminate the surrounding water bodies. But studies (7–9) have shown that stabilization of FA reduces the amount of heavy metals in the leachate.

As the behavior of the stabilized FA, when used as a structural layer material such as base course is very little known, the present study was taken up to assess the engineering properties of stabilized FA and study its performance under the traffic loading. SD or aggregates may be added to the FA in suitable proportion to improve the density of the mixture that increases the strength. Further, modification with polypropylene fibers can be done to improve the brittle nature of the stabilized material. Therefore the present study aims at determination of optimum proportion of FA and SD in the composite mixture (keeping amount of FA on the higher side) and fixing the dosage of cement and fibers in the mixture, making the cement-stabilized FA suitable for pavement structural layers.

MATERIALS

Fly Ash

The FA used in this study was obtained from a local source and can be classified as class F FA according to ASTM C 618 (10). Some of the physical properties and chemical composition of

the FA are listed in Table 1. SD was obtained from a local stone crushing unit with a maximum particle size of 4.75 mm with a coefficient of curvature of 0.98 and coefficient of uniformity of 5.85. As the coefficient of uniformity is less than 6, it can be designated as poorly graded sand (SP). Cement of grade OPC 43 (Ordinary Portland Cement) conforming to Indian standard practice IS 8112 (11) was used in the present study. The grade is based on the 28-day compressive strength of the cement mortar which is not less than 43 MPa. The cement can be classified as type Type I portland cement as per the ASTM C 150 (12). CETEX construction polypropylene fibers (100% virgin) were used for this study. The fibers are 100% alkali resistance and have high acid and salt resistance. It has a specific gravity of 0.91 and melting point of 162°C. Two lengths of fiber, i.e., 12 and 24 mm were used, each having 35 μ (approx.) diameter.

Sample Preparation

Prior to the preparation of specimen, FA and SD sample was dried in an oven for 24 h with a temperature of over 100°C. The oven dried FA and dust was mixed thoroughly to obtain a uniform mixture. Then the desired quantity of cement and water was added to the mixture and mixed thoroughly. For specimen with fiber, mixing was done through the high shear mechanical mixture as hand mixing was not adequate in dispersing the fiber throughout the mixture.

Two series of samples were prepared for this study. In the first series, three FA and SD ratio ($P_{FA}:P_{SD} = 80:20, 70:30, \text{ and } 60:40$) were evaluated at three cement percentages (2%, 4%, and 6% by total weight of the mix) to achieve target 7-day unconfined compressive strength (UCS) of 3 MPa laid down by Indian Road Congress (IRC) (13) for rural road application. In the second series, two different length of fiber (i.e., 12 and 24 mm) was added to the design mix (FA, SD, and cement) at 0.5% of the total mix to study the effect of fiber. The stabilized mixture is designated as xFA-ySD-zPP where x:y is the ratio of FA and SD. The z corresponds to the length of the fiber used in the mixture. For specimen without fiber, the value of z is 0.

TABLE 1 Physical and Chemical Properties of the FA

Physical Properties	
Properties	% Fraction by Total Weight
Specific gravity	2.012
Gravel-sized fraction	0
Sand-sized fraction	27
Silt-sized fraction	70
Clay-sized fraction	3
Chemical Composition	
SiO ₂	28.13 %
Al ₂ O ₃	11 %
TiO ₂	1.63 %
Fe ₂ O ₃	4.10 %
MnO	0.2 %
MgO	0.1 %
CaO	0.6 %
Na ₂ O	0.1 %

For UCS test, cylindrical samples (50 mm diameter, 100 mm length) were prepared whereas beam specimens of size 75 mm x 75 mm x 285 mm were made for flexure-based test. The specimens were compacted with appropriate compactive effort to achieve maximum dry density (MDD). Compacted specimens were removed from the mold and wrapped in polythene before curing. The curing was done at a temperature of 27°C and 97% relative humidity.

EXPERIMENTAL INVESTIGATIONS

The following procedures are been followed for experiments are detailed below.

Unconfined Compressive Strength Test and Indirect Tensile Strength Test

The UCS test was conducted according to ASTM D 1633 (14). Cylindrical specimens were loaded at a strain rate of 1.25 mm/min until failure and the maximum stress at failure was reported as the UCS value. In the indirect tensile strength (ITS) test, cylindrical specimens having 100 mm diameter and 60 mm thickness were loaded across its vertical diametric plane at a load rate of 1 mm/min till failure. The ITS value (f_{ct}) was determined using the failure load data and specimen geometry.

Poisson's Ratio

In the study, Poisson's ratio has been determined by indirect diametric tensile testing procedure suggested by Paul and Gnanendran (15). Cyclic haversine loading at 30% of failure load of IDT on frequency of 1 Hz has been used. The specimen was conditioned for the first 1,000 cycles to eliminate possible permanent deformation accumulation (16). A thin iron strip of width 12 mm was used for uniform distribution of load over the entire depth of the sample. Two linear variable differential transformers (LVDTs) were placed on the surface of the specimen along the horizontal and vertical diameter to measure the deformation of horizontal and vertical diameters. Poisson's ratio and dynamic stiffness modulus (E_{IDT}) was determined using Equation 1 and 2, respectively.

$$\mu = \frac{-c_g - a_g \frac{\delta v}{\delta h}}{d_g + b_g \frac{\delta v}{\delta h}} \quad (1)$$

$$E_{IDT} = \frac{P}{t \delta h} (a_g + b_g \mu) \quad (2)$$

where

- P = the cyclic load applied in N which is equivalent to 30% of failure load at ITS;
- t = the thickness of the sample (60 mm);
- μ = the Poisson's ratio;
- E_{IDT} = the dynamic stiffness modulus from ITS testing (MPa);
- δv and δh = the deformation along vertical and horizontal diameter, respectively (mm); and

a_g , b_g , c_g , and d_g = constants which are dependent on gauge length (distance between gauge points at which LVDT are placed) to diameter ratio of sample.

In the test condition, this ratio was 1, and therefore constant values were chosen accordingly.

Flexural Strength Test and Flexural Modulus Test

FS is defined as the maximum flexural stress generated at the outermost fiber of the specimen at the time of failure. The FS of all the beam specimens were determined as per ASTM D 1635 (17). The loading arrangement is shown in Figure 2.

As there is no standard code for the determination of FM, the protocol suggested by Mandal et al. (18) has been modified for this study. The testing setup used for the FS test was also used for this test. Cyclic haversine load pulses of frequency 1 Hz was applied to the specimen with no rest period. Each load pulse consists of 0.5 s loading and 0.5 s unloading. The load magnitude in each cycle was 30% of FS. A total of 100 load cycles were applied where first 50 cycles used for conditioning. The mid-span deflection was measured with a LVDT while applying load. The load and deflection data from the last 50 cycles were used for calculating FM. The average modulus value of last 50 cycles was considered as the FM value of the specimen.

Flexural Fatigue Test

Flexural fatigue test was conducted on specimens just after the FM test, as the FM test is a nondestructive test. The same setup used for the FM test was used for this test. A modified version of the test protocol suggested by Mandal et al. (18) was used for the test. The specimens were subjected to haversine load pulses of frequency 2 Hz without any rest period. Each load pulses was having 0.25-s loading and 0.25-s unloading. A contact load of 10% of the desired load

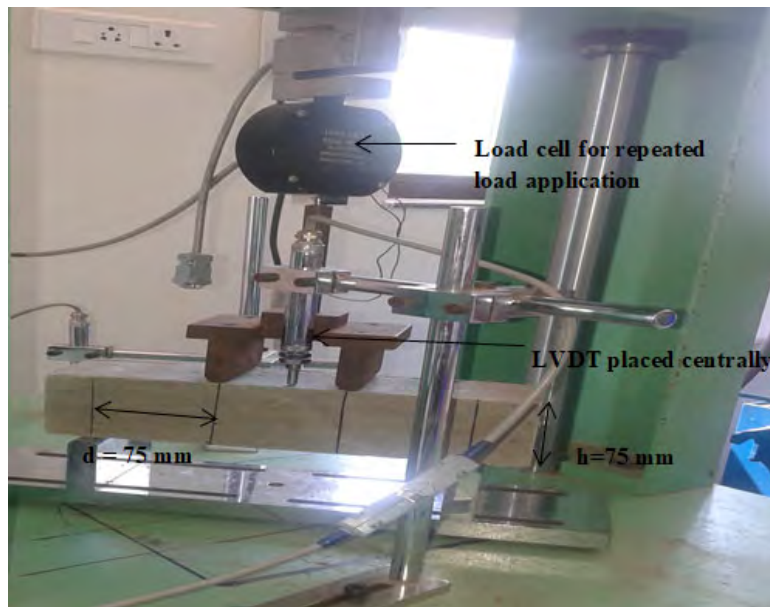


FIGURE 2 Testing setup for FS and FM.

was applied to the specimen to avoid rocking motion. The load magnitude was varied from a range of 95% to 60% of FS.

The fatigue life of a specimen is defined as the load cycle needed to reduce the specimen modulus to 50% of the initial modulus. The initial modulus is the average modulus of the first 50 cycles of fatigue test

Durability Test

As there is no test method available to assess the durability of stabilized FA, test method suggested by Mumtaz et al. (19) was adopted in this study. As per this method, 7-day and 28-days-cured specimens were subjected to 10 cycles of wetting and drying (W-D) followed by UCS testing for determination of residual UCS. Each W-D cycle consisted of immersing the specimens inside water for 5 h at 24°C and then drying it in the oven for 42 h at 72°C. To simulate a more destructive effect (20) the samples were immersed in water at room temperature for 2 h before residual UCS testing. No brushing was involved in this test as it is not considered appropriate for finer materials like FA.

Leaching Studies

For this study, ASTM water extraction procedure was followed to check the safety of using FA as a base layer in roads. In this procedure, 7-day and 28-day samples were air dried and ground to pass through 2-mm sieve and were stored in a plastic bottle for analysis. About 25 gm of FA sample was mixed with 100 ml distilled water and then kept in a shaker platform at 70 1-in. (25-mm) strokes per minute agitation. After the desired period of agitation, the extract was stirred with a glass rod and filtered through 0.45- μm filter paper. The solution was acidified with HNO_3 (1.5 ml per 100 ml) such that pH of the solution is less than 2 and was kept at 10°C for preservation. After the generation of this leachate, heavy metal detection was done through atomic absorption spectrophotometer. The test was done for six toxic heavy metals—mercury, lead, chromium, cadmium, iron, and zinc. The test for arsenic which is commonly found in FA could not be performed due to some resource constraints. U.S. Environmental Protection Agency guidelines of the permissible limits for heavy metal in primary drinking water quality was taken as standards for allowable limits in this work (21). Threshold values for the maximum concentration of trace elements in drinking water was taken as 100 times that of the allowable limit.

RESULTS AND DISCUSSION

Grain Size Distribution of Fly Ash, Stone Dust, and Composite Mixtures

The grain size distribution has a strong influence on the density, the construction material can achieve. The grain size distribution curves for the FA and SD and composite mixtures are shown in Figure 3. From the grain size distribution curve, it can be noted that 70FA-30SD-0PP have a well-graded particle size distribution.

Moisture Density Relationship of Stabilized Fly Ash

Figure 4 shows the compaction curve with and without fiber for different FA and SD composition. For different composition of FA and SD, it was observed that MDD decreased from 1,631.5 kg/m³ to 1,468.4 kg/m³ with the increase in the proportion of FA whereas optimum moisture content (OMC) increased from 13% to 18%.

The inclusion of fibers by 0.5% showed a little impact on OMC of the mixes, as the fibers are non-water absorbent. Inclusion of 12-mm fibers made the material cohesive as MDD value was found to increase whereas for 24-mm fibers, due to balling of fibers, the material became stiff, resulting in less compaction. However, no significant effect of fiber inclusion on the values of OMC and MDD was noted.

Mechanical Strength Parameters

It can be seen from the test results presented in Table 2 that UCS value increases with an increase in cement percentage for all three compositions of FA and SD. But the composition of 70% FA and 30% SD (70FA:30SD) resulted in optimum UCS value with maximum utilization of FA, compared to the other two compositions. This may be because of better particle packing and formation of an adequate amount of calcium silicate hydrate gel.

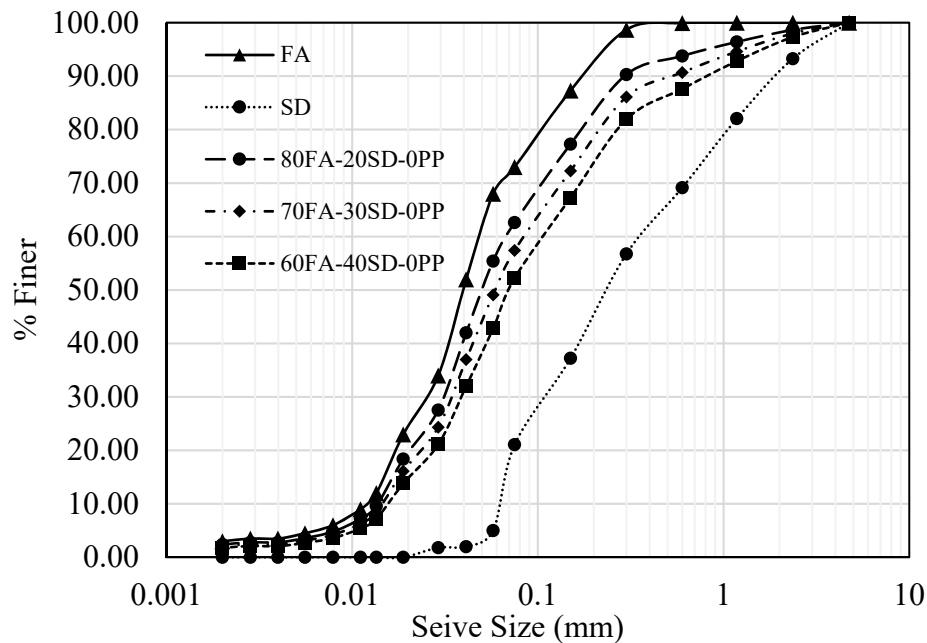


FIGURE 3 Grain size distribution curves of FA, SD, and FA-SD composites.

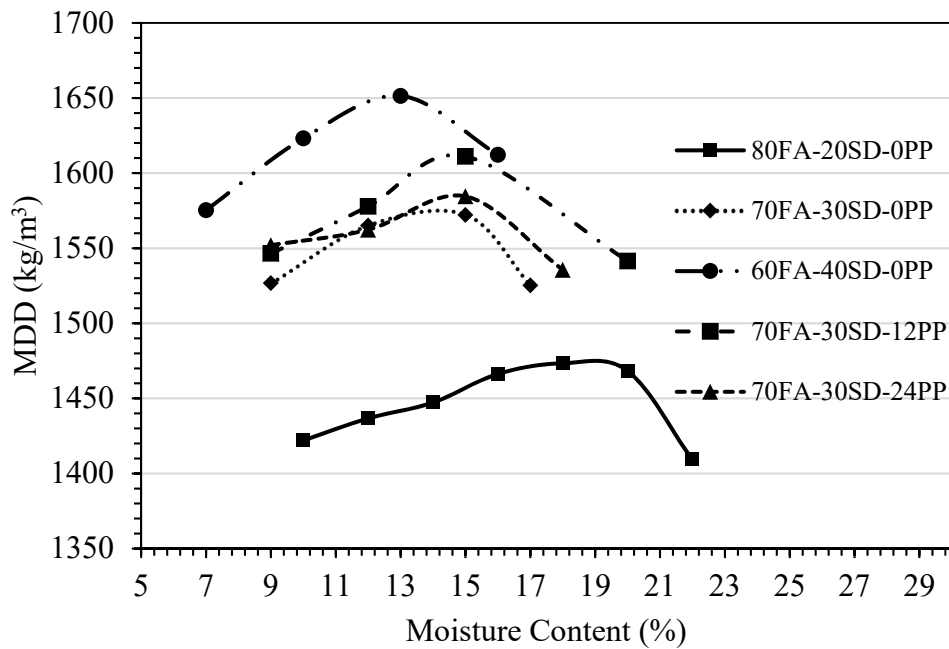


FIGURE 4 OMC versus MDD curve of different FA–SD composites.

TABLE 2 Seven-Day UCS Values for Different Compositions of FA and SD

% of Cement	7-Day UCS Value of FA–SD Composition (MPa)		
	80:20	70:30	60:40
2	0.6	1.09	0.9
4	1.5	1.87	1.8
6	2	3.2	2.8

From the observation of failure patterns of UCS samples without and with fibers, it was confirmed that the brittleness of the composite material can be significantly reduced by inclusion of polypropylene fibers. Considering the advantages of 12-mm polypropylene fibers, all, further investigations were done with 70 FA – 30 SD – 12 PP composition. The UCS values after 7, 28, days and 56 days for this composition were determined as presented in Figure 5. It may be observed that the strength increased with the curing period and at 56 days, the strength reached as high as 4.93 MPa. Kumar and Singh (6) had reported a significant gain in UCS value after the inclusion of randomly oriented fibers in FA soil mixture, which could not be seen in this study.

FS values obtained from testing of beam samples with 70FA-30SD-12PP composition at different cement percentages are presented in Table 3. It may be observed from that increase in the cement percentage and curing period has a positive effect on FS value.

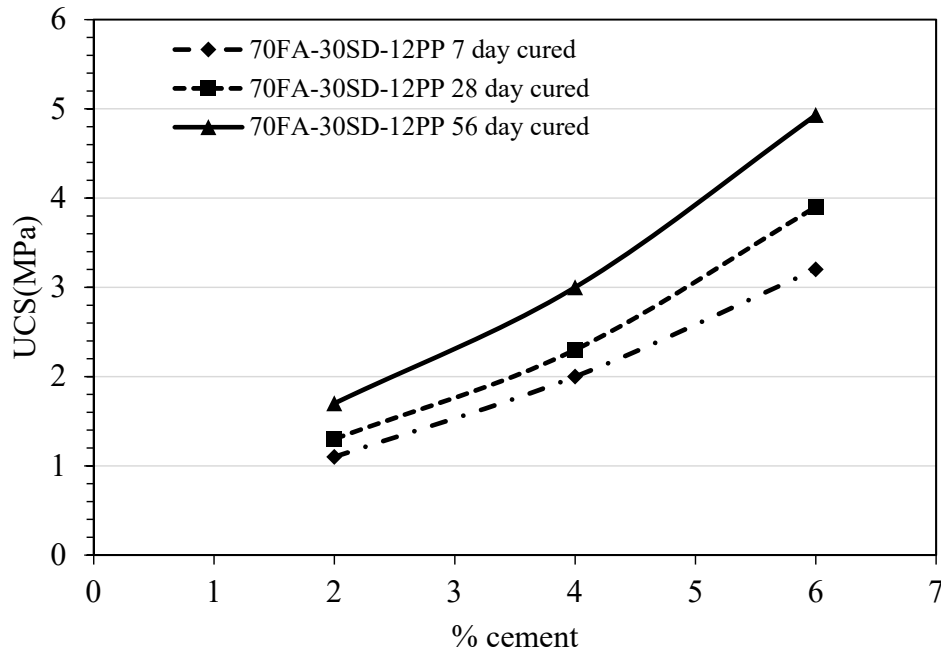


FIGURE 5 UCS of 70 FA-30 SD-12 polypropylene composition at different curing period.

TABLE 3 FS and ITS values at different binder content

% of Cement	ITS (MPa)		FS (MPa)		FM at 28 Days (MPa)	Dynamic Stiffness Modulus from IDT Test at 28 Days(MPa)
	28 Days Cured	45 Days Cured	28 Days Cured	45 Days Cured		
2	0.162	0.178	0.84	0.85	3595	1755
4	0.304	0.408	1.944	2.18	5935	3291
6	0.566	0.951	2.38	3.3	7729	6118

According to Vutukuri et al. (22), IDT test is only valid when the primary failure occurs along the loading diameter. Therefore only those results were considered, which had the primary fracture along the loading diameter.

It can be seen from Table 3 that the ITS value increased with increase in cement percentage for both the 28 and 45 days cured specimens. According to Austroads (23), for cement-stabilized material, IDT should be in between 0.125 to 0.1 times the UCS value. For the subject material due to the incorporation of fiber, a high tensile strength was expected and it was found to be 0.13 times that of the UCS value.

Stiffness Parameters

FM and dynamic stiffness modulus after 28 days curing for 2%, 4%, and 6% cement-stabilized specimens were determined at 30% of failure load and are presented in Table 3. The results show that increasing cement content has a positive effect on both the FM and dynamic modulus.

Flexural Fatigue

Four point flexure fatigue test results were obtained and fitted to SR-N curve (Figure 6) which represents the relationship between stress ratio (SR) and fatigue life (N). The SR is the ratio between applied flexural stress and FS which minimized the variation in FS on individual specimens on fatigue life. The best fit equation is given by Equation 3, which resulted in an R^2 value of 0.78 and may be considered as a good correlation between SR and N.

$$\ln N = 22.83 - 18.66 SR \quad (3)$$

Durability Studies

The residual UCS values were determined for both 7 and 28 days cured specimens and presented in Table 4. It may be observed from the results that for 28 days cured samples, loss in strength were less, compared to 7 days cured samples. The loss in strength decreased with increase in cement percentage.

Leaching Studies

The leaching test results extracted from the AAS analysis are presented in Table 5. It may be seen that except for mercury, for all other metals, the concentration was below threshold limits for both 7 and 28 days cured samples. Therefore ex situ treatments of FA for mercury using methods like soil washing–acid extraction, thermal treatment, vitrification, etc., is needed prior to its usage.

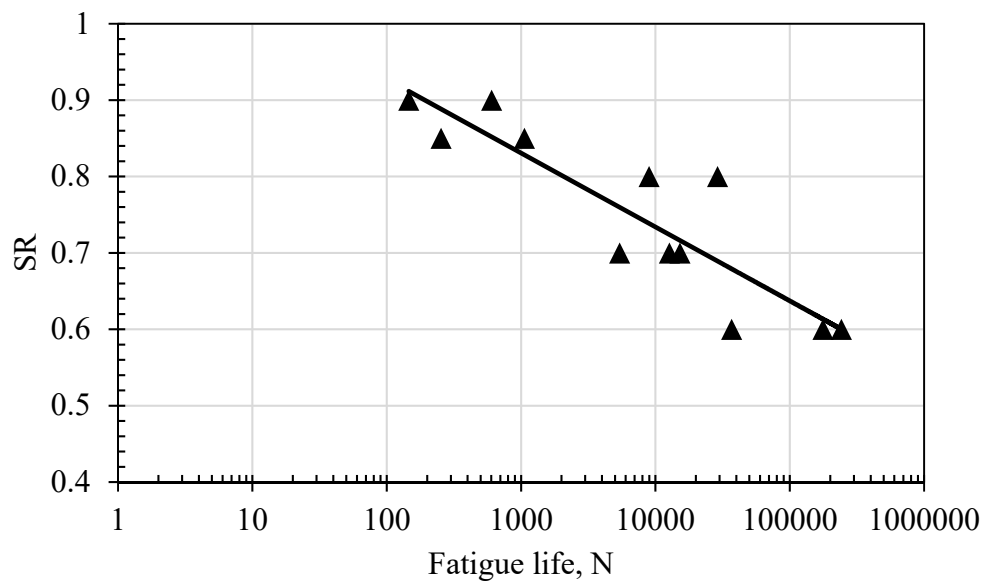


FIGURE 6 SR-N curve for 70FA-30SD-12PP composition.

TABLE 4 Percentage Decrease in UCS Value after Wetting-Drying Cycles

% of Cement	% Decrease in UCS Value After 10 Wetting-Drying Cycle	
	7 Days Cured Specimen	28 Days Cured Specimen
2	15	11.7
4	11	10.1
6	9.4	7.6

TABLE 5 Heavy Metal Concentration (ppm) in Leachate of 7 and 28 Day Cured Sample

Curing Days	% of Cement	Mercury	Lead	Chromium	Cadmium	Iron	Zinc
7	2	0.12	1.21	0.058	0.05	0.102	0.042
	4	0.41	1.56	0.049	0.007	ND	0.506
	6	0.23	1.04	ND	0.009	ND	ND
28	2	0.12	0.023	0.007	0.04	0.008	0.016
	4	0.3	0.135	0.014	0.06	0.009	0.08
	6	0.1	0.588	0.018	0.01	0.003	0.202
Allowable limit		0.002	0.05	0.05	0.005	0.3	5
Threshold limit		0.2	5	5	0.5	30	500

Correlation Between the Mechanical Strength Parameters

This is a common practice to evaluate the strength in terms of UCS for the stabilized layers. For use of stabilized FA as base-course material, the tensile strength, FS, and FM also need to be determined to justify suitability of the material. However, in field, it is not always possible to find all the above-mentioned values as these experiments require special equipment. In the present study, an attempt was made to correlate the FS and FM values with UCS (which is a simple compression test). Also, the relationship between different stiffness parameters like FM and IDT modulus were also established as presented in Equations 4–8.

$$UCS = 0.612 * \% \text{ of cement} \quad (R^2=0.98) \quad (4)$$

$$FS = 1.4635 \ln(UCS) + 0.4741 \quad (R^2=0.94) \quad (5)$$

$$FM = 3669.9 \ln(UCS) + 2401.3 \quad (R^2=0.96) \quad (6)$$

$$FM = 2.772 FS + 903.13 \quad (R^2=0.97) \quad (7)$$

$$FM = 0.9087 * E_{IDT} + 2371.6 \quad (R^2=0.94) \quad (8)$$

CONCLUSIONS

The present study was conducted to characterize cement-stabilized fiber-reinforced FA as a structural layer material for potential application in rural road pavements. An extensive laboratory investigation was carried out to determine various mechanical properties, durability

characteristics and environmental acceptability of the material through leaching study. The following conclusions were drawn from this study:

FA and SD in a proportion of 70:30 was found to provide adequate strength with maximum utilization of FA, when stabilized with cement. UCS value increased with increase in cement percentage. Strength also increased with increase in curing time. No significant gain in UCS was observed with the addition of 0.5% polypropylene fiber, however, the inclusion of fibers reduced the brittle behavior of the material. Addition of polypropylene fibers showed an increase of 105% and 107% in FS for 28 and 45 days cured samples respectively with 6% cement stabilization (compared to 6% cement stabilization without fiber reinforcement). An increase in IDT strength was also noted with the inclusion of the fiber. The FM of the stabilized FA increased significantly with the addition of fibers i.e. for 6% cement-stabilized FA-SD, the FM (at a loading rate of 1 Hz) was found to be more than 7,000 MPa. Poisson's ratio of cement-stabilized FA-SD was found to be in the range of 0.1 to 0.2. High fatigue life was observed for cement-stabilized FA-SD due to the inclusion of fibers in the mix. The correlations developed in this study will help the engineers to predict complex mechanical properties from simple UCS test results. Durability studies (using wet-dry durability test) showed improved moisture resistance with an increase in curing period and an increase in cement content. Leaching test results showed that the concentration of heavy metals in leachate is well below the threshold limits of the standard of drinking waters except for mercury. Therefore ex situ treatments of FA for mercury using methods like soil washing/acid extraction, thermal treatment, vitrification, etc., is needed prior to its usage.

Based on the laboratory test results, it can be concluded that cement-stabilized fiber reinforced FA-SD has the potential to be used in the subbase and base layers of rural road pavements with prior ex situ treatment of FA for mercury. However, a field trial is needed before recommending its usage in large scale.

FUTURE SCOPE

The present study only targets mechanical properties to decide the dosage, however to set the upper limit of the stabilizer content, a shrinkage study is proposed to avoid having a high-strength crack-susceptible mix. Also, curing behavior of stabilized FA at different temperatures and humidity conditions need to be studied. Due to resource constraints, presence of heavy metals like arsenic, manganese, cobalt, etc., could not be conducted. Therefore it may be considered under the future scope of research. Finally, the study may be extended with FA from different sources to establish generalized relationships for its use as a conventional road construction material.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: Study Conception and Design: Bhowmik and Sahoo; Data Collection: Bhowmik and Beriha; Analysis and Interpretation of Results: Bhowmik, Beriha, and Sahoo; and Draft Manuscript Preparation: Bhowmik, Beriha, and Sahoo. All authors reviewed the results and approved the final version of the manuscript.

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STABILIZATION

Assessment of the Potential for Salt Damage to Asphalt Surfacing When Chloride-Treated Unpaved Roads Are Upgraded to a Seal Standard

DAVID JONES

University of California, Davis

Experience gained during research into chemical dust control—fines preservation has indicated that the reduced maintenance on, and reduced gravel loss from, treated roads justifies the use of these chemical treatments as a cost-effective road maintenance and management strategy. Treatments also serve as a means of preserving layer thickness and integrity in stage construction if upgrading the road to a paved standard is planned. However, the potential for damage to primed bases and bituminous surfacings placed on chloride-treated layers as a result of salt crystallization underneath the surface during hot dry conditions in arid and semi-arid climates has to be considered. A laboratory test method was therefore developed to gauge the potential for this type of damage. Calcium chloride, a commonly used dust palliative, was used in the experiment. The test, which simulated a worst-case scenario, followed by two field experiments that were evaluated for 36 months, indicated that salt damage to base materials, whether primed or unprimed, and to bituminous surface treatments or thin asphalt concrete layers is unlikely to occur on roads previously treated with calcium chloride. Care should be taken in the choice of surfacing if the electrical conductivity of the top 25 mm of treated material on the road exceeds 0.15 S/m prior to priming the base and placing the surfacing. This limit can be relaxed if laboratory testing, as described in this paper, indicates that salt damage is unlikely.

To view this paper in its entirety, please visit:

https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

STABILIZATION

Stabilization of Pavement Foundation Material Using Polymeric Additive *Characterization and Performance*

ROMEL N. GEORGEES
Kirkuk University, Iraq

In Australia, polyacrylamide (PAM) additives have recently been shown to provide advantages for the construction and maintenance of low-volume roads over traditional cementitious additives. However, limited attempts to assess the improvements in geotechnical characteristics of pavement foundation materials resulting from the use of polyacrylamide additive have been carried out. This study has been undertaken to investigate the benefit of using a synthetic PAM additive on the engineering properties of pavement subgrade soils. The primary objective of this study is to investigate the potential of a synthetic PAM additive on improving the physical and engineering properties of subgrade soils. The focus of this study is on the changes to engineering and physical properties at specimen and particle levels of three types of soils ranged from non-cohesive to medium and very cohesive soils.

The experiments conducted refer to representative three subgrade soil samples (from Victoria, Australia) consists of crushed rock material, sandy clay soil and a mix of fine-grained gravel and silt soil. On the other hand, the polymeric additive used is a synthetic soluble anionic PAM with a moderate charge density of approximately 18% and a high molecular weight typically between 12 and 15 Mg/mol.

Laboratory test samples were prepared in accordance with relevant Australian standards AS 1289.1.1. The required amount of PAM, according to the supplier is 0.002% by dry weight of the soil. Particle size distributions of the three soils are ranged from well to medium and poorly graded. Geotechnical results show that the crushed rock material has no plastic limit value with very small quantity of clay fines. However, a sandy clay soil and a gravelly clay soil show moderate to high plasticity indices, respectively.

At specimen level, the mechanism of PAM stabilization was investigated using the potential improvements of dry density, unconfined compressive strength (UCS), elastic modulus, and toughness characteristics. However, at particle level, analysis was carried out using X-ray diffraction (XRD) and scanning electron microscopy tests to identify any changes in the matrix of the PAM-treated soils. These two levels of assessment were specifically employed to assess the mechanism by which PAM interacts with soil particles.

The results show that PAM plays an important role in increasing the dry density, strength and elastic modulus of the soil tested, as well as increase in toughness. The level of improvement in strength ranged from 11.6% to 26.8%. Soils with less fines content reveal greatest strength gain and soils with higher fines content exhibit less strength gain. The effectiveness of soil stabilization using PAM additive is clearly evidenced when assessing the soil capacity to absorb energy, i.e., increased toughness and hence increased capacity to carry loads. The increase is found to vary depending on the soil type. The level of improvement ranges from 12.6 % to 87.7%. Treated samples of soils with less fines content show higher toughness index and treated

samples of higher fines content show less. Using dry density, UCS and elastic modulus parameters to assess the potential improvements of PAM stabilization for soils with different amounts of clay, the maximum difference in these parameters between treated and untreated samples is found at 1% clay content. Therefore, increasing clay percentage in the soil matrix did not contribute significantly to increase the adsorption of PAM onto soil particles.

On the other hand, the analysis at particle level shows that there are no changes in mineralogy of soil samples after being treated with PAM. The diffractograms indicates similar phases in both untreated and treated samples for all three soil types. The intensity of the peaks decreases for the treated samples due to the presence of PAM, which decreases the concentration of soil minerals in the sample tested by XRD (increases in the distance between the planes in the corresponding unit layers). The interaction between PAM and soil particles takes place mainly by adsorption and physical bonding. Investigating the surface morphologies of the treated and untreated samples for the tested soils shows that treated soils display characteristics of greater bonding between individual particles that appeared to be glue-like. The “glue” increased the required tensile strength between individual particles and hence, more aggregation has been noticed as well.

The resulting benefits of such an investigation can be observed in the improving knowledge of the behavior of PAM-treated materials in pavement structure. Further, the assessment will enhance the understanding of engineering significance of using synthetic polyacrylamide for pavement materials modification and will also result in increased reliability of these alternative sustainable materials and further incorporation into practice by road authorities. Moreover, the resulted increase in layer toughness will prolong intact foundation resulting in delayed deformation in the foundation (subgrade or subbase) layers, and thus delaying the major rehabilitation of the pavement.

Unpaved Roads Management 2

UNPAVED ROADS MANAGEMENT 2

Inventory-Based Rating System

A Stable and Implementable Method of Condition Assessment for Unpaved Roads

PETE TOROLA

TIM COLLING

JOHN KIEFER

Michigan Technological University

The current rating systems for unpaved roads lack stability and reliability and, therefore, provide little benefit as a project- or network-level metric. Since many of these systems are derived from paved road assessment systems, they focus heavily on surface distresses rather than road width, drainage, and other features. Because unpaved roads can change rapidly, measuring surface distresses is an unreliable rating factor. The Inventory-Based Rating (IBR) System™ assesses unpaved roads on surface width, drainage adequacy and structural adequacy. These features impact road users and have significant costs associated with creation and maintenance. The system defines a baseline condition for each inventory feature with its tiered good–fair–poor rating. Five counties, selected based on their road network classification, participated in a pilot IBR data collection. User feedback was also collected from participants. The study showed very high repeatability and reliability of the IBR system. It also provided productivity benchmarking, which can forecast the time commitment for data collection. User feedback resulted in modifications to the system.

INTRODUCTION

Since the early 1990s, Michigan’s road-owning agencies have been assessing and reporting to the Michigan legislature on the conditions of their paved roads (e.g., asphalt, concrete, and sealcoat) using the Pavement Surface Evaluation and Rating (PASER) (1–3) system, which provides a cost-effective, network-level condition assessment metric for paved roads. In 2002, the Michigan Transportation Asset Management Council (TAMC) assumed responsibility for collecting and reporting statewide PASER data as well as bridge condition data, which uses the National Bridge Inventory rating system, to the legislature (4). Then, in 2018, the TAMC required data collection on unpaved roads using the IBR system, which provides a cost-effective and stable network-level condition assessment measure for unpaved roads. Michigan Technological University’s Center for Technology & Training (CTT) developed the IBR. Colling and Kueber-Watkins detail the genesis of this assessment system in the research report *Inventory Based Assessment System for Unpaved Roads*, submitted to TAMC (5), which outlines possible testing and statewide implementation of the IBR system.

Limitations of Existing Unpaved Road Assessment Systems

Condition assessment systems serve two purposes: to provide project-level guidance for inferring the necessary treatment for a given asset and to provide a network-level metric for evaluating overall system performance. The PASER system for paved roads, for example, offers project-level guidance through condition ratings that help road owners determine appropriate treatments (1–3); it also offers network-level measures that enable efficient, easy determination of the necessary investment for maintaining or working toward a condition target. The best assessment systems serve both purposes.

Many condition assessment systems exist for unpaved roads (6). Most unpaved road condition assessment systems evolved from systems for paved roads and, thus, focus on the extent and severity of surface distresses. For paved road networks, surface condition significantly impacts road use and its decline typically drives improvement work. Surface distress works well for measuring the quality of paved roads because surface distresses change slowly, remaining relatively static over the course of a year (1–3), and require significant effort to repair. Because of this slow rate of decline coupled with significant effort to repair, a condition rating every one to two years provides sufficient data for managing paved roads.

Unlike paved roads, unpaved roads can have rapid surface condition changes over weeks or even days, making surface condition data a quickly-outdated (7) and highly-variable network-level metric. In addition, poor unpaved road surface condition does not always reflect loss in road value or the surfacing's life as some surface distresses may be remedied by low-cost maintenance like grading. Furthermore, the quality of other inventory elements—such as ditches and culverts, lane widths, shoulders, and structural gravel—can adversely influence road use. Road users, for example, may consider potholes or ruts on an unpaved road as a secondary inconvenience compared to a narrow surface width that precludes the operation of two-way vehicle traffic at any significant speed. Finally, many unpaved roads do not have or need basic inventory elements common to paved roads, and they fluctuate greatly in design, construction, use, and maintenance. Thus, an exclusive focus on surface condition is problematic for unpaved roads.

Premise of Inventory-Based Rating System

The IBR system (5) assesses conditions for three characteristic elements of an unpaved road. Of all the elements of unpaved roads, the IBR elements—surface width, drainage adequacy and structural adequacy—impact road use the most and carry the greatest level of investment to create them. Further, these IBR elements do not change rapidly and, thus, a rating only requires updating when construction activities occur or when lack of maintenance leads to loss or degradation of a road feature. When these elements do degrade, they require significant construction or maintenance efforts to improve. Monitoring these IBR elements over time at a network level provides measures that illustrate the impact of investments on the unpaved road network.

Defining a baseline condition for each of the IBR elements creates a reference for road comparison; each element's baseline is determined by physical characteristics that are considered acceptable for the majority of road users with guidance from design standards. The IBR system™ classifies elements at the baseline condition as good. Not meeting the baseline condition results in a lower rating of fair (a range moderately below the baseline) or poor (a

range significantly below the baseline). IBR elements are apparent enough to be evaluated from a moving vehicle and do not typically require hand measurement. More information on the genesis of the specific criteria and bin ranges for each element can be found in Colling and Kueber-Watkins' report (5).

The good, fair, and poor ratings for each IBR element (detailed below) accrue "points" that come together to form an IBR number. The IBR numbers create a ten-point system, where a 10 is reserved for roads newly constructed to the baseline conditions while 1 indicates a road significantly below the baseline for all three assessed elements. For each element being rated on a road segment, characteristics that meet the baseline criteria, or are assessed as good, generate more points. The number system used for IBR approximates the PASER scale. More detail on the derivation of IBR numbers can be found in the report by Colling and Kueber-Watkins (5).

The IBR system uses the following criteria:

Surface Width

Surface width is assessed by estimating the width of the traveled portion of the road, including travel lanes and any travel-suitable shoulder.

- Good: 22 ft (6.7 m) wide or more;
- Fair: 16 to 21 ft (4.9 to 6.4 m) wide; or
- Poor: 15 ft (4.6 m) wide or less.

Drainage Adequacy

Drainage adequacy is assessed by, first, estimating the difference in elevation between the ditch's flow line or level of standing water (if present) and the top edge of the shoulder and, second, determining the presence or absence of secondary ditches (i.e., shoulders over 6 in., or 15 cm, tall that are able to retain surface water).

- Good: 2 ft (61 cm) or more of difference in elevation; no secondary ditches are present;
- Fair: from 0.5 to less than 2 ft (15 to less than 61 cm) of difference in elevation or 2 ft (61 cm) or more difference in elevation with secondary ditches present; or
- Poor: less than 0.5 ft (15 cm) of difference in elevation, secondary ditches may or may not be present.

Structural Adequacy

Structural adequacy is assessed by estimating the thickness of good quality gravel (crushed and dense graded). Assessment should rely on local institutional knowledge and should not require involved testing or probing of existing conditions.

- Good: more than 7 in. (20 cm) of good gravel;
- Fair: 4 to 7 in. (10 to 18 cm) of good gravel (i.e., needs placement of 1 to 4 in., or 2.5 to 10 cm, gravel to meet baseline condition); or

- Poor: less than 4 in. (10 cm) of good gravel (e.g., needs placement of 5 to 8 in., or 13 to 20 cm, gravel to meet baseline condition).

OBJECTIVE AND SCOPE

This study aimed to estimate the scope, cost, and other planning factors necessary for potential statewide IBR collection. The study gathered data on various types of unpaved roads in Michigan—with differences in users and network types—under real world conditions to determine the repeatability and accuracy of the IBR system. The study also benchmarked data collection speeds, determined training and guidance needs, and secured direct feedback from transportation professionals who would collect and use IBR data. This study sought to define the type of information necessary for implementing full-scale collection and to assess the value of these data as a local agency road-management tool through direct user feedback.

METHODS

Selection of Data Collection Locations

Michigan's unpaved roads vary greatly from county to county in their use, construction, distribution, and maintenance. Based on overall function, management, and maintenance, the project team defined three classifications of unpaved road networks (see Figure 1).

Low-Volume Terminal Branch Networks

These unpaved roads provide access to only a few properties, are primarily the “ends” of the road system, and are often seasonal roads. They experience low traffic volumes. Counties in the Upper Peninsula and northern Lower Michigan generally fall into this category.

Agricultural Grid Networks

These unpaved roads support the local agricultural economy by providing regular access to farms. They experience seasonally higher volumes of traffic and larger truck loads. Generally, these networks are maintained all year because they serve both residents and agricultural industries.

Suburban Residential Networks

These unpaved roads enable year-round local access to rural residential properties located near urban centers. These roads serve predominantly passenger vehicle traffic. These road networks are near urban centers and are typically located in the population belt between Grand Rapids and Detroit.

The study sought to collect 1,000 mi of unpaved road rating data using the IBR system in a minimum of four counties, at least one from each network classification, spread throughout the state. This sample size could enable accurate predictions for statewide data collection rates,

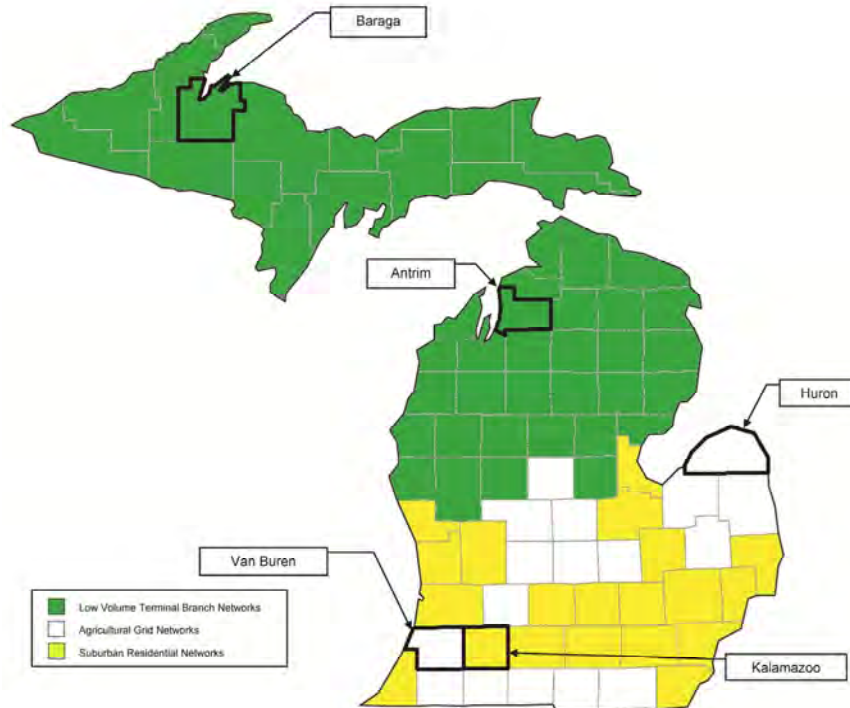


FIGURE 1 Qualitative classification of counties based on unpaved road network type. Volunteer pilot counties are outlined in bold.


determination of the validity of the system, and necessary improvements to the training materials. Cooperation was voluntary, so county road commission and regional planning staff participated in the study at their own expense.

Pre-Field Work Training

Prior to rating unpaved roads, CTT engineers trained participating agency employees and planning agency representatives. First, participants received the *Inventory Based Assessment Systems for Unpaved Roads* report for their review. Then, participants took part in a 2-h training presentation and in-class rating exercises that provided experience using the IBR system. They also received a two-page quick reference handout that detailed the IBR criteria (Figure 2).


Data Collection Methodology

This study had three discrete data collection events. The first event gathered IBR data (i.e., assessments of surface width, drainage adequacy, and structural adequacy, and the resulting IBR number) and productivity benchmarks (i.e., time spent rating and miles rated) in each county over 1 to 2 days per county. For each roadway segment, each IBR element received an assessment of good, fair, or poor according to the IBR system criteria. Individual team members generated “blind” IBR data from random road segments which was compared to the actual measured assessment of the segment.

<h2 style="margin: 0;">IBR Field Guide</h2>	 <p style="margin: 0;">Inventory-based Rating System™ for Gravel Roads</p>
<p>The Inventory-Based Rating System™ for unpaved roads functions by defining a baseline condition for each of the three inventory features: Surface Width, Drainage Adequacy and Structural Adequacy. These features do not change rapidly and are apparent enough to be evaluated from a moving vehicle without the need for fine measurement. The resulting 1 through 10 IBR number can be found in the <i>Rating Lookup Chart</i> on the back or it will be generated automatically when using the Roadsoft® Laptop Data Collector.</p>	

	GOOD	FAIR	POOR
Width	<p>22 feet or greater in surface width</p> <p>Vehicles have sufficient room to pass by each other when approaching in the opposite direction. Reduction of speed is unnecessary.</p> <p>Remedy/Action: None</p>	<p>16 to 21 feet in surface width</p> <p>Vehicles should reduce speed to pass by each other when traveling in the opposite direction.</p> <p>Remedy/Action: 1' to 6' of widening</p>	<p>15 feet or less in surface width</p> <p>One vehicle should slow down and pull over and the other should reduce speed to pass by when traveling in the opposite direction.</p> <p>Remedy/Action: 7' to 15' of widening</p>

Quick Tip: When driving at 20 mph, does the driver feel they need to slow down when approaching another vehicle in the opposite lane? (see additional guides on back)

Drainage Adequacy	<p>Distance X is the difference in elevation from the ditch flow line (or any standing water, whichever is less) to the top edge of the shoulder.</p> 		
	GOOD	FAIR	POOR
	<p>Distance X is 2 feet or more, and: No secondary ditches* are present.</p> <p>Remedy/Action: None</p>	<p>Distance X is 0.5 to < 2 feet, or: Distance X is two feet or more, with secondary ditches* present.</p> <p>Remedy/Action: Vertical separation from runoff water is present but more is needed and/or remove secondary ditches*.</p>	<p>Distance X is less than 0.5 feet</p> <p>Remedy/Action: Vertical separation from runoff water needs to be created.</p>

*Secondary ditches should only be considered if they are over six inches tall.

	GOOD	FAIR	POOR	
Structural Adequacy	<p>Existing gravel thickness is: greater than 7 inches</p> <p>Alternatively, if thickness is not known then rate by Historical Measure: 1 inch ruts or 3 foot potholes</p> <p>Emergency maintenance to make road passable was</p> <p>Remedy/Action:</p>	<p>4 to 7 inches</p> <p>did not develop throughout the year</p> <p>not required, leaving the road passable throughout the year (when plowed)</p> <p>None</p>	<p>less than 4 inches</p> <p>developed during the thaw or very wet periods</p> <p>necessary to make the road passable during wet periods</p> <p>Placement of 1 to 4 inches of good quality gravel would be recommended as a fix assuming drainage is good**</p>	<p>developed during much of the year</p> <p>required to make the road passable throughout the year</p> <p>Placement of 5 to 8 inches of good quality gravel would be recommended as a fix assuming drainage is good**</p>

**Look into what is causing structural problems because more gravel is not a good remedy for bad cross slope drainage.

FIGURE 2 Front page of the IBR system quick reference guide.

The second data collection event verified gravel thickness at randomly selected locations. The CTT project team measured gravel depth on a sample of the rated roads in each county. These gravel depth measurements determined the accuracy of local agency staff knowledge about a road's structural adequacy.

The third data collection event is addressed in the section "Combined PASER and IBR Data Collection."

IBR System Data Collection

In order to gather IBR data quickly and accurately, collection tools included Roadsoft and the laptop data collector (LDC) software programs, which would likely be used in full-scale collection. Roadsoft is a GIS-based asset management program used by agencies in Michigan for storing, managing, and analyzing roadway assets and associated data. The LDC facilitates field collection of data for Roadsoft by connecting with a recreational-grade GPS to associate spatial locations with the data. Roadsoft and the LDC use a statewide unified framework base map, allowing data stored in Roadsoft to be related to other Michigan agencies at regional and state levels.

Prior to the data collection event, each county provided the project team with a copy of their Roadsoft database. From this initial inventory of each county's unpaved roads network, the project team planned the routing and size of collection areas with each agency's management, engineering staff, and foremen. Selected portions of the unpaved road networks were to be representative of the county and were to generate useful data for agency management. Subdividing data collection areas by township yielded meaningful reporting blocks and reflected individual township policy for constructing and maintaining unpaved roads.

During field collection, collection teams entered IBR data into the LDC, which minimized transcription or location errors. For safety reasons, field collection involved a minimum of three raters, with duties being driving, data entry, or navigating. To minimize their influence on raw data collection, the CTT staff entered data into their own LDC and did not direct or guide assessments from the collection team. Data collection occurred on a continuous basis from a moving vehicle except when stops were necessary to investigate hard-to-see or hidden features. To orient collection teams to field conditions, initial data collection efforts involved physical checks of road width and ditch depth using a tape measure. Each team member determined an IBR assessment and, in some instances, the IBR number, and all members agreed upon the data. When they lacked a consensus, raters and the project team employed physical checks to determine assessments.

At random intervals (every 20 to 60 min) during data collection, teams made blind ratings of road segments. For blind assessments, raters individually observed and assessed IBR elements from the vehicle and recorded their assessments and, in some instances, the resulting IBR number; team members were not permitted to exchange information or talk. After all team members submitted their data, the group discussed the data until they reached a consensus. Raters then verified the accuracy of surface width and drainage adequacy consensus assessments using physical checks; the local agency representative verified the structural adequacy consensus assessment since gravel thickness could not be measured during field data collection. The CTT project team recorded consensus data in the LDC.

Benchmarking productivity involved tracking start–end times (including time traveling to–from data collection areas, but not time spent driving to meet the rating team) and break times (excluding lunch breaks) as well as vehicle miles traveled and miles of road rated. The LDC

tools supplied the rated road mileage data. Rating productivity data represents the teams' overall average collection rate for IBR data without collecting paved road condition or other data.

Gravel Thickness Data Collection

Following IBR data collection, the CTT project team checked the gravel depth for at least nine road segments (using a core drill or demolition hammer) in each pilot county on random county roads that had been rated during collection events. Gravel thickness was measured at the center of the travel lane on one randomly selected side of the road. These thickness measurements determined the accuracy of structural adequacy estimates supplied by the local agency representative, who solely used local knowledge. The CTT project team verified with local agency maintenance staff that no significant additions or removal of gravel occurred between the first collection event and this collection.

Combined PASER and IBR Collection

The third data collection event occurred only in Baraga County. The rating team collected PASER data for paved roads and IBR data for unpaved roads in a combined collection, and the project team gathered productivity benchmarks for PASER and IBR data collections to determine the impact of combined data collection efforts. On the first day, the rating team collected both IBR data on unpaved roads and PASER data on paved roads. On the second day, they collected only PASER data.

User Feedback

The CTT project team gathered user feedback on the IBR system from the study participants. They collected comments at the training, during data collection, and during a post-collection conference call. These comments served to refine the rating system, correct training deficiencies, and identify training areas needing more explanation.

RESULTS

Network Classification and IBR Collection Results of Participating Counties

The participating counties were Antrim, Baraga, Kalamazoo, Huron, and Van Buren (Figure 1).

Antrim County

Antrim County classifies as a low-volume terminal branch network because its population was less than 100,000 people (8) and more than 40% of the land area was covered by forests (9). Efficient travel was difficult due to lakes and streams dividing the county. The rated road network predominantly consisted of short-length, low-volume, seasonal, dead-end roads.

Antrim County's rated unpaved roads exhibited narrow widths with both minimal drainage and structural gravel layer, leading to overall low IBR numbers. Several unpaved roads in the framework base map terminated early or were non-existent; thus, data collection verified

and documented corrections to the framework base map, thereby better defining Michigan's road system.

Baraga County

Baraga County classifies as a low-volume terminal branch network because its population was less than 100,000 people (8) and more than 40% of the land area was covered by forests (9). The unpaved roads provide mostly seasonal or very low-volume access to recreational and forest properties. They are often ends of the road network; thus, rating road segments required more total miles (kilometers) of travel. Roadside vegetation height complicated productivity by requiring the rating team to exit the vehicle to assess ditch presence/absence and depth.

Baraga County's rated unpaved roads generally had narrow widths (slightly wider than one lane), minimal drainage, and little or no structural gravel layer; this led to overall low IBR numbers. While these characteristics are conventional for very low-volume unpaved roads that enable access to a few rural properties, many of Baraga County's rated nonseasonal unpaved roads had minimal ditches and structural gravel.

Huron County

Huron County classifies as an agricultural grid network because its population was less than 100,000 people (8), its land area has less than 40% forests coverage (9), and its road network follows 1-mi-long section-line grid patterns. Generally speaking, IBR data collection for agricultural grid networks like Huron County is efficient because the interconnected grid pattern of their unpaved roads permits increased collection speeds. These roads accommodate higher speeds, higher volumes, and heavier travel loads, and they are reliable for connecting locations (i.e., farm-to-market roads).

Huron County's rated unpaved roads were generally wide, fully ditched, and contained significant structural gravel layers; this led to high IBR numbers. Notably, all of the townships used for data collection had significantly more unpaved miles (kilometers) of road than paved miles (kilometers).

Kalamazoo County

Kalamazoo County classifies as a suburban residential network because its population was over 100,000 (8). Kalamazoo County's unpaved network was concentrated along the county borders and away from the city of Kalamazoo; the network serves agricultural and rural residential needs. IBR data for the entire 103.1-mi (106.0-km) network was collected in 1 day. Kalamazoo County's rated unpaved roads exhibited moderately poor IBR numbers.

Van Buren County

Van Buren County classifies as an agricultural grid network because its population was less than 100,000 people (8) and its land area has less than 40% forest coverage (9). Most land use was rural residential and agricultural. The unpaved network interconnects with paved roads, increasing the efficiency of data collection. But, since Van Buren County had more paved roads than Huron County, collecting data required more travel between unpaved segments. As with Baraga County, unpaved roads often had high grass along the shoulders, making drainage

adequacy assessment difficult. Van Buren County's rated unpaved roads had fair surface widths, fair drainage adequacy, and good structural adequacy leading to moderately good IBR numbers.

Productivity Benchmarking

Productivity benchmarking can help forecast the time commitment for collecting IBR data for Michigan's gravel roads. Therefore, the CTT project team recorded and calculated IBR data collection speeds to account for the unique geographic and road network features of each county. The main factors that influenced IBR data collection speed were the network classification (which related to the connectivity of the unpaved roads) and travel speed (which was dictated by the condition of the road being rated). Recorded collection times represent the time actively rating roads or transiting to and from rating segments; however, the collection time does not account for breaks for lunch and switching of rating crews.

Table 1 summarizes the productivity benchmarking for the IBR data collection. Antrim County's IBR data collection was the slowest. Huron County had the most productive collection.

The time of year likely influences IBR data collection speed as well. Collecting data later in the growing season is increasingly difficult and less reliable since drainage adequacy features can become hidden by growth of roadside vegetation.

Combined PASER/IBR Collection Benchmarking

In Baraga County, 2 days of IBR-only collection gathered 99.2 mi (159.6 km) of data at 8.8 mi (14.2 km) rated per hour. An additional day of combined IBR (unpaved) and PASER (paved) data collection yielded 40.9 mi (65.8 km) of IBR data and 110.4 mi (177.7 km) of PASER data for a total of 151.3 mi (243.5 km) of data collected at 20.9 mi (33.6 km) rated per hour. Another additional day of PASER-only data collection resulted in 81.6 mi (131.3 km) of data at 14.8 mi (23.8 km) rated per hour. The rate of collecting IBR data and PASER data together was higher than collecting either PASER data only or IBR data only due to minimizing the time traveled without rating.

TABLE 1 IBR Data Collection Statistics by County (Statistics Are Indicative of Collecting Only IBR Data on Unpaved Roads)

County	Collection Time (h)	Gravel Miles Rated (km)	Rating Productivity in mph (km/h)	Total Miles Driven (km)	Travel Speed in mph (km/h)	Total Driven Miles (km) that Were Rated
Antrim	11.50	71.976 (115.834)	6.3 (10.1)	234.5 (377.4)	20.4 (32.8)	31%
Baraga	11.33	99.205 (159.655)	8.8 (14.2)	238.0 (383.0)	21.0 (33.8)	42%
Huron	8.67	245.185 (394.587)	28.3 (45.5)	289.0 (465.1)	33.3 (53.6)	85%
Kalamazoo	9.92	103.163 (166.025)	10.4 (16.7)	314.0 (505.3)	31.7 (51.0)	33%
Van Buren	12.42	141.524 (227.761)	11.4 (18.3)	318.0 (511.8)	25.6 (41.2)	45%
Total	53.83	661.053 (1063.862)	12.3 (19.8)	1393.5 (2242.6)	25.9 (41.7)	47%

Systemwide IBR Collection Estimates

This study's overall average rate for collecting IBR data on unpaved roads was 12.3 mi (19.8 kilometers) per hour. Thus, to capture the estimated 40,000 centerline miles (estimated 64,000 kilometers) of unpaved roads in Michigan requires roughly 3,200 h of data collection. This averages to 39 h of IBR data collection per county.

If one assumes that this study experienced average collection rates and that unpaved roads were evenly distributed in each county, then segregating counties by their road network classification can also provide an adjusted average of the hours needed to collect IBR data. Classifying Michigan's counties by network (refer to Figure 1) yields:

- 46 low-volume terminal branch networks (Antrim and Baraga Counties):
 $(6.3 \text{ mph} + 8.8 \text{ mph}) / 2 = 7.55 \text{ mph}$ average collection speed
 or: $(10.1 \text{ km/h} + 14.2 \text{ km/h}) / 2 = 12.2 \text{ km/h}$ average collection speed
 $46 \text{ counties} \times 481 \text{ mi (774.1 km) per county} / 7.55 \text{ mph (12.2 km/h)} = 2,930 \text{ h}$
- 17 agricultural grid networks (Huron and Van Buren Counties):
 $(28.3 \text{ mph} + 11.4 \text{ mph}) / 2 = 19.85 \text{ mph}$ average collection speed
 or: $(45.5 \text{ km/h} + 18.3 \text{ km/h}) / 2 = 31.9 \text{ km/h}$ average collection speed
 $17 \text{ counties} \times 481 \text{ mi (774.1 km) per county} / 19.85 \text{ mph (31.9 km/h)} = 411 \text{ h}$
- 20 suburban residential networks (Kalamazoo County):
 10.4 mph collection speed
 or: 16.7 km/h collection speed
 $20 \text{ counties} \times 481 \text{ mi (774.1 km) per county} / 10.4 \text{ mph (16.7 km/h)} = 925 \text{ h}$ where
 mph = miles per hour, km/h = kilometers per hour, and 481 mi (774.1 km) is the
 average per Michigan county based on the estimated 40,000 centerline miles
 (estimated 64,000 km). Therefore, the time to collect unpaved road condition data
 is approximately 4,300 h—or roughly 52 h per county—for IBR-only data
 collection in Michigan:
- Total hours
 $2,930 \text{ h} + 411 \text{ h} + 925 \text{ h} = 4,260 \text{ h total.}$

The combined PASER and IBR data collection was significantly more productive than PASER or IBR collection alone. Baraga County's combined data collection was 41% more productive than PASER collection alone. While this gain would be rare for other network types, the project team believes that combined collection rates averaging 20 mph (32.2 km/h) are likely. This means that collecting 100% of the 40,000 centerline miles (estimated 64,000 km) of unpaved roads would only require an additional 2,000 h—approximately 24 h per county—during a combined collection event. Table 2 shows systemwide estimates.

Repeatability of Measurement

Repeatability relies on the accuracy and consistency of each rating team member's perception of road conditions during rating. Accuracy and consistency can be ascertained and validated by subtracting the point values of periodic blind individual assessments from group consensus assessments—or ground truth—for surface width, drainage adequacy, and structural adequacy as well as the overall combined IBR number [see *Inventory Based Assessment Systems for Unpaved*

TABLE 2 Systemwide IBR Data Collection Estimates

Collection Method	Rating Productivity mph (km/h)	Time to Collect 40k Unpaved mi (64k mm) in Michigan (h)	Average Time per County (h)
IBR only (average rate)	12.3 (19.8)	3,252	39
IBR only (segregated by county type)	7.55 to 19.85 (12.15 to 31.95)	4,260	52
Combined PASER and IBR	20 (32.2)	2,000	24

Roads (5) for details on point value calculations]. High match percentages and low point spreads for each element indicates accurate and consistent rating.

The study collected 281 blind rating sets from 58 road segments, divided almost equally between the five counties. Blind assessments matched ground truth with a frequency of 92.2% for surface width, 85.1% for drainage adequacy, and 90.7% for structural adequacy (Figure 3). Comparing the aggregate IBR numbers from blind assessments and the ground truth showed that 72.2% were exact matches and 92.9% were within a tolerance of ± 1 rating point on the 10-point IBR scale.

While ground truth for surface width and drainage adequacy were verifiable during IBR system data collection, structural adequacy required core samples. To determine the accuracy of ratings, the project team compared their field measurements with the assigned structural adequacy assessments (which are measurement ranges) that relied on local institutional knowledge. When actual measurements were within the gravel thickness bin range of the consensus assessment for structural adequacy, they were considered a match. Measurements outside of the bin range were considered errors, with the error amount being calculated as: actual gravel thickness – upper/lower bin range = error amount. Gravel thickness data matched the bin ranges selected by local agency staff 79.6% of the time; in the 20.4% of thickness measurements that were not exact matches, raters were more likely to overestimate gravel thickness (Figure 4).

Feedback from Users

Participants provided 72 comments during training, data collection, and the post-collection conference call meeting. After removing repeat comments, 63 comments were unique: 13 comments referred to the software tools, 37 comments pertained to the IBR system, three comments involved the training materials, and 10 comments addressed miscellaneous issues.

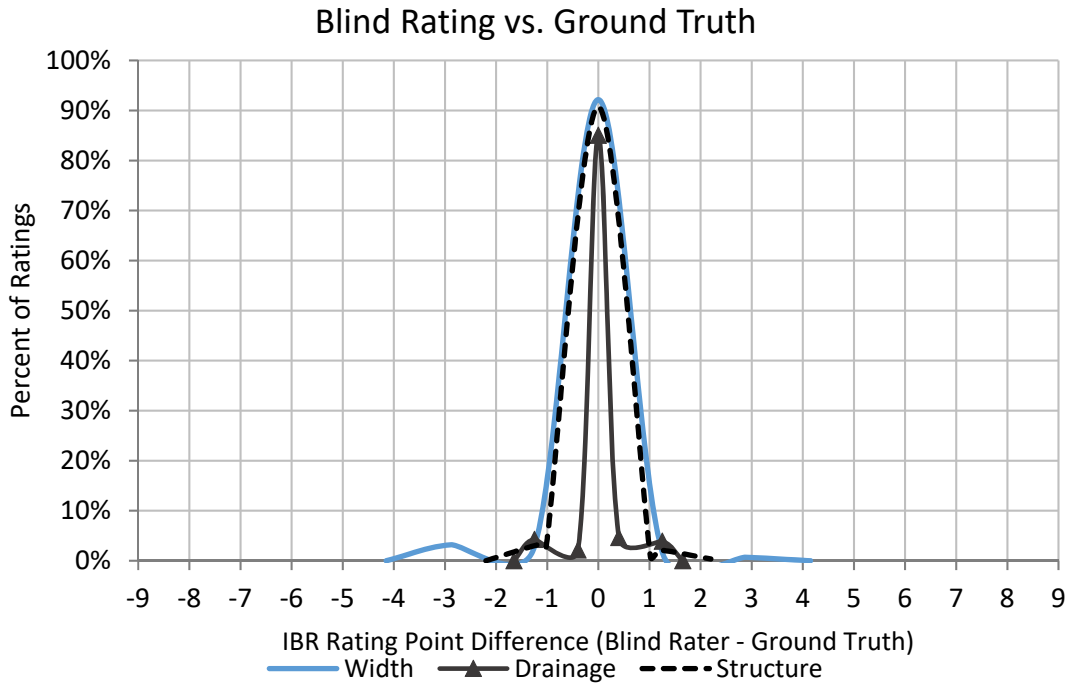


FIGURE 3 IBR element point difference (rater minus ground truth).

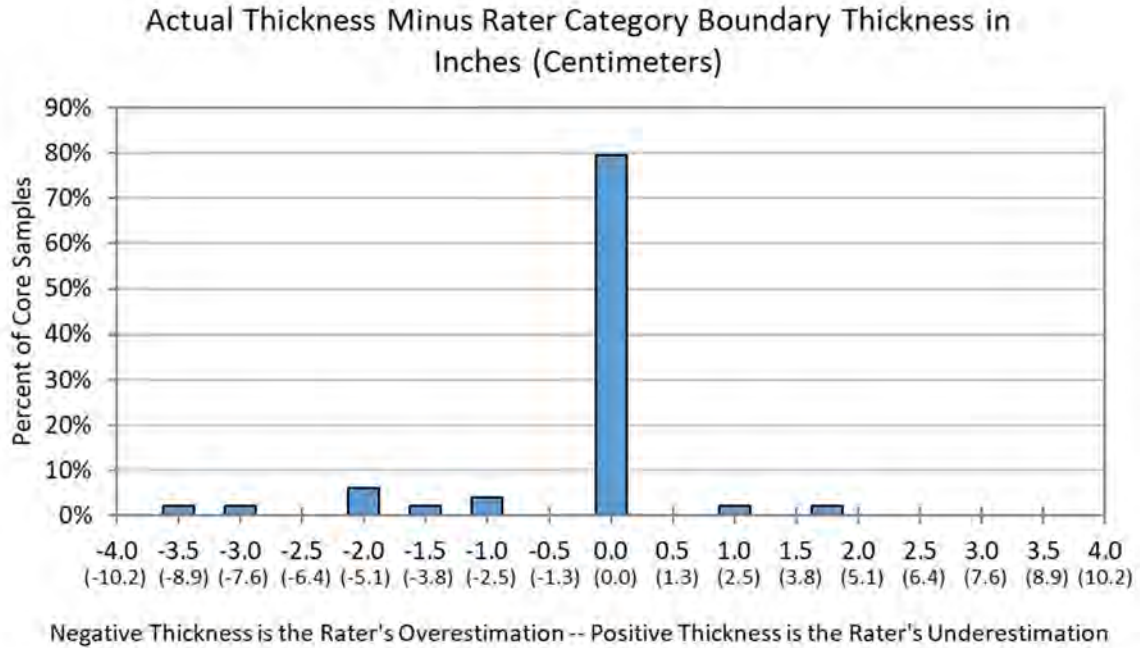


FIGURE 4 Validation of the institutional knowledge on gravel thickness.

CONCLUSIONS

Recommendation for Modification of the Original System

Overall, user feedback was positive and helpful. Of the 63 unique comments received, 37 comments resulted in modification of the IBR system, training materials, and/or software. Twenty-six comments were not addressable.

Modifications to the Drainage Adequacy Measurement Rating Guidance

The original IBR system dictated an average assessment for drainage adequacy, that is, fair for a road where one side is good and the other is poor. Antrim County Engineer Burt Thompson suggested rating only the worst side of the road when conditions on each side differ. The revised IBR system, thus, simplifies drainage adequacy to depend upon the rating for the worst side.

Modifications to the Structural Adequacy Measurement Rating Guidance

The original IBR provided structural adequacy guidance that was noncontiguous: roads with good structural adequacy have 8 in. (20 cm) or more of gravel and roads with fair structure adequacy have 4 to 7 in. (10 to 18 cm). The IBR system now has a revised criterion for good structure adequacy that eliminates this gap by changing the range to greater than 7 in. (18 cm).

Several agencies suggested modifying the range of gravel thickness based on their practices. The AASHTO Design Catalogs (7, 10) recommend aggregate base thicknesses for unpaved roads that have not had in-depth road design analyses. These recommendations depend on United States Climatic Regions. Since Michigan is in Climatic Region III, the recommended aggregate thickness ranges from 6 to 17 in. (15 to 43 cm) depending on traffic volume and subgrade quality. Most of AASHTO's recommendations for aggregate thicknesses are close or exceed the good IBR range. Therefore, the good structural adequacy range for the IBR system was not modified.

Concerns over the Intent of Good–Fair–Poor Designations

The good–fair–poor designations intuitively gauge inventory features relative to a baseline condition. Those designations were neither an indictment nor endorsement of a network's condition. However, local agencies expressed concern over the stigma associated with good, fair, and poor designations. Similarly, when Michigan began PASER collection, agencies perceived a stigma associated with the rating scale, but that dissipated with education and experience. The CTT project team believes that IBR system education will alleviate these perceptions.

Repeatability/Reliability of the System

Repeatability of the IBR system was very high both on assessing IBR elements and determining an IBR number when comparing consensus data (control) to individual data (blind). Individual IBR numbers were identical to consensus IBR numbers 72.2% of the time and were within a tolerance of ± 1 rating point 92.9% of the time. In comparison, the best year for PASER agreement (2008) had only 48% exact matches between the control and rating teams and 86.6%

within a tolerance of ± 1 rating point. The drainage adequacy element had the lowest exact match percentage for blind assessments versus the control at 85.1%. The CTT project team believes that high vegetation on roadway shoulders negatively influenced the agreement on drainage adequacy assessments since obscured ditches could not be examined or measured during blind assessments. With regard to structural adequacy, the high accuracy for estimating gravel thickness using local knowledge and surface observations—79.6% of the time—illustrates that local agencies know the structure of their unpaved roads.

RECOMMENDATIONS FOR IMPLEMENTATION

Based on this pilot data collection, the following recommendations can be made for agencies implementing the IBR system. First, IBR data should be collected when roadside vegetation is short, which increases the repeatability of drainage adequacy assessments. Second, initially collected IBR data should be updated after any road project that changes the IBR elements, such as ditching, widening, or adding gravel. This method would tie investment reporting to updating IBR data, which is easily recorded in Roadsoft as projects are completed. Third, re-rating the entire network should occur on a 3- or 4-year cycle to detect changing conditions outside of construction projects, such as changes due to loss of gravel or ditch sedimentation. Fourth, IBR data and paved road condition data (e.g., PASER) should be collected concurrently to improve efficiency of collection.

ACKNOWLEDGMENT

The authors would like to acknowledge the staff assisting in the field collection of data for this report: Kevin Arndt, Bruce Holdwick, and Mark Piotter, PE, of Huron County Road Commission; Megan Arndt and Fred Nagler, PE, of Kalamazoo Area Transportation Study; Ken Barnett, Craig Ericksen, Joel Hoort, and Linnea Rader of Van Buren County Road Commission; Travis Bartholomew and Bill DeYoung of Road Commission of Kalamazoo County; Bob Fitzgerald and Burt Thompson, PE, of Antrim County Road Commission; Doug Mills, PE, and Scott Swanson of Baraga County Road Commission; Gautam Mani of Southwest Michigan Planning Commission; Lynne Parker of East Michigan Council of Governments; Jeffrey Rautiola, PE, of Michigan Department of Transportation; and Steve Rouser of Western Upper Peninsula Planning & Development Region. We also thank Anne Dahlquist, Luke Peterson, and Sean Thorpe of Michigan Technological University for participating in the data collection.

The authors also would like to acknowledge the significant work that went into the development of the IBR system by Melanie Kueber-Watkins from Michigan Technological University; that work is outlined in the report *Inventory Based Assessment Systems for Unpaved Roads*, by Colling and Watkins (5). Finally, the authors would like to recognize both the Michigan TAMC for their support and forward thinking in addressing unpaved roads assessment issues, and Victoria Sage from the CTT at Michigan Technological University for assistance with preparing this paper.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the paper as follows: study conception and design: all authors; data collection: all authors; analysis and interpretation of results: all authors; draft manuscript preparation: all authors. All authors reviewed the results and approved the final version of the manuscript.

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Additional Resource

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Pavement Management Tools to Economically Evaluate Interventions on Unpaved Roads

PEDRO GUILHERME PINHEIRO SANTOS FERNANDES

LUIZ ANTÔNIO FERREIRA DE SANTANA

ERNESTO FERREIRA NOBRE JÚNIOR

Federal University of Ceará

ALINE CALHEIROS ESPÍNDOLA

Federal University of Alagoas

In Brazil, unpaved roads play an important role in rural communities, linking them to health and education facilities as well as transporting their farming production to nearby cities. For this reason, public agencies need to manage and preserve this infrastructure in order to ensure satisfactory accessibility conditions for these communities. Unpaved roads are generally managed by municipal infrastructure divisions, which do not have a pavement management system (PMS) or even specialized employees. Therefore, the Federal University of Ceará developed methods to assess and classify surface conditions of unpaved roads. The usage of this tools meets the need of PMS for small and medium-sized towns, including maintenance reference costs to unpaved roads. Consequently, this paper has the objective of presenting some steps of the proposed PMS methodologies, including a budgeting tool to quantify the maintenance strategies cost and also to evaluate technical and economic feasibility of these interventions. For the unit costs development, manpower, equipment, materials prices and distresses severity levels were considered for each maintenance intervention. The application of the methods was validated at the road segment of São Brás–Olho D'água, presenting satisfactory and consistent budget values. An economic analysis simulating the evolution of pavement conditions using ALYNOMO method was also performed.

INTRODUCTION

Unpaved roads play a significant role in Brazilian rural economy (1). In developing countries, these roads are generally the key logistic infrastructure of rural areas, connecting family farming production to city areas and allowing rural communities access to health and education facilities (2). According to CNT (Confederação Nacional do Transporte), 78.5% of Brazilian road network is composed of unpaved roads, corresponding to an extension of 1,349,939 km (3). Therefore, this infrastructure network needs to be efficiently managed and preserved in order to guarantee safety and wellness of rural communities (4).

Brazilian unpaved roads are generally managed by municipal infrastructure divisions or state's highway departments (DERs), and, in a few cases, by the National Department of Transportation Infrastructure (DNIT). As an example, Ceará state (CE) has 0.64% of its unpaved road network managed by DNIT, 6.52% by the DER–CE and 68.37% by municipalities. In general, infrastructure divisions of small and medium-sized cities are not composed of enough

employees who are specialized on road maintenance, so many of the resources are mismanaged or unapplied (5). As the pavement is continuously damaged by rains and traffic, unpaved roads are directly affected by the lack of investments, having an impact on rural population who see their accessibility increasingly limited (6).

Moreira argues that a possible solution to the maintenance problems of unpaved roads is the employment of a PMS that not only performs emergency conservation activities, but also preventive ones. Moreira also discuss the need for a management system which presents a lifetime and economic evaluation including several possibilities of maintenance. Moreira states the system should be simple due to the technical limitations presented by employees and managers of Brazilian municipal infrastructure divisions (7). On account of this, universities and agencies have been developing studies and methodologies related to PMS implementation in developing countries like Brazil (8).

As an example, Queiroz proposed a PMS system to middle-income countries (9), as well as Giné (10) who developed a sustainable management system for rural road networks. According to Queiroz, it is possible to implement a low-cost PMS, since methods and procedures have already been properly created or reviewed considering less-sophisticated equipment for pavement assessment. Another important issue in a PMS implementation is the usage of a realistic strategy analysis (9). Plessis-Fraissard says that pavement assessment and economic analysis of maintenance interventions must be based on reliable local data, allowing construction of consistent pavement degradation models, as well as financial impact models, assessing costs for road users and agencies (11). Therefore, a PMS compatible with developing countries must consolidate three points in the implementation phase: standardization of data collection procedures; consistent pavement degradation models; and reliable long-term economic analysis of distinct management scenarios.

In the context of CE, engineers and researchers have been developing a PMS implementation proposal for unpaved roads. It was divided in five stages, as shown in Figure 1. First, Silva developed a methodology for collecting and analyzing field data correlated with defects of the unpaved road (1). Second, Almeida settled prioritization criterion of interventions (12). After that, Moreira et al. created a rolling surface evaluation model considering topographic, geotechnical, and climatic aspects (7).

The last stage of the PMS is the development of an economic analysis methodology which evaluates different maintenance scenarios, considering the severity of each intervention. However, the current lack of a budgeting reference or cost accounting handbook including maintenance costs of unpaved roads might compromise the construction of a reliable analysis. For this reason, this study proposes to create a cost catalog.

A cost catalog is essentially a handbook with all maintenance prices. The idea is to create a unit cost for all available maintenance strategies in CE, considering type and severity of

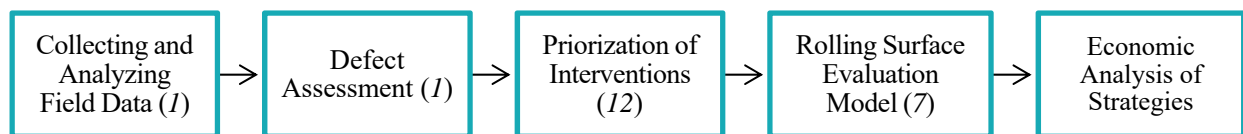


FIGURE 1 Structure of unpaved road simplified PMS for CE.

distresses. For instance, if a manager were to identify a specific road extension with low severity potholes, he would find platform leveling as maintenance strategy and would find a price per hectare for it. Thus, it is possible to price maintenance and rehabilitation processes, providing costs of PMS strategies.

OBJECTIVES AND SCOPE

The main objective of this work was to provide an economic analysis tool for managers, with the ability to perform financial quantification of maintenance strategies as well as evaluation of technical and economic feasibilities of intervention processes to which an unpaved road is submitted. Having a goal to develop a cost catalog, the present work proposed to develop unitary cost compositions for each intervention. Later, the catalog was validated in a proposed case study, being submitted to evaluations on defect assessment, rolling surface conditions, and, lastly, to an 18-month net present value (NPL) economic analysis.

PAVEMENT MANAGEMENT SYSTEM FOR UNPAVED ROADS LOCATED IN CEARÁ

Pavement Management System Development

Before the case study section, it is necessary to understand how the predecessor stages were developed and proposed. Following Queiroz (9), Silva created simple and low-cost tools to access pavement conditions. Silva prioritized the evaluation of three types of distresses: improper cross-section, inadequate road side drainage, and ruts (1).

Silva sought to find a parameter that indicated how severe the distresses of improper cross-section and inadequate road side drainage would be. Consequently, he decided to use the slope of the most rugged regions. Silva also created intervals according to the slope, allowing him to classify distress severity in low, medium, or high.

For the field data collection, Silva developed two equipments, the transverse irregularity meter (MIT) and the longitudinal declivity meter (MDL) (1) (Figure 2). The MIT is composed of an aluminum bar with small measuring rods and the MDL is composed of two measuring rods connected by a level hose. With these tools Silva could plot transversal and longitudinal sections for every 5 m, assigning a severity level to each measured extension. At last, Silva improved the measuring methodology for ruts of Eaton and Beaucham (13), using a truss, as shown in Figure 2.

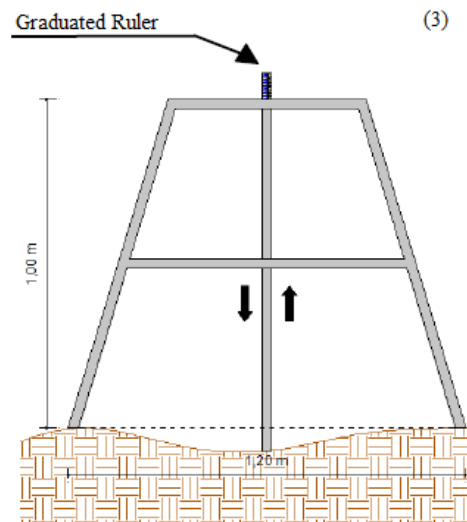
After Silva's development of data collection and distress assessment, Almeida created a prioritization methodology based on analytic hierarchy process (AHP). Firstly, Almeida determined which maintenance procedures would be used in each situation, considering the distress type and severity. Secondly, he developed a multicriteria model of prioritization based on AHP, taking into consideration physical, climatic, traffic, management, and social aspects that influence the road functioning. As a result, he developed different hierarchy levels and applied the management technique in the municipal district of Aquiraz. In this study, only the maintenance procedures of Almeida were considered, once the study case considered a relatively small extension (12). Therefore, it did not require the usage of a complex management tool for maintenance prioritization.



(a)



(b)



(c)

FIGURE 2 Equipment used for field data collection: (a) MIT; (b) MDL; and (c) truss.

Moreira presented one of the most important and innovative stages of the PMS proposed to the unpaved roads of CE. He developed a model for assessing surface conditions in unpaved roads, proposing an approach which focuses on intertemperate weather influences (e.g., rain, wind), traffic and soil type (7). Moreira's study incorporated the assessment procedures of Silva (1) and Eaton and Beaucham (13), creating an evolution model of distress present in unpaved roads, the ALYMONO method (7). With this evolution model, it is possible to estimate the pavement conditions after a certain period. Thus, it is possible to compare distinct maintenance scenarios and observe their NPL values.

ALYMONO Method

In this study, the rolling surface conditions were evaluated by the ALYNOMO method. The intention of this analysis is to create a model of the pavement performance, comparing indexes of pavement usefulness in different time periods.

The ALYMONO method assess usefulness conditions of unpaved roads considering topographic and geotechnical aspects, using a value of usefulness, called index of the usefulness (IS) (7). This index is calculated following nine steps.

1. The road extension is divided in geotechnical zones (GZs) based on the geotechnical classification method of the Highway Research Board (HRB).
2. The GZs are subdivided in topographical zones (TZs). The TZ subdivision is based on the sharp variation of longitudinal ramps, featuring a single ramp value for each TZ, accepting little discrepancy.
3. Each distress found are georeferenced and classified according to type. After that, the distress area or length is calculated by counting 1-cm spaced contour lines. Depending on these calculated values, it is adopted a different individual level of severity (LIS) for each distress. The LIS values for low, medium, and high severity, correspond to 1, 2, and 3, respectively.
4. Once every value of LIS is obtained, it is calculated the average value of LIS for each distress located in a specific TZ. In other words, it is calculated the average severity (AS) in a topographical zone.
5. After AS calculation, it is time to find relative surface (RS) density of each distress in a topographical zone. The RS is the value of the sum of the areas or the lengths of each distress type divided by the respective TZ total area or total length.
6. With AS and RS results, it is calculated the index of relative usefulness (ISR) by topographical zone which is AS and RS multiplied for each type of defect.
7. Once each ISR is found for each distress, it is obtained a value called IC which corresponds to a reference value of TZ condition, being the higher value of ISR.
8. Once the IC value is inversely proportional to the pavement usefulness, it is used a value called IS which corresponds to $IS = 1 - IC$, ranging from 0 to 3. The IS is the reference index of usefulness of ALYNOMO method, being used to classify unpaved roads rolling surface conditions. The classification ranges are: 0.000 to 0.199 for poor conditions; 0.200 to 0.649 for very bad conditions; 0.650 to 1.099 for bad conditions; 1.100 to 1.599 for regular conditions; 1.600 to 2.199 for good conditions; and 2.200 to 3.000 for excellent conditions.
9. With the IS values and associated period of each analysis, it is possible to fit a curve and then develop predictive modeling function represented by quadratic equation ($ax^2 + bx + c$).

CASE STUDY

Developing the Cost Catalog

As mentioned in the introduction, a cost catalog is a handbook with all maintenance prices. Consequently, this study created a unit cost for all maintenance activities, considering type and severity of distresses. For the development of unit costs, 13 types of unpaved roads pavement distresses were considered based on the *Unsurfaced Road Maintenance Management Special*

Report of Eaton and Beaucham (13) and the Brazilian Studies of Oda (14), IPT (15), and Almeida (12). The distresses are listed below and shown in Figure 3.

1. Improper cross-section;
2. Inadequate road side drainage;
3. Corrugations;
4. Dust;
5. Potholes;
6. Ruts;
7. Loose aggregates;
8. Sandy ground;
9. Slippery track;
10. Formation of mud;



FIGURE 3 Examples of pavement defects.

11. Erosions;
12. Heavy aggregate dispersion;
13. Rock outcropping.

In order to price the maintenance activities, resource, manpower and construction material costs were considered. These costs were acquired from the current Brazilian construction budgeting manual, *Referential System of Costs* (SICRO). Due to the lack of information in SICRO system about operations and materials included in an unpaved road, some prices were obtained from National System of Construction Indexes and Prices (SINAPI), focused mainly residential construction.

For all maintenance activities, three types of employees are required, each one with a distinct specialization—instruction level and remuneration. Based on DNIT, the additional 5% of working tools used in the service must be added to the manpower cost (16). Table 1 shows the hourly wage of all labor categories.

According to DNIT, different types of equipment can supply execution necessities of all described interventions, such as trucks, loaders and compactors. Consequently, 10 construction equipments were included in cost compositions, following the execution recommendations of DNIT (4). Table 2 presents a list of equipment and the productive and unproductive costs, based on DNIT (16).

Many of the material prices for maintenance execution were obtained of SINAPI system by Caixa Econômica Federal (17), except mortar and concrete which were based on DNIT (16). Table 3 shows the price per unit of each material.

TABLE 1 Manpower Remuneration

Manpower	Remuneration (R\$/h)
Construction helper	15,94
Team leader	26,71
General laborer	21,34

TABLE 2 Equipment Productive and Unproductive Costs

Equipment	Cost	
	Productive	Unproductive
5-m ³ dump truck, 136 kW	R\$137.45	R\$43.31
20-t.m crane truck, 136 kW	R\$172.68	R\$73.29
15-t truck, 188 kW	R\$174.80	R\$47.64
10.000 l water tank truck, 188 kW	R\$181.39	R\$51.29
8.000 l water tank truck, 136 kW	R\$144.09	R\$47.03
3.3-m ³ loader, 213 kW	R\$326.51	R\$127.90
Vibratory plate compactor, 3 HP	R\$3.04	R\$0.80
Motor grader, 93 kW	R\$182.05	R\$80.01
11.6-t self-propelled vibration sheep's foot roller, 82 kW	R\$123.22	R\$55.44
Track-type tractor with blade, 112 kW	R\$184.66	R\$74.90

TABLE 3 Material Costs

Material	Unit	Price
Gravelly river sand	m ³	R\$55.00
Cement and sand mortar 1:4	m ³	R\$201.09
Cyclopean concrete fck = 20 MPa	m ³	R\$217.53
Simple concrete pipe d = 600 mm	m ³	R\$65.09
Excavation drill	un.	R\$33.15
Form boards (timber)	m ²	R\$57.51
Cobbles	un.	R\$49.89
Concrete porous pipe	m	R\$23.44

In Brazilian public projects, there is an increment in the total cost, known as the benefits and indirect costs rate (BDI). It is an addition to the direct service costs, covering financial expenses with equipment mobilization and demobilization, assembly and dismantling services, complementary technical services, administrative expenses, risks and contingencies, profit, and government taxes. The Brazilian Court of Auditors (TCU) sets the acceptable ranges of BDI rates for public construction. The BDI range for road construction is 19.60% to 24.23%; the average value of 20.97% were applied in this study (18).

After defining the manpower, equipment and materials costs, and the BDI rate, the unit costs were built up as shown in Figure 4.

COMPOSITION OF UNIT COST							UNID. M3
SERVICE: Erosion - Mechanical Recomposition							
EQUIPMENT	QUANTITY	UTILIZATION		COST		COST PER HOUR	
		PRODUCT.	UNPRODUCT.	PRODUCT.	UNPRODUCT.		
Track-type tractor w th blade - 112 kW	1,00	1,0000	0,0000	184,88	74,90	184,88	
Tire loader with capacity of 3.3 mf - 213kW	1,00	1,0000	0,0000	326,51	127,90	326,51	
Dump truck w th a capacity of 5 mf - 136 kW	1,34	1,0000	0,0000	137,45	43,31	184,19	
Motor grader - 98 kW	1,00	0,2400	0,7600	182,05	80,01	104,50	
Sheeps foot roller w th self-propelled vibration of 111.6 t - 82 kW	1,00	1,0000	0,0000	123,22	56,44	123,22	
A) TOTAL - EQUIPMENT						923,08	
MANPOWER				QUANT.	HOURLY WAGE	COST PER HOUR	
Servant				1,00	15,9400	15,94	
Team Leader				0,10	26,7069	2,67	
Additional tools employed (5.0%)						0,93	
B) TOTAL - MANPOWER						19,54	
C) MATERIAL		UNID.	COST	QUANTITY	UNIT COST		
COST PER HOUR OF MATERIAL =>						0,00	
D) TEAM PRODUCTION:		15,00	TOTAL COST PER HOUR (A + B + C)			942,62	
E) IMPLEMENTATION UNIT COST					(A+B+C) / D=>		62,84
BONUS (BDI)		=>	20,97%			13,18	
TOTAL UNIT COST						76,02	
OBSERVATION:							

FIGURE 4 Example of unit cost table for unpaved roads.

In Figure 4, it is possible to observe how a unit cost is calculated. Following Almeida (12) and DNIT (16) utilization and production values for machinery, it is set the necessary quantity of equipment as well as the utilization percentage. Thus, the values of quantity, utilization, and productive and unproductive costs are multiplied, being defined as each equipment's cost-per-hour. After that, all equipment costs-per-hour were summed, resulting in the total equipment cost. A similar process occurs with the manpower and material cost. Later, the total unit cost is obtained. Otherwise speaking, the equipment (A), manpower (B), and material (C) total costs are summed, divided by the team production (D) and multiplied by the BDI rate. The team production was previously calculated based on the DNIT manual (16).

The same process was done for all maintenance activities, considering all associated distresses and severity levels. Table 4 shows the summary of the cost catalog. The distress column follows the list of 13 types of unpaved roads pavement distresses previously mentioned.

Budgeting Process and Economic Analysis

The study area is 3.84-km long situated between the municipalities of São Brás and Olho D'água Grande. The analyzed region is characterized by high temperatures, low rainfall frequency, and no percolation or infiltration problems, presenting two different GZs subdivided in three distinct TZs, as shown in Figure 5.

In this case study, the rolling surface conditions were monthly evaluated, beginning from the last maintenance. It was a 12-month monitoring activity based on Silva (1), Almeida (12), and Moreira (7) methods. Three main defects were observed and quantified—corrugations, loose aggregate, and improper cross-sections. All found distresses were classified as medium severity, totalizing a 1,547-m-long extension.

In the twelfth monitoring month, the maintenance activities were budgeted, as presented in Table 5. For the total volume calculation, a 7-m-long average track width and 0.2-m depth were considered.

After the previous budgeting process, this study proposed a simulated economic analysis, presenting how a municipal infrastructure division can utilize a cost catalog in a PMS and compare different maintenance strategies, analyzing usefulness and economic aspects. The first step was modeling the pavement usefulness evolution. Figure 6 shows the curve-fitting functions of each TZ.



FIGURE 5 Linear representation of the road extension (in Brazilian “estacas” = 20 m). The triangles represent the initial and final point of GZs and the circle represents the division point between TZ 1 and TZ 2.

TABLE 4 Summary of the Proposed Cost Catalog

Distress (Severity)	Recommended Maintenance Activities	Unit	Unit Value (RS)	Distress (Severity)	Recommended Maintenance Activities	Unit	Unit Value (RS)
1 (low)	Platform leveling	ha	243,87	1 (medium)	Platform leveling; granular material addition; water addition; homogenization; compaction	m ³	18,57
3 (low)				3 (medium)			
6 (low)				6 (medium)			
7 (low)				7 (medium)			
10 (low)				9 (low, medium, and high)			
12 (low and medium)				10 (medium)			
1 (high)	Platform cutting; granular material addition; water addition; homogenization; compaction	m ³	21,73	2 (high)	Reconstruction of ditches; deep drain installation; increase of manholes; rip rap protection	m	17,36
3 (high)							
4 (medium and high)							
6 (high)							
7 (high)							
12 (high)							
2 (low)	Ditch cleaning	m	8,44	2 (medium)	Manhole cleaning	m	23,64
4 (low)	Water addition	T.km	1,64	8 (low)	Platform leveling; execution of primary coating	m ³	11,27
8 (medium to and high)	Greide elevation; primary coating	m ³	12,74	10 (medium and high)	Construction of manhole	m	1033,8
					Greide elevation	m ³	18,57
11 (low, medium, and high)	Reconstruction	m ³	606,63 76,02	13 (low, medium, and high)	Reconstruction	m ³	10,64

TABLE 5 Distresses Location and Maintenance Budget

Distresses Location														
Corrugations (low)				Loose Aggregate (medium)				Improper Cross-Section (medium)						
Chainage (20 m)				Ext. (m)	Chainage (20 m)				Ext. (m)	Chainage (20 m)				Ext. (m)
Start	End	Start	End		Start	End	Start	End						
5	+ 0	8	+ 15	75	11	+ 0	13	+ 0	40.0	19	+ 0	39	+ 0	400.0
104	+ 0	108	+ 0	80	88	+ 18	91	+ 15	57.0	59	+ 0	88	+ 10	590.0
					105	+ 0	107	+ 0	40.0	184	+ 15	197	+ 0	245.0
					152	+ 5	153	+ 5	20.0					

Maintenance Budget				
Interventions	Unit	Quantity	Unit Value	Total Value
Corrugations (medium)	m ³	217,00	R\$18,57	R\$4.029,69
Loose aggregate (medium)	m ³	219,80	R\$18,57	R\$4.081,69
Improper cross-section (medium)	m ³	1729,00	R\$18,57	R\$32.107,53
Total				R\$40.218,91

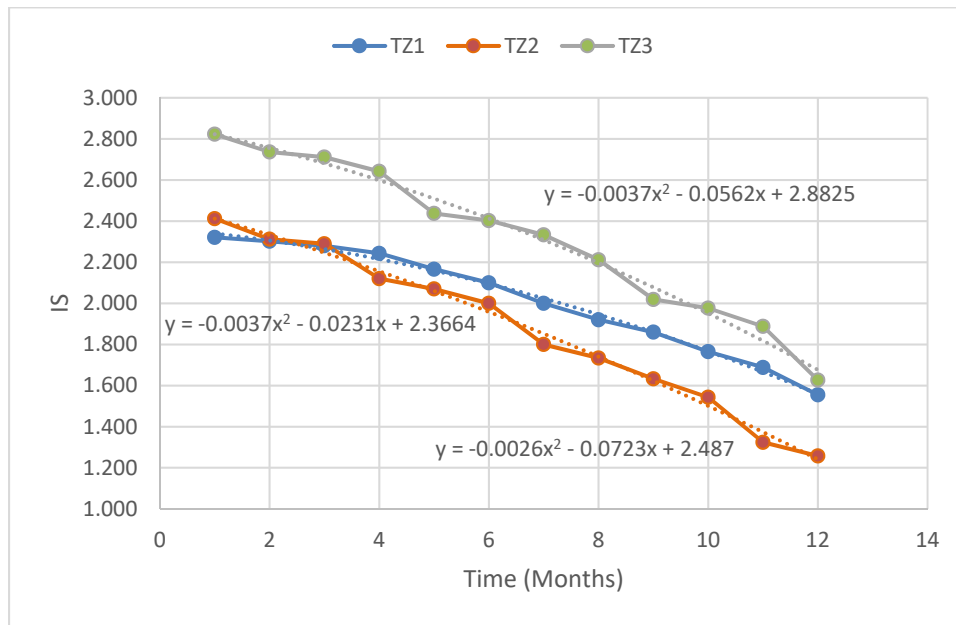


FIGURE 6 Predictive modeling of performance (ALYNOMO method) of TZs.

After the curve-fitting, three model equations were found, as shown below.

$$y = -0,0037x^2 - 0,0562x + 2,8825 \tag{1}$$

$$y = -0,0037x^2 - 0,0231x + 2,3664 \tag{2}$$

$$y = -0,0026x^2 - 0,0723x + 2,4867 \tag{3}$$

In this economic analysis, two maintenance cycles were analyzed: A 12- and an 18-month cycle. Therefore, the predictive model was used to each situation, observing the final rolling surface condition for each TZ. Figure 7 shows two types of curves. The first one presents the effect of a maintenance in the twelfth month; the second type presents the effect of no maintenance over months.

It is possible to see that the surface condition would be bad or very bad for TZs in the moment of maintenance in the 18-month cycle. Due to this condition, it is very likely that the maintenance intervention would be done for a high-severity condition. However, it is difficult to predict the increment of distress occurrence and extension. Consequently, this study proposed that the distress extension would be increased for the 18-month cycle. As a result, four distinct scenarios were analyzed: 12-month maintenance cycle; 12-month maintenance cycle with 20% growth of distress extension; 12-month maintenance cycle with 30% growth; and 12-month maintenance cycle with 50% growth.

This economic analysis was based on a net present value (NPL) evaluation, considering a direct benefit of R\$2000,00 per semester when the analyzed road had regular or better rolling surface condition. The analysis was done for a period of 3 years, utilizing an interest rate of 6.5% based on the Brazilian reference rate, called SELIC (19). Table 6 shows all budgeted scenarios and the respective net present value of each situation, considering the maintenance of 18-months scenarios as a high-severity level maintenance.

The 12-month maintenance cycle had the best NPL, representing an investment of R\$84,603,82 which means that the first maintenance scenario is the most economical and efficient in terms of usefulness and performance. With this case study, it is evident that the cost catalog can be used both as a budgeting reference and as an economic tool, playing a crucial role in a PMS system.

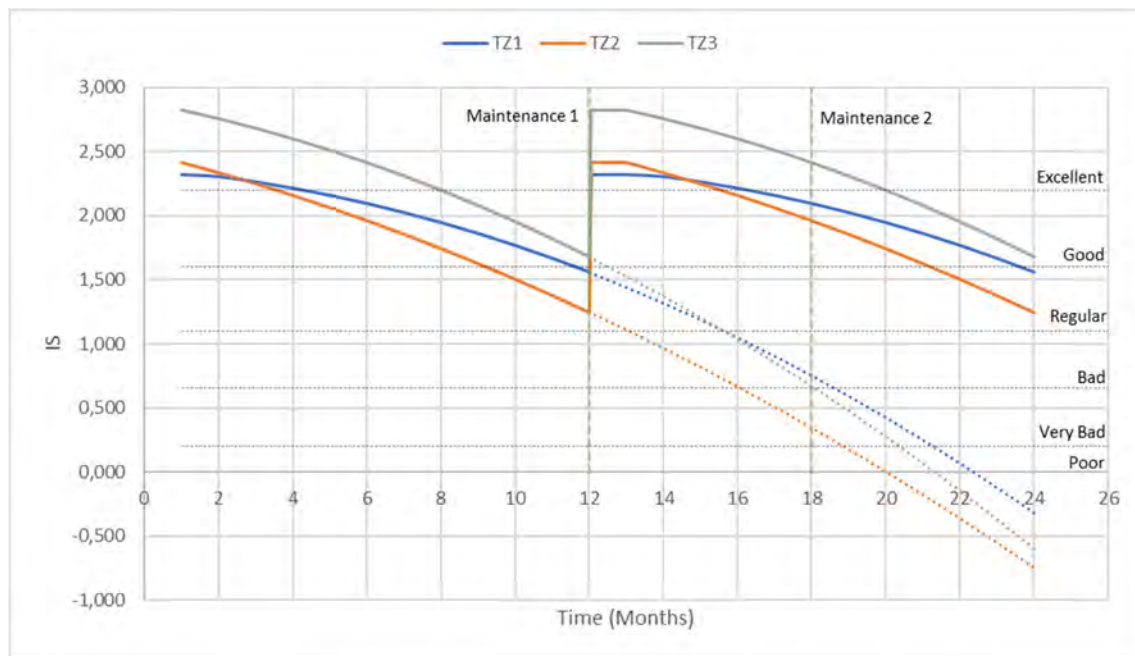


FIGURE 7 Predictive modeling of performance (ALYNOMO method) for distinct situations.

TABLE 6 Maintenance and net present value for proposed scenarios.

Scenario	Maintenance Total Cost	NPL
12 month	R\$40,218.91	-R\$84,603,82
18 month (20% growth)	R\$56,475.40	-R\$82,025.05
18 month (30% growth)	R\$61,181.68	-R\$89,146.52
18 month (50% growth)	R\$70,594.25	-R\$103,389.45

CONCLUSIONS

The development of a cost catalog and an economic analysis methodology is essential to implementation of a PMS. That is why this study presented a significant contribution for unpaved roads management in Brazil. The cost catalog is easy to apply and understand once all proposed maintenance activities present a cost per unit. Consequently, managers can easily employ this budgeting tool in municipal infrastructure divisions.

Although the case study did not consider users and agencies economic benefits, it was possible to observe the importance of preventive management. Thus, the next step in the development of a simple PMS for the state of CE is the creation of a model which predicts the economic and social benefits related to good performances of unpaved roads. Another aspect to be analyzed is the evolution of distresses severity for different geotechnical and climate conditions.

ACKNOWLEDGMENTS

The authors acknowledge CAPES (Coordenação de Aperfeiçoamento de Pessoal de Nível Superior) and CNPq (Conselho Nacional de Desenvolvimento Científico e Tecnológico) for supporting this research.

The authors confirm contribution to the paper as follows: Study Conception and Design: Espíndola, Fernandes, and Santana; Data Collection: Santana; Analysis and Interpretation of Results: Fernandes, Espíndola, Santana, and Nobre; Draft Manuscript Preparation: Fernandes, Espíndola, Santana, and Nobre; Article Revision: Fernandes. All authors reviewed the results and approved the final version of the manuscript.

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Development and Application of a Sustainable Management System for Unpaved Rural Road Networks

ALONDRA CHAMORRO GINE
Pontificia Universidad Catolica de Chile

SUSAN TIGHE
University of Waterloo

For the sustainable management of rural roads, the social, institutional, technical, economic and environmental aspects should be considered under a long-term perspective. The current practice in developing countries is that only some of these key sustainable aspects are considered in the management process. In addition, rural roads maintenance management is commonly performed under a short-term basis, not considering the life-cycle costs and benefits in the economic analysis and project prioritization. This paper presents the development of a sustainable management system for rural road networks and its application in developing countries. The approach considers the development of a sustainable framework; application of a network-level condition evaluation methodology; condition performance models for gravel and earth roads; cost-effective maintenance standards; a long-term prioritization procedure that accounts for sustainable aspects; and a computer tool that integrates the system components. The management system has been applied and validated in two unpaved rural road networks in the developing countries of Chile and Paraguay. Sensitivity analysis was carried out to assess the impacts of input parameters in the performance of developed system. As a result of the research an adaptable and adoptable sustainable management system for rural networks has been developed to assist local road agencies in developing countries.

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A Pragmatic Approach for Dust Monitoring on Unsealed Roads

MARK ALLAN

THEUNIS F. P. HENNING

MARK ANDREWS

The University of Auckland

LARS FORSLÖF

Roadroid

Unsealed roads remain the backbone of the economy for many countries around the world. Controlling dust on these roads requires useful and cost-effective monitoring. As existing surveying methods are expensive and the condition of unsealed roads can change rapidly, there is a need for cost-effective options to survey these roads. This paper introduces a low-cost qualitative methodology to estimate dust emissions from unsealed roads using automated analysis of photographs taken from camera-equipped smartphones. Visual indicators extracted from photographs can be analyzed using machine-learning techniques to classify dust severity. Image analysis cannot provide information on dust particles directly since the particles' dimensions are much smaller than a single image pixel. However, dust leaves a fingerprint in the image that acts as a proxy for the combined size and density of the particles. Indicators of road dust in an image include attenuation of high spatial frequencies, reduced saturation, and reduced color gamut. These effects are caused by the scattering and diffraction of light by dust. Challenges to accurate classification include image blur, large image regions containing only light dust, and regions of sky and road within the image. Image preprocessing, classification techniques that assess dust intensity within dusty regions and texture analysis will mitigate these issues. The research concluded that the image analysis methods are showing strong potential for their intended purpose. It demonstrated that a number of techniques could be utilized for the image analyzes, each having strong points and limitations within specific operating environments.

INTRODUCTION

Unsealed roads remain the backbone of the economy for many countries around the world. Despite being regarded as marginal, unsealed roads are the starting points for agricultural products, tourism, forestry harvest, and mining industries. A common complaint from road users and adjacent residents and landowners is the roughness and dustiness of these roads. This paper investigates a cost-effective alternative for assessing the dustiness of unsealed roads.

Unsealed roads release dust into the air when driven over by vehicles. Road dust has a variety of negative effects, including health risks due to increased fine particles, air pollution, nuisance effects on adjacent properties, road safety risks due to reduced visibility, and loss of fine road aggregates. The measurement of dust emissions can be undertaken either by the use of stationary detection devices or by mobile dust-monitoring systems such as the Traker System (1).

A common issue with condition surveys on unsealed roads is that they can be expensive and road conditions can change rapidly, resulting in a poor return on the survey investment. There is therefore an increasing need for more cost-effective options to survey these roads.

The objective of this paper is to introduce a low-cost qualitative methodology to estimate dust emissions from unsealed roads using pixel analysis of photographs taken from camera-equipped smartphones. It investigates and describes the various methods of image-based analyses that could be used for this purpose.

METHODOLOGY SUMMARY OF DUST-MONITORING TECHNIQUES

This project investigates methods for the collection of road dust data through analysis of images taken from moving vehicles.

Existing equipment typically uses light interference patterns to measure the density of fine particles in the air. These devices are either used for stationary tests or fitted into a moving vehicle (2). These methods tend not to be practical for continuous data collection. Stationary measurements are only able to assess small parts of the network and mobile measurements using custom-fitted vehicles are not easily procured by road asset managers.

Survey photographs provide an alternative source of data on road dust. This research is developing a system that can analyze survey photographs taken from the rear window of a moving vehicle, and classify the severity of dust emissions in the image. The images are sourced from a mobile application that can be installed on a smartphone and used to automatically take photographs and gather road data during road surveys.

The image analysis system will have the following steps:

1. Acquisition of raw image data using smartphone cameras.
2. Screening and preprocessing of images to correct for differences in ambient lighting, or reject images with significant motion blur, etc.
3. Segmentation of each image into the background sky and relevant foreground.
4. Extraction of features indicating the presence of dust.
 - a. Histogram distribution of pixel intensities. Dust tends to decrease the contrast and increase average intensity of an image, giving a more “washed out” appearance.
 - b. Distribution of pixel colors. Dust attenuates most colors by scattering and absorption and reduces the color gamut of the image.
 - c. Attenuation of high spatial frequency components. The presence of dust tends to blur sharp edges, leading to a reduction in overall high spatial frequency content.
 - d. Visibility distance. Dust limits the maximum distance at which geometric features can be reliably discerned in an image and tracing the extent of dominant lines in the image can be used to estimate visibility.
5. Extracted features are input to a machine-learning system previously trained on manually labelled datasets.
6. Dust severity is classified.

The software provides a qualitative five-point assessment of dust severity, rather than a quantitative measure. The outputs are consistent with the Unsurfaced Road Condition Index (URCI) method (3) and have been validated against road survey data. Though less precise than

quantitative measurements of density, the low-cost and ease of use will make regular dust monitoring across an entire road network practical for road asset managers.

PRINCIPLES OF THE TECHNOLOGY

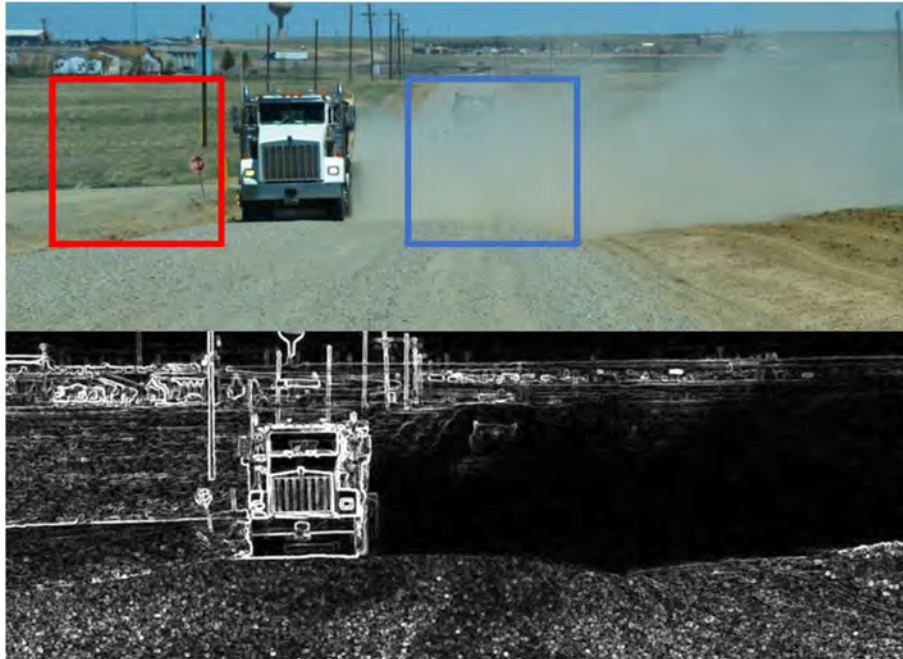
There are numerous methods for measuring the volumetric density and size of airborne particulate matter. These mainly laboratory-based—or at least stationary—methods range from simple mechanical sieves to laser light scattering systems (4). The challenge for the current project has been to develop a system that is mobile, inexpensive, resilient in a potentially harsh mechanical environment, and produces results that correlate with proven measurement techniques. By adopting an image analysis-based approach, we have leveraged the computing power and mechanical robustness developed by smartphone manufacturers and can concentrate solely on the signal processing problem. Furthermore, the prevalence of cheap smartphones makes it possible to crowdsource the task of surveying an extensive network of unpaved roads, and to also update the overall network evaluation more frequently than is currently possible.

An ideal mobile dust measurement system gives information on the distribution of atmospheric particle sizes and the density of the particles. Image analysis cannot provide this information directly since the particles' dimensions are much smaller than a single image pixel. However, when the density of dust particles is sufficiently large, it leaves a fingerprint in the collected images that acts as a proxy for the combined size and density of the particles (5). In order to calibrate our image-based approach against standard particle measurement methods, we simultaneously collect both image data and particle size data using low-cost laser-scattering sensors.

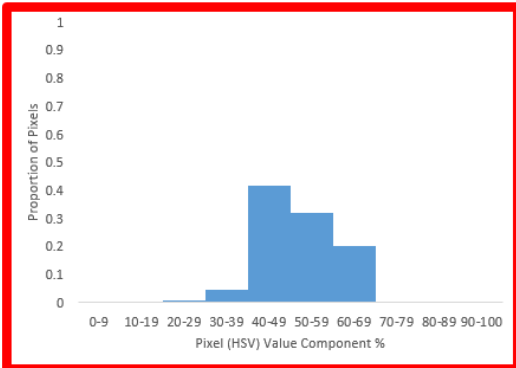
Figure 1 illustrates two techniques for extracting image features that act as proxies for the presence of image dust—spatial frequency content and pixel intensity distribution. The monochrome image demonstrates the lack of high spatial frequencies in regions containing dust. In non-dusty parts of the image, even relatively uniform areas still contain significant high-frequency detail. The histograms show that the image region obscured by dust exhibits lower variance in intensity and saturation, due to the scattering and diffraction of light by dust. The research further indicated that the efficacy of particular techniques depends on the environment and prevailing background visibility on a given day.

Various confounding factors became apparent as more widespread data collection progressed. The image features correlating with dust, mentioned above, also correlated with images that contain no dust (6). For example, a constrained color gamut may simply reflect a natural environment with subdued colors. This represents a challenge for any machine-learning system. Visual features found to make incorrect classification more likely are noted below, along with potential methods to mitigate these issues.

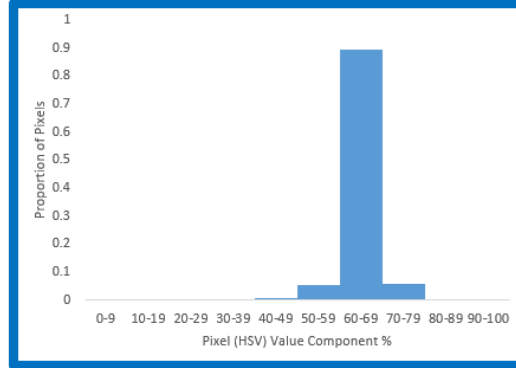
- Image defects in poor quality images such as blur or light glare.
 - These defects reduce edge intensity and reduce saturation, both indicators of road dust.
 - Provided that the smartphone is set up effectively, the ratings of poor images are likely to be outliers and so are unlikely to affect the practical effectiveness of visual methods.



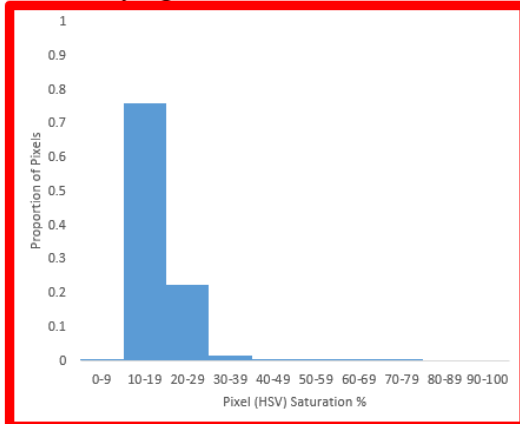
Non-Dusty region - image intensity



Dusty region - image intensity



Non-Dusty region - colour saturation



Dusty region - colour saturation

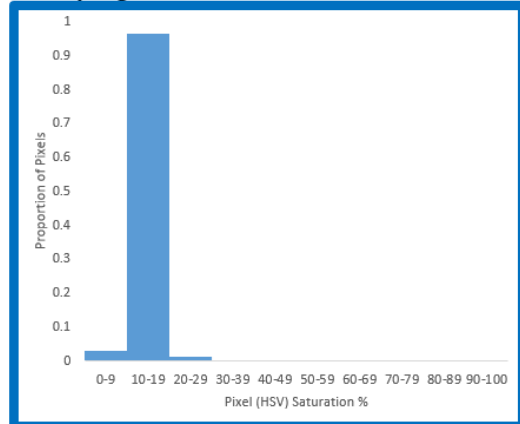


FIGURE 1 Two techniques for image analysis to identify dust on unsealed roads.

- Preprocessing methods may be able to detect defective images and mitigate this issue.
- Large regions of light dust within an image may provide a stronger dust signature than small regions of heavy dust.
 - Light dust may cover large regions of an image, increasing the presence (and total quantitative value across the image) of dust indicators without meriting an increase in dust intensity classification.
 - In photographs taken at higher vehicle speeds, heavy dust clouds may make up a comparatively small part of the image and so have less of an effect on road indicators.
 - The intensity of dust indicators within dusty image regions needs to be considered in order to make effective classifications.
- Visual similarity between unsealed road surfaces and road dust, and between cloudy skies and road dust.
 - Image regions picturing unsealed road surfaces and cloudy skies tend to produce similar effects on spatial frequency and color as road dust.
 - Regions of sky may be identified (and thus removed from assessment) by using region growing algorithms that are grown from the top of the image.
 - Identifying regions of road in an image is more difficult, as regions of dust in the image are likely to overlap regions of road, particularly on sloping sections of road.
 - Additional visual indicators such as texture analysis may better distinguish dusty image regions from the sky and the road.

A possible issue with using URICI dust severity categories is that the URICI method is intended to be used by evaluators traveling in a vehicle and the assessment criteria are phrased in terms of the dust's effect on safe vehicle speed as well as on dust intensity. Provided that visual indicators of road dust correlate with subjective assessment of dust severity using URICI, these indicators will still be useful for classifying images into URICI categories and so this should not be a significant issue.

SUMMARY AND CONCLUSIONS

This paper documents research into the feasibility of using photographic images taken with cell phones to assess the dustiness of unsealed roads. It aims at a qualitative categorisation of dust emissions in order to identify roads with dust emissions above acceptable levels. The research has focused on different image analysis techniques that will drive the algorithms for the cellphone assessment technology. The research concluded that the image analysis methods are showing strong potential for their intended purpose. It demonstrated that a number of techniques could be utilized for the image analyses, each having strong points and limitations within specific operating environments.

ACKNOWLEDGMENT

The research has been sponsored by Roadroid and New Zealand Local Councils, including Auckland Transport, Wairoa, Whangarei, and Kaipara.

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Visual Inspection-Based Maintenance Strategy on Unsealed Road Network and Gravel Loss

VASANTSINGH PARDESHI

*Scenic Rim Regional Council
University of Technology, Sydney, Australia*

SANJAY NIMBALKAR

University of Technology, Sydney, Australia

Gravel loss on unsealed roads is a major financial setback for all road agencies. It is also environmentally unsustainable to continue forever with continual gravel loss. Unsealed road management issues include difficulty in forecasting behavior, significant data collection needs, and vulnerability in level-of-service and maintenance practices. Quality of gravel material is one of the greatest influencing factors. All over the world and in Australia–New Zealand various studies have been completed and gravel loss models have been developed to estimate gravel loss. Those studies have developed gravel loss models and deterioration model and maintenance strategy. Some of the studies have used different approach such as system dynamic modeling approach, which can be grouped under asset management approach. This literature review summarizes various approaches related to gravel loss and briefly provides the available opportunities for further research. The Scenic Rim Regional Council (SRRC) has commenced long-term gravel loss project considering flood repair work of unsealed road network after the reconstruction work of gravel network due to Ex-Tropical Cyclone Debbie. Aim of this project is to refine gravel loss model based on proposed minor changes to material specification to the Paige-Green method and Australian Road Research Board’s specification mentioned in *Unsealed Road Manual*. At this stage SRRC maintenance strategy is based on visual inspection only which has inherent drawbacks. New maintenance strategy will be developed based on modified gravel loss model.

Safety

SAFETY

Development of a New Daytime Process for the Evaluation of Sign Retroreflectivity

DALE HEGLUND

LEANNA EMMER

CURT GLASOE

Upper Great Plains Transportation Institute, North Dakota State University

KENNY TETRAULT

CONNIE HOWELL

Burke County, North Dakota

INTRODUCTION

Local government agencies are required to implement and use an assessment or management method to maintain regulatory and warning sign retroreflectivity at or above the minimum levels in Table 2A–3 of the 2009 *Manual on Uniform Traffic Control Devices*. This requirement was implemented June 14, 2014.

The effort required to evaluate signs, the costs to complete sign reviews, and an understanding of the various allowable processes are hurdles that may challenge the small budgets and staffing of some local agencies.

Generally, either an assessment or a management system can be used to evaluate sign retroreflectivity. Acceptable assessments include visual nighttime inspections and measured sign retroreflectivity methods. Management methods include expected sign life, blanket replacement, and control sign life determinations. In addition, agencies may use other assessment or management methods developed based on engineering studies. This option provides a window of opportunity for this project.

Burke County Road Department, North Dakota, employees Kenny Tetrault and Connie Howell developed and tested an assessment process that is easy and fast to complete. As an added benefit, this low-cost process can be completed during normal work hours. Additionally, the county is able to utilize the full life of each sign, a shortcoming of management systems that rely on scheduled sign replacement based on the estimated life of a sign.

The purpose of this project is to validate that the Burke County Daytime Sign Retroreflectivity Assessment Method (also known as the North Dakota Daytime Sign Inspection Method) can be used to evaluate sign retroreflectivity minimum values as defined in the MUTCD and to define the steps needed to insure process validity.

METHODOLOGY

Burke County maintains about 1,500 road signs. After examining the approved methods, county staff thought there had to be a better, faster, easier, and less-expensive way to check the retroreflectivity of their signs. They experimented with various techniques and came up with a

process that uses an LED light to illuminate the sign and create the headlight effect used for nighttime inspections (Figure 1). Before and after LED illumination photos are taken (Figure 2). These photos are used to develop a sign condition rating.

Equipment used for the process included county truck, digital camera (\$300), bright LED lights with attachments (\$350), metalwork, wiring, and miscellaneous items. Collected data was entered into the county's existing sign inventory program.

Two powerful LED lights are installed on top of the inspection truck. Cloudy days provide ideal illumination and photo results. The pickup is parked 50 to 75 ft from the sign. A "before" picture is taken and then the bright LED lights are turned on and an "after" picture is taken of the sign. The two pictures are compared to determine the reflectivity. If there is any doubt about the sign's reflectivity, a comparison sign can be used at the same sign post and the pictures can be retaken to determine if the existing sign is in need of replacement. A few signs needed to be cleaned prior to the inspection.



FIGURE 1 LED light to illuminate the sign and create the headlight effect used for nighttime inspections.



FIGURE 2 (a) Before and (b) after LED illumination photos, taken to develop a sign condition rating.

Burke County inspects signs annually. The cost estimate for a Burke County sign inspection cycle with a retroreflectometer gun exceeded \$11,000.00. A nighttime inspection requires shift work that may not work well for counties with only a few road employees like Burke County. Along with the ability to complete this inspection during normal working hours, the cost to complete the inspection was only \$2,625.

FINDINGS

Burke County's initial daytime sign assessment results were favorable. Poor signage was identified and replaced, marginal signing was identified and flagged for upcoming replacement, and good signing was retained.

North Dakota State University's Upper Great Plains Transportation Institute, in partnership with its North Dakota Local Technical Assistance Technical Program (NDLTAP) team, plans to complete a research project to validate and define the process steps. This effort is proposed to support and help to develop the Burke County process, with hopes that the system can become an approved assessment method, saving local agencies time and money, and help agencies that may be having difficulty complying with sign assessment requirements.

The research project will be completed in 2019. The project will begin with a literature review. Field work will follow, with a sample of 200 Burke County signs inspected by a two-person crew. The crew will complete a nighttime visual inspection, a daytime retroreflectometer review and then the Burke daytime method. Criteria will be set for the Burke method to insure that distance, light differential, and other targeted variables are controlled to aid in the development and documentation of the procedure process. Light conditions will be evaluated for all readings. Testing will compare results from sunny versus cloudy days. In addition to the Burke County LED set-up, the research will compare various LED outputs and height placements, recording height of the light above the ground, and height above the driver. Testing distances will be recorded, with a range of distance data sets collected. Statistical review will be used to recommend a procedure to use for this test method. Distance from sign, LED light intensity, light placement, daytime sunlight conditions, and recording camera criteria are anticipated components of the daytime sign inspection specifications. Along with data output comparisons, the study will include cost comparisons for each method. NDLTAP will disseminate the findings to local leaders across the nation.

CONCLUSION

Most highway departments, including those at the state, local, and municipal levels, maintain a sign inventory as part of their sign retroreflectivity management plan. Sign retroreflectivity is a critical component in assuring safety on the local road system.

Burke County Road Department employees in North Dakota developed and tested an assessment process that is easy and fast to complete, is low-cost and can be completed during normal work hours.

UPGTI-NDLTAP will complete a research project to validate, define and document the process steps. This effort is proposed to support and help to develop the Burke County process,

with hopes that the system can become an approved assessment method, saving local agencies time and money.

Results of the North Dakota Daytime Sign Inspection Method for Sign Retroreflectivity will be presented at the 12th International Conference on Low-Volume Roads in Kalispell, Montana, September 15–18, 2019.

RESOURCES

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Initial Analysis of Ohio's Township Sign Grant Program

ALEJANDRO CHOCK

Mott MacDonald

INTRODUCTION

As in many states, low-volume roads make up the largest ownership segment of Ohio's total center-lane mileage. This abstract will analyze the initial resultant data from a statewide program aimed directly at addressing crashes on Ohio's low-volume township roadways. The abstract builds off the work co-authored by the author for a Transportation Research Board paper published in the *Transportation Research Record* while specifically looking at the data, and real-world examples of the program (1).

Beale et al. (1) reviews in detail the township qualification process, funding, and program management aspects of the program, and begins to explore the results of the implementation. This abstract will focus primarily on the results and will explore real-world examples to highlight the quantitative and qualitative benefits produced by the implementation. The analysis conducted in this abstract will show how Ohio implemented a low-cost countermeasure across a wide cross section of low-volume roadway applications to reduce crashes, and most importantly, immediately reduce fatalities and serious injuries.

In Ohio, low-volume roads are primarily township owned and maintained, with township roads representing 34% of Ohio's center-lane mileage. In total, 84% of Ohio's center-lane mileage is neither owned nor maintained by the state (local system). Most of Ohio's crashes (72%), fatalities (53%), and serious injuries (63%) occur off the state system based on the most recently published statistics, presenting a challenge to statewide policymakers concerning the development of crash reduction strategies (2).

Given Ohio's status as a home-rule state, additional challenges arise concerning the funding, program management, and ultimately the implementation of substantive crash reduction strategies on the local system. As one of many steps taken to address crashes on the local system, and to reach the ultimate goal of zero deaths on Ohio's roadways, the Ohio Department of Transportation (DOT) developed a Township Sign Grant Program (3).

The program uses Highway Safety Improvement Program funds to provide grants for up to \$50,000 in systemic, retroreflective, and supplemental roadway signage for qualifying townships for use at high crash and high-risk locations, reducing the paperwork burden on these smaller local public agencies by streamlining the federal aid process on their behalf (1). Signage provided through the program is limited to regulatory and warning signage typically found at horizontal-vertical curves and at intersections (4). Generally, the signage is intended to replace substandard signage and signs with poor or no retroreflectivity or to supplement deficient signage installations.

A thorough crash data analysis of the 24 pilot townships involved in this implementation revealed immediate and significant reductions in crashes and crash severity. One highlight of the analysis was the complete elimination of fatalities across the pilot townships in the first year

following implementation. In the two full calendar years since implementation, fatalities in the pilot townships are down by 67%, serious injuries by 33%, and total crashes by 15%.

The author believes that these data present a case for other states to follow Ohio's example and implement similar strategies aimed at low-cost, high-impact countermeasures in the low-volume roadway sphere. The results of Ohio's implementation clearly show the significant potential benefit of similar implementations across a wide variety of low-volume roadway applications and demonstrate the value of improved roadway signage as a viable countermeasure for practitioners in situations where more substantive countermeasures are otherwise unfeasible or cost prohibitive.

METHODOLOGY

Ohio DOT has developed several tools aimed at facilitating crash data analysis as part of its effort to make data-driven decisions when funding projects, especially those aimed at improving roadway safety. Crash data analysis for the Township Sign Grant Program was conducted using Ohio DOT's Traffic Information Mapping System (TIMS) and its fully integrated GIS Crash Analysis Tool (GCAT). Data from TIMS-GCAT was processed using Ohio DOT's Crash Analysis Module Tool (CAMTool), an Excel based tool used in to clean¹ raw crash data for detailed data analysis.

The initial implementation included 24 pilot townships, with signage installed during the year 2015. These pilot townships represent a broad cross section of the state, as shown in Figure 1. Representing suburban, exurban, agricultural, and Appalachian regions of Ohio, these townships provide a realistic cross section of the state from which to draw substantive conclusions on the success of the program.

The analysis consists of two parts—a general crash data analysis comparing before and after results of the implementation and a benefit-cost analysis of the before and after crash data. For both parts of the analysis, the 3 years prior to 2015 installation (2012–2014) represent the “before” dataset, and the 2 years following 2015 installation (2016–2017) represent the “after” dataset. The selection of these years is intentional to keep the analysis to whole calendar years, with the analysis omitting 2015 as installation occurred over the entire year.

Crash frequencies present the most direct comparison of program success. The crash data analysis compares before and after crash frequencies using the KABCO severity scale. In Ohio the KABCO scale represents fatal (K); incapacitating (serious injuries) (A); non-incapacitating (B); possible (C); and no-injury (O) crashes (5). The crash data collected represents multiple years, so the analysis compares the average yearly frequencies of specific crash severities before implementation to those after implementation.

The benefit-cost analysis uses the same crash data and KABCO severity scale, along with calculated costs based on the comprehensive societal² and human capital costs³ of a given crash, shown in Table 1.

The values shown represent Ohio DOT's adjusted values for FY2017 (6), applied to calendar year 2016. As a result, the benefit-cost analysis of this abstract uses only 2016 data for the after period, as opposed to the complete 2016–2017 dataset.

Benefit-cost ratios represent the multiplication of average yearly crash frequencies by the costs broken out by the KABCO severity scale as shown above. The difference in total cost for comprehensive societal and human capital costs between the before dataset and the after dataset

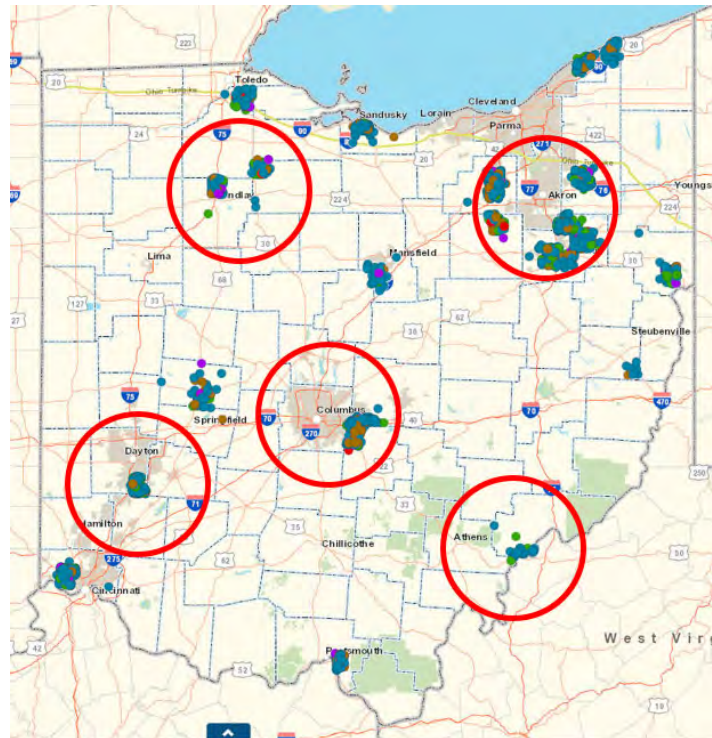


FIGURE 1 Geographic location of pilot townships in Ohio.

TABLE 1 Values for Comprehensive Societal and Human Capital Costs
 (Source: *Highway Safety Manual*. American Association of State Highway and Transportation Officials, Washington, D.C., 2010)

Crash Severity	Comprehensive Societal Cost	Human Capital Cost
K	\$5,731,115	\$1,669,104
A	\$303,038	\$149,276
B	\$110,683	\$56,146
C	\$62,311	\$38,056
O	\$10,046	\$8,576

is considered the overall benefit, with the costs represented by the total expenditure of the state to install the signage. In this case, a reduction in crashes and crash severities would provide a positive benefit–cost ratio.

FINDINGS

Analysis of the after implementation data reveals a 15% reduction in total crashes, a 67% reduction in fatalities and a 33% reduction in incapacitating injuries. These results occurred during a period of increases in Ohio’s (7) crash data. While pilot township crashes fell by the numbers shown, statewide total crashes increased by 10%, fatalities increased by 9%, and injuries⁴ increased by 11%.

Additional analysis revealed notable reductions in several key contributing factors, namely a 27% reduction in non-dry pavement (wet, snow, ice) crashes, a 6% reduction in crashes during the typical winter months of November–March, and a 5% reduction in crashes during typical overnight hours (7 p.m.–6 a.m.).

The cost of implementation for the 24 pilot townships was \$522,924.29, with an average of \$21,788.51 spent. The actual amounts spent per township vary significantly, partly because not every township requires the same quantities or types of signage. The minimum spent in a given township was \$733.86, and the maximum was \$51,096.29. Using 2016 as the after dataset, the total reduction in crashes and crash severity produced a comprehensive societal benefit of \$32,727,526.67 and a human capital benefit of \$11,227,502.67. Benefits are driven mainly by the reductions in fatalities and serious injuries when compared to the before dataset. The overall benefit–cost ratios are thus 62.59:1 and 21.47:1 respectively.

Qualitative benefits, such as improved retroreflectivity, play an important role in the quantitative results gathered. Figure 2 presents before and after imagery of an installation in Licking County, Ohio. In the improved condition (upper right), the installed chevrons provide motorists a visual cue of the curve ahead, while the roadway remains clearly visible. However, in nighttime conditions (center bottom), the purpose of this signage program becomes visible.⁵ On a completely dark roadway, typical of many low-volume facilities at night, the horizontal curve would be otherwise invisible to an unfamiliar, distracted, or fatigued motorist if not for the retroreflective signage.



FIGURE 2 Before (upper left) and after (upper right, lower center) images of the same signage installation.

CONCLUSIONS

The data presents a compelling case for the use of improved retroreflective signage on low-volume roads as a strategy for reducing crashes and crash severity. Since the datasets represent a vast cross section of the state, including suburban, exurban, agricultural, and Appalachian areas, the program presents a realistic example of potential benefits in almost any low-volume roadway application, and the immediate potential benefit this low-cost solution could have across the country.

Following the success in the pilot townships, Ohio DOT increased the budget for the program, and immediately began additional rounds of grants and signage installations. As of this writing, over 100 townships have received upgraded signage, with the ultimate goal of providing upgraded signage to all townships in Ohio who choose to take advantage of the program.

The author believes that Ohio's program provides an example for the nation that targeted, low-cost improvements can provide significant benefits even in the face of rising crash and fatality/injury frequencies state and nationwide. The author encourages practitioners and policymakers involved in safety and the design of low-volume facilities to use these data as a starting point for the development of similar programs in their respective states as yet another component in the drive towards zero deaths on our roadways.

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NOTES

1. Crash data is “cleaned” by removing erroneous entries. Because the Ohio Department of Public Safety (ODPS) crash reports are not physically linked to Ohio DOT’s GIS systems, but rather coded in after the fact, crash reports are sometimes geolocated to the wrong location all together, or, for example, the wrong ramp of an interchange. The CAMTool provides a link to the actual ODPS crash report that can be examined to verify location, date, time, and other pertinent crash details against the data presented in the CAMTool to ensure the correct data are analyzed.
2. The cost of each crash by either crash type or severity that includes the human capital costs in addition to nonmonetary costs related to the reduction in the quality of life in order to capture a more accurate level of the burden of injury.
3. The cost of each crash by either crash type or severity that includes monetary losses associated with medical care, emergency services, property damage, and lost productivity.
4. “Injuries” is used because the specific dataset provided by the state does not breakout crashes by the KABCO severity scale. In this case, “injuries” represents all crashes in the A, B, and C crash severities.
5. No pun intended.

SAFETY

Identifying Candidate Locations for Safety Improvements on Low-Volume Rural Roads

The Oregon Experience

AHMED F. AL-KAISY

FAHMID HOSSAIN

Montana State University

LEVI EWAN

Gallatin County

Low-volume roads constitute a significant proportion of the roadway network in rural areas, but they are usually associated with sparse crash data. This makes it impractical to rely on crash history alone to identify candidate locations for more-detailed safety investigations and potential improvements. This paper presents the development of a prioritization scheme, in the form of a crash risk index, to be used in ranking candidate sites for safety improvements on low-volume roads in the state of Oregon. The index developed utilizes information on highway geometry, roadside features, traffic exposure, and crash occurrence in assessing risk rather than relying solely on crash history in identifying hazardous locations. A roadway sample with a total length of around 830 mi was used in this study to represent different geographic regions in the state. Subsequently, extensive roadway, traffic, and safety data for the study sample were acquired and utilized in the development of the proposed index. A case study application of the proposed crash risk index on a 16-mi low-volume road corridor is presented, which shows how to apply the index practically on a typical low-volume road using information readily accessible to the agency.

To view this paper in its entirety, please visit:

https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

SAFETY

Pedestrian Safety Assessment Around Schools in Small Cities *Case Study*

HESHAM MAHGOUB

Engineering and Research International

ABNISH RAJBANSHI

South Dakota State University

Due to the increasing population and the resultant volume of vehicles in urban streets, there is a need for a pedestrian safety update. Pedestrians are the most vulnerable road users and protection of children is a first consideration. Parents in small cities and counties hesitate to let their children walk or bike to school due to variety of factors influencing pedestrian safety. The traffic control devices and pedestrian facilities around school zones need to be updated to account for the increase in population and vehicle volumes.

There is a need to perform pedestrian safety studies for small cities and counties. The procedure does not need to be performed by an expert and does not require more workers or funding. Therefore, the main goal is that the study would be easy to implement and serve as a simple tool for the inventory and evaluation of the condition of the pedestrian facilities, traffic control devices, and roadway characteristics in school zones while identifying the safety issues related to traffic and pedestrian behavior.

This paper presents and utilizes a simplified methodology to evaluate the existing pedestrian safety around school zones in Brookings, South Dakota. The methodology used in this study is designed to be as simple as possible and utilizes the main components of the Safe Routes to School (SRTS) program. The results of this study can be taken as a record for the SRTS program and can be helpful in performing road safety audits around school zones.

INTRODUCTION

Pedestrian safety depends mostly upon the public understanding of standard accepted methods for efficient traffic control. This principle is especially important in the control of pedestrians, bicycles, and vehicles in the vicinity of schools. Until, and unless, schoolchildren and vehicle operators understand the traffic control rules and benefits of using them, neither of them can be expected to move safely in school zones. School zones are one of the most critical high pedestrian areas, where mainly children and young kids are walking without supervision. According to several studies, these groups of pedestrians show unpredictable behavior and take more risks when walking or crossing the roads around a school area. The appropriateness of the school infrastructures, the sidewalks, pedestrian crossing, and the traffic control devices determines the comfort and safety level of pedestrian and cyclists in school zones. Pedestrian safety is always an important issue, as everyone's life, wherever he lives is priceless. Mainly, for the children who are more vulnerable to accidents, special safety programs have to be organized in the areas where they walk. To create a safe environment for children to walk to and from

school appropriate pedestrian facilities, traffic control devices, law enforcement and supervisions have to be provided by the appropriate authorities.

Brookings is a city in Brookings County, South Dakota, with a population of 20,184 (1). As the number of vehicles typically increases with the population of the city, the existing pedestrian safety in school zones must be updated. The South Dakota SRTS program has done some contribution relating to improvement of schoolchildren's safety. The SRTS program works with the school students and parents at a grass roots level to identify improvements that will make biking and walking to and from school a routine that is a part of S.D. students' experience.

Travel data collected from an online survey for the 2008 SRTS application shows that only 10% of the total numbers of students in all grades walk and only 1.7% ride a bicycle to and from school (2). The reasons included not only the distance or bad weather (cold at wintertime), but also other factors related to pedestrian safety. An online survey was completed to collect parent's comments for why they do not let their children walk or bike to school. The main reasons include the following:

1. Crossing guards are not present at all of the intersections on the way to school;
2. Heavy traffic and uncontrolled intersections;
3. High-speed vehicles, busy streets, and no adult supervision;
4. Not enough traffic control devices (signs, etc.);
5. Sex offenders in the area;
6. Insufficient crossing time at the signalized intersections; and
7. Children don't know about the traffic rules and are not aware of risks.

There are additional reasons related to safety that make parents hesitate to let their children walk or bike alone. Apart from the above-mentioned reasons, records from the Brookings Police Department show numerous traffic violations near school zones and injuries including school children. To create a safe environment for children to walk to and from school appropriate pedestrian facilities, traffic control devices, law enforcement, and supervisions have to be provided by the appropriate authorities.

This paper presents and utilizes a simplified methodology to evaluate the existing pedestrian safety around school zones in Brookings. The methodology used in this study is designed to be as simple as possible and utilizes the main components of the SRTS program. The results of this study can be taken as a record for the SRTS program and can be helpful in performing road safety audits around school zones.

OBJECTIVES

The main objectives of this study can be summarized as follows:

1. To present a simplified procedure to assess the existing condition of pedestrian facilities and traffic control devices in school zones in small towns and counties.
2. To identify the strengths and weakness in the traffic control devices and pedestrian and other facilities by observation, as well as, comparison with the standard guidelines.

3. To evaluate the school zones with their pedestrian facilities and traffic control devices and to recommend possible priorities and suggestions to schools with low number of good conditions.

This type of pedestrian safety project is highly recommended for small cities and counties like Brookings, with low population and volume of vehicles. The procedure does not need to be performed by an expert, and does not require more workers or funding. Therefore, the main goal is that the study will be easy to implement and it will serve as a simple tool for the inventory and evaluation of the condition of the pedestrian facilities, traffic control devices, and roadway characteristics in school zones while identifying the safety issues related to traffic and pedestrian behavior.

STUDY LOCATION

Brookings city has five public schools serving kindergarten to high school students (pre-schools not included). The study locations include the area around each of these schools. The adjacent streets to the school building and the surrounding intersections fall under the area of observation. As an example, the school information and the satellite map of Hillcrest School, with the study area highlighted are shown in Table 1.

The highlighted areas include adjacent, as well as non-abutting streets to Hillcrest school where the study was carried out. It includes the streets surrounding the school and busy intersections where a lot of students are seen walking and crossing the roads. Some parts of the streets that are not adjacent to the school are also considered because they are the source of vehicles as well as pedestrians entering the school zone.

METHODOLOGY AND DATA COLLECTIONS

This section mainly describes the procedures used for collecting data and the methodology applied to evaluate different factors influencing pedestrian safety. Several surveys were carried out in the evaluation process for the traffic control devices, pedestrian facilities, and pedestrian and traffic behavior in school areas. Figure 1 illustrates the procedure applied for the evaluation of pedestrian safety around school areas in Brookings.

The step by step procedure shown in Figure 1 can be explained as follows:

1. Identification of school zones, mainly including the adjacent streets to each school was the first step of this procedure. School information, including the number of students enrolled, was collected from the school district and through school visits. General ideas about the routes used by students to walk to and from the school were also identified.
2. Preliminary survey included visiting each school, taking pictures of the school area including any proper or inadequate existence of the pedestrian facilities, traffic control devices, etc. Satellite maps were also carried to sketch the rough location of traffic control signs, presence of crossing supervision, and types of nearby intersection (signalized or stop controlled).

TABLE 1 School Information

No.		
1	Name	Hillcrest Elementary School
2	Academic level	Grades K–3
3	Student enrollment	490
4	School time	8:15 a.m. – 3:22 p.m. (Mon.–Fri., except Wed., 8:15 a.m. – 2:22 p.m.)
5	Adjacent road type	One lane each direction, two-way traffic
6	Location	304 15th Ave.
7	City/municipality	Brookings

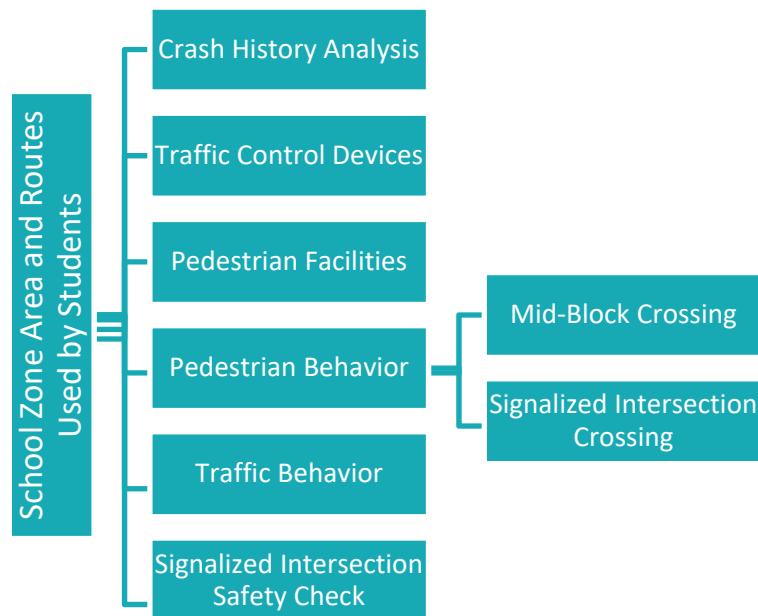


FIGURE 1 Methodology applied for the evaluation of pedestrian safety.

3. Inventory of data for the evaluation of pedestrian safety was carried out, which included crash history reports and the following surveys:

- The crash report for past 3 consecutive years around five school areas in Brookings were collected from the Brookings Police Department.
- Investigation study was performed to identify the adequacy and performance of pedestrian facilities, traffic control devices which included following checks:
 - Distances between traffic signs, and the nearby crossing, to check the location of signs are adequate with the *Manual for Uniform Traffic Control Devices (3)*;
 - Width and condition of sidewalk;
 - Location, size, and marking of crosswalk; and
 - Other facilities like school parking, pick-up and drop-off area, bus loading zone, and playgrounds.
- A pedestrian behavior survey to monitor the students and other pedestrians' crossing behavior was performed. The survey was carried out at a mid-block crossing and at a signalized intersection crossing.
- For the observation of traffic behavior, a spot speed study was conducted at adjacent streets to schools that had accident history.
- Signalized intersections mostly busy with school pedestrians were investigated for pedestrian safety (e.g., wheelchair ramps, pedestrian signals, street lights).

4. Evaluation of existing conditions of pedestrian safety was performed by analyzing data collected from the inventory process. Comparison among schools with their existing pedestrian safety conditions was then performed.

5. Possible recommendations were given if any improper trends were identified or improper facilities existed in any school.

Crash History

Accident data collected from the City of Brookings Police Department around each school for the past 3 consecutive years. Figure 2 illustrates different contributing factors for accidents around school areas. Y-axis denotes the number of accidents, whereas X-axis represents the year at which the crash occurred.

Speeding, improper driving, failure to give right-of-way, etc., falls under driver behavior. Icy roads and obstructed view falls under weather and roadway conditions; illness and drunk driving falls under driver's physical condition. In 3 years of accident reports, drivers' carelessness and their intentions of not following traffic rules have been observed as the main causes of crashes. Causes such as drivers' physical inability, less visibility, and slippery roads due to bad weather conditions were less numerous. As motorist behavior appears to be the significant cause of accidents, more strict law enforcement needs to be established around school areas.

Adjacent Streets Safety Evaluation

Streets adjacent to each school were surveyed for evaluation of the adequacy of the existing traffic control signs and pedestrian facilities with the standards. Traffic control signs like "Advance School Crossing" and "School Speed Limit," etc., were measured with tape from the nearby crossing to identify if they were located properly.

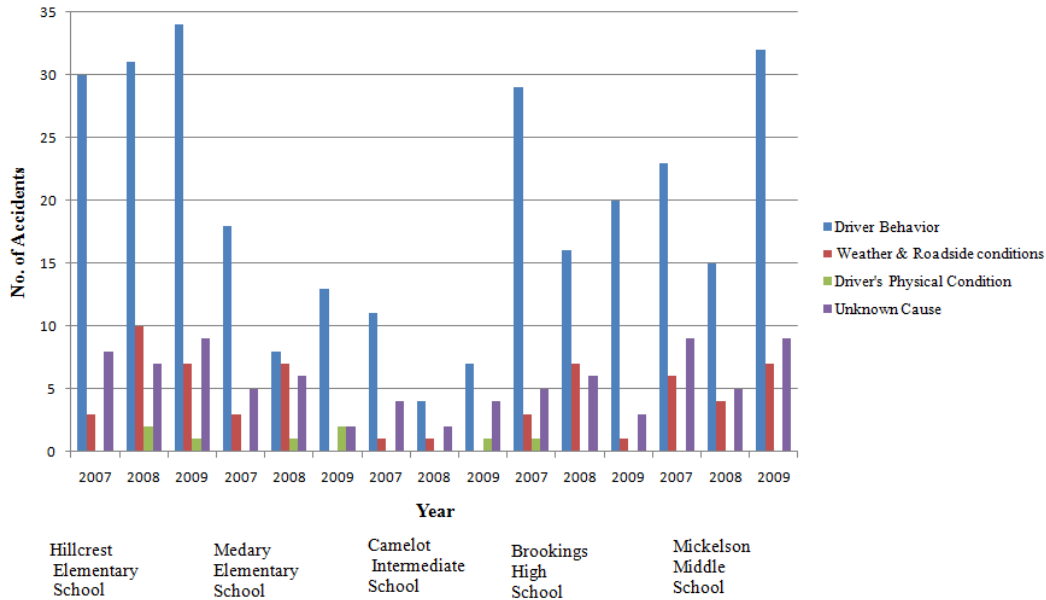


FIGURE 2 Crash history analysis around Brookings’ school areas.

Crosswalk markings and stop line sizes were checked with the standards along with the distance between the crosswalk and the stop line. On-street parking in either side of the adjacent roads to the school was observed. Conditions and width of the sidewalk on adjacent streets were also examined.

Evaluation of condition of sidewalk was done using three different categories—deficient, regular, and excellent—as shown in Table 2.

The data collected from this survey for the Hillcrest Elementary School, as a sample, is shown in Table 3.

TABLE 2 Conditions of Sidewalk (4)

Deficient	Regular	Excellent
Sidewalk doesn't have effective width of 5 ft	Sidewalk having effective width of 5 ft	Sidewalk with effective width of 5 ft or more
Contains fixed obstacles, poles, or trees	No fixed obstacles to make hazard for pedestrian	
Side walk with a drop off of 3 in. or higher between contiguous blocks or with holes or deformations present making hazard to pedestrian or obstacles to wheelchair	Sidewalk with drop offs less than 3 in. between contiguous blocks or with minor surface deformations	No drop offs between contiguous blocks, no surface deformations, no broken blocks and obstacles that affect the sidewalk width, comfort and safety of the pedestrian

TABLE 3 Adjacent Streets Data Collection of Hillcrest Elementary School

No.	Adjacent Road to the School	Pedestrian Facility, Traffic Control Device, or Traffic Characteristic	Distance or Size of the Safety Signs and Facility		Remarks	
			Traffic Direction			
			North (N)	South (S)		
1.	Along 15th Ave.	Sidewalk width	4 ft 4 in.	5ft 3 in.	N: effective width less than suggested (5–7 ft); S: as required	
		Sidewalk condition			N: deficient with failed curb at majority of length; S: regular	
		Pedestrian crosswalk marking size (each strip)	7 ft 5 in. x 1 ft 2 in. 6 ft 5 in. x 1 ft 2 in.	5 ft 6 in. x 1 ft 2 in.		Crosswalk strip sizes are appropriate to the guideline but needs to be repainted
		Stop line width	2 ft	2 ft		Appropriate width but needs repainting
		Stop line distance from the crosswalk marking	4 ft 2 in.	4 ft 2 in.		Meets MUTCD requirement
		School speed limit sign (15 mph)	38 ft after cross walk	1st at 331 ft; 2nd at 690 ft from the crosswalk		
		On street parking	Parallel 1 h No parking	8 a.m. – 5 p.m. weekdays		
2.	Along 3rd St.	School advance warning sign	103 ft before crosswalk	310 ft before crosswalk	E: does not match as suggested (min. 150 ft); W: appropriate	
		Crosswalk marking size (each strip)	7 ft 2 in. x 1 ft 6 ft 2 in. x 1 ft 2 in.	6 ft 6 in. x 9 in.		Falls under appropriate size but needs to be painted
		Stop line marking width	2 ft	2 ft		Appropriate width but needs repainting
		Stop line distance from the crosswalk	5 ft 2 in.	4 ft		Appropriate
		Sidewalk	5 ft 6 in.	4 ft 6 in. 9 ft 3 in. from bus loading zone		E: trees and drive ways before street, no sidewalk after 16th Ave.
		School speed limit sign(15 mph)	80 ft from crosswalk marking	620 ft from the crosswalk marking		
		On-street parking		No parking midnight to 4 p.m. weekdays		

Observation of Pedestrian Behavior

Pedestrian behavior trends while crossing the roadway or signalized intersection is a very important safety issue. There are always safety concerns when there are conflicts between pedestrians and the vehicles, and pedestrian behavior in a certain way can reduce or increase the possibility of crashes. For the pedestrian behavior survey, the mid-block crossing on 5th Street South in front of Medary Elementary and the signalized intersection on 8th Street South at 17th Avenue South were chosen. Both locations do not have crossing supervision and are used by a large number of students after and before school. The signalized intersection at 8th Street South and 17th Avenue South is mainly used by students from Mickelson and Camelot schools.

The data was collected for 2 days in each location after the school was dismissed. In the mid-block crossing, it was observed whether students or regular pedestrian cross the street using the cross walk or not. Similarly, for the signalized intersection, pedestrians were observed if they cross the street during pedestrian indication or not. Data collected for pedestrian behavior survey at the mid-block crossing in 5th Street South are shown in Table 4.

Spot Speed Study

For the observation of traffic behavior at the school areas, a spot speed study of the vehicles was performed. Three locations at streets adjacent to schools were chosen where accidents were reported—5th Street South adjacent to Medary Elementary and adjacent to Brookings High

**TABLE 4 Pedestrian Behavior at Mid-Block Crossing Survey
(5th Street South at Medary Elementary School, December 10, 2010)**

School Pedestrians					
Time Period (min)	Uses Pedestrian Crossing		Does Not Use Pedestrian Crossing		Total
	Male	Female	Male	Female	
5	10	5	5	2	22
10	3	2	4	1	10
15			2		2
20					
25					
Total	13	7	11	3	34
Regular Pedestrians (Including Students' Parents)					
Time Period (min)	Uses Pedestrian Crossing		Does Not Use Pedestrian Crossing		Total
	Male	Female	Male	Female	
5	5	3	5	2	15
10	3	2	2	1	8
15		1	2	1	4
20					
25					
Total	8	6	9	4	27

School and 17th Avenue South in front of Mickelson Middle School. The speed study was performed in the afternoon just before the end of school until the students were seen in the adjacent streets.

After the speed of each vehicle was identified, 85th percentile speed was determined for each location. The 85th percentile speed is the speed at which 85% of the observed vehicles are traveling at or below. This speed is normally assumed to be the highest safe speed for a roadway section. The 85th percentile is used in evaluating and recommending posted speed limits based on the assumption 85% of the drivers are traveling at a speed they perceive to be safe. To check if the 85th percentile speed is too high compared to the posted speed limit, a rule of thumb of 5 mph is used sometimes. If the 85th-percentile speed is 5 mph or more above the posted speed limit, the situation should be evaluated (5). Data collected in the study areas indicate that the 85th percentile speed was between 20 to 22 mph.

Pedestrian Safety at Signalized Intersection

Four sites were chosen to evaluate the signalized intersections around school areas. Selected sites were at the intersections where school students were seen crossing the streets on their way to/or from school. The four intersections selected were the intersection at 5th Street South and Medary Avenue South; 8th Street South and Medary Avenue South; 7th Avenue South and 8th Street South; and 17th Avenue South and 8th Street South. Pedestrian signals were checked to see if there was sufficient time for a pedestrian to cross the street.

Only the intersection at 8th Street South and Medary Avenue South was checked for the pedestrian signal for crossing time, because this intersection is the busiest one crossed by school students in Brookings.

The total time required for pedestrian movements is calculated by Equation 1 (6):

$$T = Z + R \quad (1)$$

where

T = The total time required for pedestrian movements;

Z = The walk allowance; and

R = The time required for a person to traverse the crosswalk which can be calculated as the width of intersection in feet) divided by the walking rate or speed.

In this location, the designer used the average speed of pedestrian which is 4 ft/s, where in intersections used by very young or handicapped pedestrians, the walking rate should be taken as 3 ft/s. Having the width of intersection between 5th Street South and Medary Avenue South equal to 36 ft, the pedestrian clearance time observed was 9 s. Taking 4 ft/s as walking rate of pedestrian clearance time (R) will be 9 s, while taking as 3 ft/s, the clearance time needed will be 12 s.

Except for the pedestrian clearance time check, each of the signalized intersections selected were checked for different pedestrian facilities that affects the safety of pedestrians while crossing the street. Table 5 shows an example of the pedestrian safety check carried out in the intersection at 5th Street South and Medary Avenue South.

**TABLE 5 Pedestrian Safety Checks (Intersection at 5th Street
South and Medary Avenue South, March 21, 2011)**

Pedestrian Safety Factors	Presence or Absence/Remarks
Are all sides of the crossing is marked for pedestrian path?	Yes
Is the pedestrian signal indicator provided at every crossing?	Yes, walk/don't walk indicators
Push button, if present, is in clear view and wheelchair accessible?	Not present
Is wheelchair curb ramp present on all corners?	Yes. Absence of detectable warnings; only four ramps at four corners
Are Stop bars painted in advance of the crosswalk on all approaches?	Yes
Is streetlight present on all four corners?	Only three corners
Does the ramp lead wheelchair occupant in direct line to the pedestrian crosswalk?	No
Is there any presence of crossing supervision when school pedestrians are crossing?	Yes

DATA ANALYSIS AND EVALUATION

This section evaluates the pedestrian safety in school areas by comparing traffic control devices and pedestrian facilities that exist in each school area under study. In order to simplify the results and allow parents, law enforcements, and school officials to understand the results and draw the suitable decisions to enhance the traffic safety around the school areas, the results were presented in simple table forms and figures.































Evaluation of School Traffic Control Signs

Presence of school traffic control signs at each of the schools is compared with the MUTCD guideline for its installation. To be compliant with the MUTCD, the traffic signs should be of a particular height, size, color, design, and installed at designated locations. Not all of the school traffic control signs mentioned in the MUTCD are compulsory to install, but some of the important ones have been used to evaluate. Table 6 shows whether a school in Brookings area does have or does not have the mentioned traffic control signs and whether or not the signage is compliant with the MUTCD. For each case a different legend has been shown.




Similarly, all the pedestrian and other facilities where schoolchildren are involved were evaluated for presence/absence and standard compliance. Table 7 shows the evaluation of different pedestrian and other facilities present at each school in Brookings.

From the evaluation of traffic control signs, Camelot School seems to have the most signs compliant with the MUTCD guidelines. None of the schools under the study have the advance school bus stop sign mentioned in the guidelines, although Mickelson has different warning sign in advance of the off street bus loading zone. Only Camelot School has the "end school zone" sign. The school speed limit warning sign present in each school was appropriate.

TABLE 6 School Traffic Control Signs Evaluation

School Signs	Hillcrest Elementary School	Medary Elementary School	Camelot Intermediate School	Mickleson Middle School	Brookings High School
					
					
					
					
					

Legend

- Not present 
- Present but not MUTCD compliant 
- Present and MUTCD compliant: 

Evaluation of Pedestrian Behavior

The main objective for performing the pedestrian behavior study was to find out if any unsafe behavior trends are followed while crossing the streets, so that serious issues can be pointed out and possible safety strategies can be recommended. Table 8 shows the distribution of different type and gender of pedestrian observed during mid-block crossing study. Overall, 120 pedestrian were observed in 2 days. Table 8 presents the number of pedestrian not using the mid-block crosswalk while crossing the street, which shows a total of 52 pedestrians, including students and general pedestrians, did not use the crosswalk.

Figure 3 is a chart that illustrates percentage of students and general pedestrian not using the mid-block crosswalk while crossing the street, with the male and female presented separately. Table 8 also presents the distribution of pedestrian crossing at the signalized intersection. No general pedestrians were observed during the study period.

Figure 3 illustrates that a huge percentage of pedestrians showed carelessness while crossing the mid-block crossing in 5th Street South at Medary Elementary School. Male pedestrians were seen to ignore the crosswalk more than females, as they were more likely to take risks while crossing the streets. Although the general pedestrian had the highest percentage (53%) in ignoring crosswalk, we were mostly concerned with the high student percentages: 41% of males and 34.7% of females along with 39% overall of students did not use crosswalk while crossing the mid-block crossing.

TABLE 7 Pedestrian and Other Facilities Evaluation

Pedestrian and Other Facilities	Hillcrest Elementary School	Medary Elementary School	Camelot Intermediate School	Mickelson Middle School	Brookings High School
Sidewalk condition					
Sidewalk width					
ADA wheelchair ramp (width and slope)					
Pedestrian crossing					
Crossing marking width					
White line crossing marking					
Stop line					
Pick-up/drop-off zone					
Parking facility					
Bus loading zone					

Legend

- Not present
- Present but not compliant
- Present and compliant:

TABLE 8 Mid-Block Crossing Study Pedestrian Distribution

Pedestrians Using Mid-Block Crosswalk			
Pedestrian Type	Male	Female	Total
Students	46	23	69
General	32	19	51
Total	78	42	120
Pedestrians not Using Mid-Block Crosswalk			
Pedestrian Type	Male	Female	Total
Students	19	8	27
General	17	8	25
Total	36	16	52

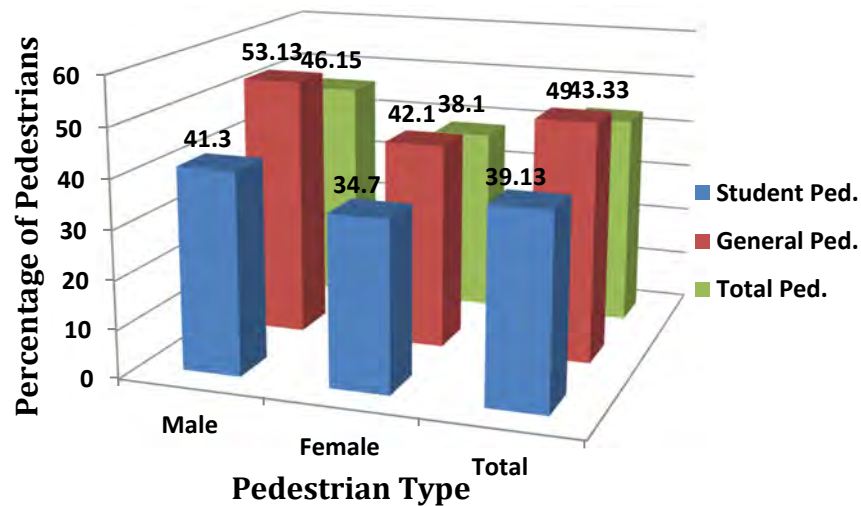


FIGURE 3 Percentage of pedestrians not using mid-block crossing.

This large percentage of students showing unsafe behavior should be addressed. The main reason for students and parents not using crosswalks was looking for the shortest path to reach their vehicles parked on street at a distance from the mid-block crossing. Due to insufficient space for parking and pick-up/drop-off zone, parents are forced to park along an adjacent street. This creates a lineup of vehicles on the street. When students come out of the school, they use the shortest path to reach the parent's vehicle instead of a mid-block crossing.

Evaluation of Spot Speed Study

The main objective of this spot speed study was to identify if the drivers violated the posted speed limit in the school area when children were present. If violation trends exist in large scale proper strategies should be applied to upgrade pedestrian safety.

Figure 4 presents the percentages of vehicles exceeding the posted speed limit in the three schools' area, as well as the percentages of vehicles which have speeds 5 mph or more than the posted speed limit. On 17th Avenue South, at the Mickelson School area, speeding was observed in highest amount with 90% of vehicles under study having speed more than 15 mph when children were present, followed by 68% in Brookings High School area, and 52% in Medary Elementary School area. Vehicle speeds vehicle 5 mph or more than the posted speed limit were also highest in Mickelson School area with 34%. The high school and Medary school were 32% and 16% respectively.

The spot speed study in three school area revealed a serious issue in pedestrian safety. In all school areas under study there were large number of speed limit violations. Also, the percentage of vehicles much higher than the posted speed limit was considered large. Even a single vehicle driving above the posted speed limit can cause fatal consequences in a school zone. In these and other school areas strategies like traffic calming, strict law enforcement, and awareness education should be applied.

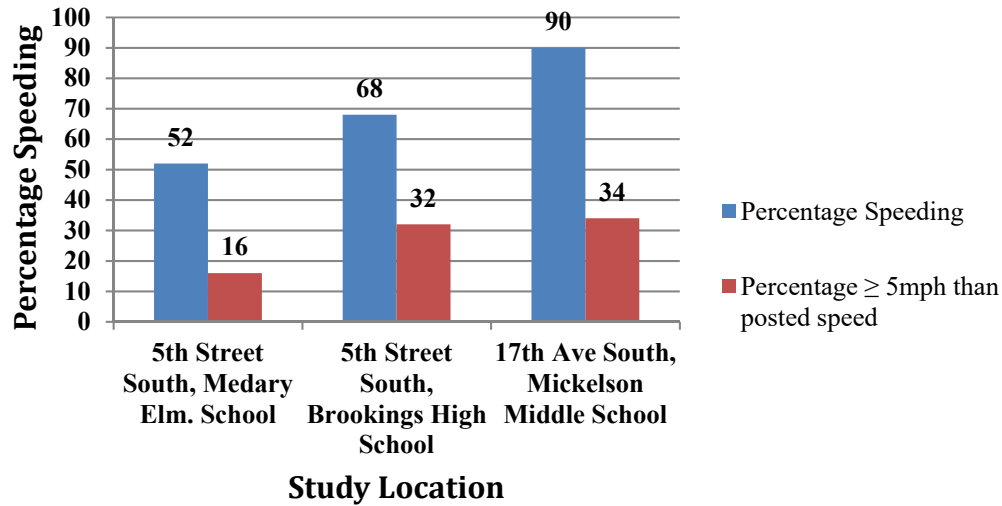


FIGURE 4 Percentage of vehicles crossing the posted speed limit.

CONCLUSION

The objective of this study was to identify study analysis methodology easy to implement and serve as a simple tool for the inventory and evaluation of the condition of the pedestrian facilities, traffic control devices, and roadway characteristics in school zones while identifying the safety issues related to traffic and pedestrian behavior.

The evaluation of existing traffic control devices and pedestrian facilities is very important to figure out the unsafe features and practices that can cause accidents. Presenting the best existing conditions of one school could set an example for other schools to follow. Whereas pointing out the weakness related to pedestrian safety in a certain school area could help concerned authorities to work on the particular improvement that is necessary.

The most important factor for the pedestrian safety is awareness or educating of students, kids, drivers and all the people who uses the road or other pedestrian facilities at some point. Problems that have been seen in all the school area under study mainly relate to the behavior of the pedestrians and motorist, rather than the facilities. Crossing the streets carelessly at an improper time on an undesignated path shows the unsafe behavior of school students. Whereas speeding in the school area where young kids are seen walking and not giving the right of way to the pedestrians are some of the unsafe behaviors demonstrated by the motorists in the area. Strict law enforcement is needed, but without awareness in people, enforcement alone cannot guarantee hundred percent safety.

RECOMMENDATIONS FOR FURTHER STUDIES

Pedestrian safety concerns during wintertime have not been evaluated in this study. Icy road conditions and poor visibility increases chances for crashes, as well as problems for pedestrians. There is a need of safety evaluation during wintertime at school areas in Brookings city. A

similar kind of procedure as mentioned in this design paper could be developed for evaluation and improvement of pedestrian safety during winter.

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SAFETY

Safety Performance Functions of Low-Volume Roadways

SUBASISH DAS

IOANNIS TSAPAKIS

SONGJUKTA DUTTA

Texas A&M Transportation Institute

The Fixing America's Surface Transportation Act mandates a highway safety improvement program for all states that "emphasizes a data-driven, strategic approach to improving highway safety on all public roads that focuses on performance." To determine the predicted crashes on a specific roadway facility, the most convenient and widely used tool is the first edition of *Highway Safety Manual*, which provides predictive models [known as safety performance functions (SPFs)] of crash frequencies for different roadways. Low-volume roads (LVRs) are defined as roads located in rural or suburban areas with daily traffic volumes of less than or equal to 400 vehicles per day. LVRs cover a significant portion of the roadways in the United States. While much work has been done to develop SPFs for high-volume roads, less effort has been devoted to LVR safety issues. This study used 2013–2017 traffic count, and roadway network and crash data from North Carolina to develop six SPFs for three LVRs, which can be used to predict total crashes, as well as fatal and injury crashes. This study also performed a sensitivity analysis to show the influence of traffic volumes on expected crash frequencies. The SPFs developed in this study can provide guidance to state and local agencies with the means to quantify safety impacts on LVR networks.

To view this paper in its entirety, please visit:

https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

Reliability-Based Prediction of Number of Crashes on Low-Volume Roads in Montana Using the Equivalent Property Damage-Only Crash Indicator

SUDIP BHATTACHARJEE

Alabama Agricultural and Mechanical University

INTRODUCTION

The *Highway Safety Manual* (HSM) (1) provides a list of methods to evaluate safety of highway networks. Almost all of these procedures provide the use of actual observed crash data and some involve the combination of observed crash frequency and predicted crash frequency using Safety Performance Functions (SPFs). One of the methods, known as Critical Crash Rate (CCR) method, involves the confidence level of predicted crashes. The CCR is calculated using the Equation 1:

$$CCR = C_o + \frac{0.5}{MEVM} + p \sqrt{\frac{C_o}{MEVM}} \quad (1)$$

where

- C_o = the weighted average Equivalent Property Damage-Only (EPDO) crashes,
- $MEVM$ = the million entering vehicle miles, and
- p = the level of confidence = 1.645 for normal distribution and 95% level of confidence.

The CCR of a site can be calculated and compared with the actual crash rate (CR) of the site to determine whether the site needs improvement. Although CCR method includes the effect of crash variability in terms of the confidence interval, the explicit calculation of the reliability against the threshold number of crashes is not performed. Moreover, the calculation of SPF in any of the HSM methods involves the use of Crash Modification Factors (CMFs) which modify the base condition to the site specific conditions. In this study, a reliability-based procedure has been developed which is based only on the observed crash data and SPF-predicted crash numbers under base conditions; no CMF is required to adjust the SPF-predicted values. With the calculation of reliability against a threshold crash number a segment of low-volume road (LVR) can be analyzed and compared with other similar roads or segments.

In many cases the crash potential assessments of LVR segments are not routinely performed using HSM methods because of lack of sufficient crash data. The Montana Department of Transportation (DOT) maintains the records of crashes observed in large network of Montana LVRs (2). This study has been primarily focused on the evaluation of the LVRs in Montana using both the CCR method and the reliability analysis against threshold crash frequencies. Both the historical crash data and predicted crashes based on forecasted annual average daily traffic (AADT) are determined using CCR and the reliability analysis. The objectives of the study were to

- Use Montana crash data for LVRs to determine CCRs of the selected sites;
- Calculate the reliability of predicted crashes for the past historical record and the future predicted AADT; and
- Compare the CCR and reliability results to verify the reliability-based procedure.

METHODOLOGY

The following steps were followed in the study to calculate the reliability of crashes.

1. Collection of LVR crash data. Ten LVR sites over the entire state of Montana were selected for the analysis and the crash data including the total, fatal, injury and property damage-only (PDO) crashes were obtained. The sites are identified in Table 1 with the AADT for a 5-year period obtained from Montana DOT traffic count data archive (3) and crash frequencies.

2. Analysis of crash data to determine EPDO crashes and the CCR. The number of crashes observed on each of the 10 roadway segments was converted to PDO crashes by using the HSM EPDO method: each fatal crash was equivalent to 542 PDO crashes and each injury crash to 11 PDO crashes. The total EPDO for each site was determined. Then the MEVM and the CCR were calculated for each site. Finally, the CR given by number of total EPDO crashes per MEVM for each site was compared to the CCR to determine whether the site needed improvement. For the sites with CR greater than CCR were selected as potential candidate for improvement projects.

3. Calculation of reliability against threshold crash frequency based on EPDO crashes for the historical crash data. The reliability is defined as the probability that the number of observed crashes is less than the threshold crash number. In this study the actual distribution of number of predicted crashes in the field was determined for each site using the following relation:

$$N_{\text{Field, predicted}} = N_{\text{Bias}} \times N_{\text{SPF, predicted}} \quad (2)$$

where

- $N_{\text{field, predicted}}$ = the predicted number of EPDO crashes in the field;
 N_{Bias} = the ratio between the observed crashes and the SPF predicted crashes; and
 $N_{\text{SPF, predicted}}$ = the SPF predicted number of crashes under base conditions.

The bias for each site was determined using the crash data. Because bias varied randomly with equal chance during each year, the bias was modeled as uniform distribution and the distribution parameters were determined (Parameter A and B in Table 1). The SPF-predicted crashes were determined by applying the two-lane rural roadway segment functions from HSM with the average AADT values. The predicted number of crashes during the period of study was then determined using Equation 2. The overall reliability calculation was performed using the following steps for each site location:

- a. The total average predicted crashes during the analysis period were determined using the SPF and base conditions. The percentage of individual crash severity type (fatal, injury, and PDO) was calculated from the total number of crashes.

b. Using the average SPF and the overdispersion parameter, the probability distribution of SPF predicted crashes was determined by random sampling from Poisson distribution with the mean determined in step a. It was found that because the overdispersion parameter was small, the negative binomial distribution of the crash frequencies coincided with the Poisson distribution.

c. The bias for each year was calculated and the bias (uniform) distribution parameters were determined.

d. The distribution of field predicted number of crashes during the analysis period was determined using Equation 2 by multiplying the bias distribution with the SPF-predicted distribution.

e. The total number of field predicted crashes was converted to EPDO crashes using the crash type distribution determined in step a.

f. The threshold number of crashes were determined using the CCR method and the reliability against EPDO per MEVM (EPDO/MEVM) was determined as: Reliability = Probability [number of EPDO/MEVM < threshold number of EPDO/MEVM].

g. The 95th percentile of the EPDO/MEVM crashes was then determined.

4. Reliability calculations based on forecasted AADT. The future AADT for each site was determined by three growth rates: 1%, 3%, and 5%. The same reliability calculation procedure outlined above was performed for each scenario, and the 95th percentile number of EPDO/MEVM crashes and reliability were determined for each case.

FINDINGS

Table 1 shows the results of the application of CCR and reliability analysis method using the historical AADT. Table 2 shows the results for forecasted AADT.

The results in Table 1 indicate that segments 1, 3, and 10 have very low reliability and require improvement. Table 2 indicates that the 95th percentile and reliability values do not depend on traffic growth rate when past 5 years of traffic data are used. The sites that have reliability less than a critical value (e.g., 80%) are selected as candidate sites for improvement projects. The agency can select a critical reliability level based on economic considerations. The entire calculation is performed using Microsoft Excel spreadsheet.

Once a segment is identified as a potential candidate based on low reliability against CCR, it can be divided in smaller subsegments and the reliability of each subsegment may be analyzed based on its own CCR using the same reliability analysis principle. Thus, a top-down systemic approach can be used for the entire network of the LVRs. First, a set of road segments each having similar characteristics within is identified. Then reliability analysis of each segment is performed using EPDO/MEVM of the segment to identify critical segments. Finally, to identify the “black spots” within each critical segment, the segment is divided in subsegments and reliability of each subsegment is calculated to evaluate its potential for improvement. The approach of reliability-based decision-making process includes the effect of uncertainties and regression to mean (RTM) bias. In addition, the approach does not require use of CMFs as it relates SPF predicted to field values. Thus this approach provides better judgement compared to the approaches where only the total number of crashes is used in the decision-making process.

TABLE 1 Analysis Results of LVR Segments Based on Past 5 Years AADT

Roadway Segment	1	2	3	4	5	6	7	8	9	10
MT Highway	508N	556N	83N	486N	223N	45N	57E	57E	18N	48N
Length (mile)	28.561	41.533	74.33	19.793	51.283	40.131	44.481	28.081	68.479	23.941
Average AADT	227.80	93.0	965.7	363.8	507.6	605.2	496.7	823.5	329.5	644.4
Fatal	2	0	7	0	1	0	0	0	0	3
Injury	8	1	100	15	10	17	8	5	12	9
PDO	11	7	226	31	16	42	22	21	15	22
Total crashes	21	8	333	46	27	59	30	26	27	34
Total EPDO crashes	1183	18	5120	196	668	229	110	76	147	1747
MEVM	11.9	7.0	131.0	13.1	47.5	44.3	40.3	42.2	41.2	28.2
EPDO/MEVM	99.6	2.6	39.1	14.9	14.1	5.2	2.8	1.8	3.6	62.0
Weighted avg. EPDO	23.8									
Critical EPDO/MEVM	26.2	26.9	24.5	26.1	25.0	25.0	25.1	25.0	25.1	25.3
Need evaluation based on CCR	Yes		Yes							Yes
Bias distribution										
Parameter A	0.507	0.888	2.284	3.554	0.269	1.161	0.232	0.239	0.355	0.223
Parameter B	4.345	2.201	4.742	6.174	1.278	2.499	1.732	1.472	1.473	3.013
95th percentile EPDO	174.4	4.4	47.6	23.3	19.2	6.8	3.7	2.6	5.0	93.6
Reliability	0.0165	1	0	0.973	0.997	1	1	1	1	0.0175
Need evaluation based on reliability	Yes		Yes							Yes

TABLE 2 Reliability and 95th Percentile EPDO Crashes for Different Traffic Growth Rate (r) for Future 5 Years

Roadway Segment	95th Percentile EPDO for r = 1%	Reliability for r = 1%	95th Percentile EPDO for r = 3%	Reliability for r = 3%	95th Percentile EPDO for r = 5%	Reliability for r = 5%
1	172.9	0.0135	163.7	0.014	165.1	0.0105
2	4.2	1	4.3	1	4.2	1
3	47.4	0	47.5	0	47.2	0
4	22.7	0.9790	22.2	0.988	22.1	0.984
5	19.6	0.9960	19.1	0.998	19.2	0.998
6	6.7	1	6.9	1	6.8	1
7	3.7	1	3.7	1	3.7	1
8	2.6	1	2.6	1	2.6	1
9	4.9	1	5.0	1	5	1
10	94.2	0.0180	92.7	0.018	91.2	0.0175

A brief description of the calculations is presented here for the roadway segment 1. The 28.561-mi-long segment of MT-508N carries 5-year (2013–2017) average AADT of 227.8 vehicles per day. The calculated MEVM = $5 \times 365 \times 227.8 \times 28.561 \times 10^{-6} = 11.9$ and EPDO/MEVM = $11.83 \div 11.9 = 99.6$. With weighted average EPDO = 23.8, CCR = $23.8 + 1.645 \times \sqrt{23.8/11.9} + 1/(2 \times 11.9) = 26.2$ EPDO/MEVM. Because the calculated CCR is less than 99.6, the actual EPDO/MEVM, this segment requires implementation of crash improvement strategies. In reliability analysis, the bias values calculated for 2013–2017 are 1.09, 2.38, 3.99, 1.76 and 2.91, from which the uniform distribution parameters are determined as 0.507 and 4.345. With the expected average crash frequency determined from SPF, Monte Carlo simulation is used to generate random uniform distributed random numbers for N_{bias} and Poisson distributed random numbers for N_{SPF} , which provide the distribution of N_{Field} according Equation 2. From the distribution of N_{Field} , the 95th percentile value is calculated as 174.4 EPDO/MEVM and the reliability = $\text{Prob}[N_{\text{Field}} < \text{CCR}] = 0.0165$. This reliability is very low indicating immediate attention required for the segment.

CONCLUSIONS

Following conclusions can be drawn from the results:

1. Based on the CCR method, there are three sites that need countermeasures to decrease the CRs. Based on reliability analysis, the same three sites have been identified as potential sites possible for countermeasure actions. The reliability analysis was performed only using SPF-predicted values under base conditions and the observed number of crashes, without using any CMF. Thus reliability analysis procedure provides an alternative and simpler method of ranking sites and determining roadway segments needing countermeasures. Because the reliability analysis procedure combines the SPF and observed crashes, it is not affected by the RTM bias.
2. The 95th percentile values for the EPDO crashes have greater variation than the CCR per MEVM crashes. Because the reliability analysis considers the actual distribution of the variables, the reliability and 95th percentile values should constitute a better designation of crash potentiality of the sites selected. The proposed method considers the effect of uncertainty and RTM bias; therefore better safety improvement decisions can be made using this method than the existing method of using EPDO crashes.
3. The 95th percentile EPDO crashes for forecasted AADT do not change significantly during the analysis period of 5 years for different traffic growth rates.

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Modeling Severities of Motorcycle Crashes on Low-Volume Roadways

AHMED FARID

MAHDI REZAPOUR

University of Wyoming

KHALED KSAIBATI

Wyoming Technology Transfer Center

Motorcycle safety is a key concern for traffic engineers. Motorcycle crashes are more likely to be severe than other vehicle crashes. Although different factors such as crash, human, roadway, and environmental characteristics that influence motorcycle crash severities were investigated in the past, motorcycle crashes on specific types of roadway facilities did not receive adequate attention. In rural areas such as those in Wyoming, which are characterized by challenging roadway geometric characteristics, crashes are more likely to be severe on low-volume roadways compared to those of other roadways. Despite the importance of low-volume roads, research aimed at examining the influence of the factors that contribute to motorcycle crashes at low-volume roads was not conducted extensively. Thus, this study is conducted to fill the knowledge gap by identifying such factors that influence the severities of motorcycle crashes that occurred in Wyoming. Ordinary logistic regression is used for that purpose. According to the results, speed and impairment are factors that are found to increase the risk of severe motorcycle crashes occurring on low-volume roads if all other conditions are the same. Also, motorcycle–animal crashes and motorcycle crashes on horizontal curves are found to be more likely to be severe crashes if all other conditions are unchanged. On the other hand, it is found that wet road surfaces reduce the risk of severe motorcycle crashes assuming no change in all other conditions. The results of this study provide insights to road safety engineers about the severities of motorcycle crashes on low-volume roads in Wyoming.

INTRODUCTION

Motorcycle safety is a key concern for traffic agencies. In 2016, the number of motorcycle rider fatalities was 28 times that of fatalities of other vehicle occupants in the nation (1). Motorcycles account for only 3% of registered motor vehicles and 0.6% of all vehicle-miles-traveled in the United States (2). However, despite this negligible share, motorcyclists are among the most vulnerable road users. They account for 14% of all traffic fatalities in the nation (3). In 2016, there were 5,286 motorcyclists killed in traffic crashes, which represented an increase of 5.1% from 5,029 motorcyclists killed in 2015 (4). Besides, the number of registered motorcycles in the United States increased by about 100% from 2002 to 2017 (4) which may be a reason for the recent high rate of fatal motorcycle crashes.

In Wyoming, there were 229 motorcycle crashes in total, incurring 24 fatalities and 198 injuries in 2016 (5). Four percent of all crashes, regardless of the types of vehicles involved, resulted in fatal or serious injuries, while 34% of all motorcycle crashes resulted in such injuries.

The factors affecting motorcycle crashes and the severity of those crashes abound. Such factors include, but are not limited to over-speeding, especially during light traffic conditions, navigating through tight horizontal curves, mountainous terrain, rain, snow, helmet use, riding under the influence (RUI), rider's sociodemographics, and the motorcycle's conspicuity. That is, motorcycles are less conspicuous to other road users than other vehicles on the road. Researchers investigated such crash contributing factors in general (6–12). However, an in-depth analysis of the factors that contribute to motorcycle crashes on low-volume roads is not conducted. This research addresses motorcycle safety in Wyoming particularly when it comes to low-volume roads. In the following sections, a review of the literature is discussed followed by the illustration of the data used for this study. Then, the analysis methodology is described. The analysis results are presented and discussed following the analysis methodology section. Finally, the conclusion follows the analysis results section.

BACKGROUND LITERATURE

It is worth discussing the recent motorcycle safety studies to identify the research findings and knowledge gap to be addressed by this study. In general, motorcycle crashes are more likely to be severe than crashes not involving motorcycles (13, 14). A review of the critical motorcycle safety studies includes the following resources.

- According to Riffat et al. (10) and Wulf et al. (12), motorcycle riders are more at-risk of being involved in severe crashes than other vehicle occupants not only because of inadequate protection but also because motorcycles are less noticeable by the road users than other vehicles.
- Cafiso et al. (15) and Savolainen and Mannering (11) investigated the human factors that contribute to motorcycle crashes. Particularly, age, gender, residency and history of past traffic violations were considered as some of the factors that contribute to motorcycle crashes.
- Kashani et al. (16) and Li et al. (17) examined the roadway and environmental characteristics influencing motorcycle crashes. They included geometric design, pavement surface types, and weather conditions.
- Vlahogianni et al. (18) and Jou et al. (19) investigated the impact of the motorcycles' physical characteristics on motorcycle crashes. The characteristics included the size and production year of the motorcycle.
- Haque et al. (7) investigated the factors that contribute to motorcycle crashes from 2003 to 2006 at three- and four-leg signalized intersections in Singapore. The number of approach lanes on both intersecting roads, wide medians, absence of the prohibition of left-turns on red, high speed limits, and higher average daily traffic on both intersecting roads were the factors that were found to increase the frequency of motorcycle crashes, given that all else was fixed, according to the results of the hierarchical Poisson model with autoregressive (AR) 1 lag. Note that in Singapore, motorists drive on the left side of the road. Remarkably, it was also concluded that motorcyclists were more self-controlled as they queued behind the stop line and exhibited a delayed response to cross the intersection when their signal turned green if their approach was monitored by a red light-running camera. Without the camera, the motorcyclists

queued beyond the stop line and responded instantly as their signal turned green. Similar conclusions were made regarding three-leg signalized T-intersections. Furthermore, it should be noted that, regarding three-leg signalized T-intersections, major one-way roads were found to be safer than major two-way roads given that all else was unchanged. That is likely because of the reduction in the number of conflict points. Even though the study was conducted on the national scale and thus represented the entire population of signalized intersections instead of low-volume signalized intersections only, the average daily traffic variable was captured and the study uncovered insights about the factors that contribute to motorcycle crashes.

- Teoh (20) analyzed two-vehicle crashes involving a commuter vehicle and a motorcycle nationwide to investigate the effects of using new in-vehicle technologies on motorcycles' safety. The author classified the crashes in categories such that each one represented crashes that would have been prevented by a technological system built in the commuter vehicle. The data were collected from both the national Fatality Analysis Reporting System and the National Automotive Sampling System's General Estimates System. The commuter vehicle technologies were front crash prevention (FCP) systems, lane maintenance (LM) systems, and blind spot detection (BSD) systems. FCP systems warn or apply emergency brakes to avoid forward collisions. LM systems warn drivers that they are deviating off lane centers except when changing lanes. Some LM systems even lock the wheel when veering off as well. BSD systems warn drivers of vehicles in the blind spot if turning into it. According to the results, the FCP, LM, and BSD systems altogether could have prevented 10% of fatal injury, 19% of other injury and 23% of the total two-vehicle crashes analyzed.

As noted previously, the literature is replete with motorcycle safety studies. However, research regarding the safety of motorcycles, particularly on low-volume roads, is limited. This study is a follow-up to Rezapour et al.'s study in which the variables that had a considerable impact on the severity of motorcycle crashes in Wyoming were identified using ordinal logistic regression and classification tree methods (9). Based on the results, posted speed limits and driver maneuverability were found to be the most influential variables that affected the severity of motorcycle crashes in the state. In this study, ordinary logistic regression is employed to model whether motorcycle crashes on low-volume roads are severe or non-severe conditional on the occurrences of the crashes.

DATA

The data used for this study, are those of motorcycle crashes in Wyoming from 2008 to 2017. The data source is the Critical Analysis Reporting Environment (CARE) package of the Wyoming Department of Transportation (DOT). For the crash years mentioned, there are 853 records of motorcycle crashes that occurred on roads with average annual daily traffic (AADT) counts of up to 2,000 vehicle per day (vpd). Even though low-volume roads are those with AADTs of 400 vpd or fewer (21), records of motorcycle crashes on roads with higher traffic counts may be sampled as a criterion for modeling low-volume road crashes. Considering only 400 vpd would result in an inadequate sample size of 79 motorcycle crashes. Besides, the Colorado DOT considers roads with AADTs fewer than 2,000 vpd including fewer than 100 trucks as low-volume roads (22). Hence, for this study, low-volume roads are considered those having AADTs of 2,000 vpd or fewer. Since Colorado is Wyoming's neighbor state, it is

assumed valid to adopt Colorado DOT's definition of low-volume roads neglecting the restriction pertaining to the truck traffic volume condition (100 trucks per day or fewer). Furthermore, the severity levels often used in the studies documented in the literature are fatal injury (K), incapacitating injury (A), non-incapacitating injury (B), possible injury (C), and property damage only, which is also known as no injury (O). Note that the designation of crashes of different severities altogether is abbreviated. For instance, all B and C crashes combined are denoted as BC crashes. It should be noted that the data lack adequate observations of crash records of each severity category: K, A, B, C, and O. Therefore, in this study, the crashes are classified as either severe (KAB) or non-severe (CO) crashes. Descriptive statistics of the variables used are presented in Table 1. The variables are classified as either crash, roadway or environmental characteristics. Note that 94.01%, 3.63%, and 2.34% of the motorcycle crashes occurred on federal aid highways, Interstates, and state highways respectively. Also, in 92.85% of the crashes, the first vehicle of which information was recorded in the police report termed "vehicle 1" in the CARE package, was traveling on a road having pavement markings or traffic control devices. In the modeling, the only roadway characteristics' variables incorporated are whether the roadway is divided or undivided, presence of a horizontal curve, and presence of a wet road surface. Also, the parameter indicating whether a fixed object was struck was incorporated in the modeling but was not found to impact the severities of motorcycle crashes. Fixed-object motorcycle crashes are coded in the original raw data as "delineator post," "trees or shrubbery," "sign support single post," "end of drainage pipe or structure or culvert," "guardrail end," or "other fixed object". That is for both the first harmful event (FHE) and modified harmful event (MHE). From the motorcycle crash records pertaining to low-volume roads, the proportion of fixed-object crashes represent only 5.76% as per the FHE and only 5.64% as per the MHE.

ANALYSIS METHODOLOGY

The ordinary logistic regression technique is employed to model the probability, or risk, of whether the motorcycle crash is a KAB injury crash (23). The probability of crash i being severe is denoted by P_i and is structured as follows.

$$P_i = \frac{\exp(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \dots + \beta_p X_p)}{1 + \exp(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \dots + \beta_p X_p)} \quad (1)$$

The parameters, β s, are the regression coefficients associated with the predictors, X s, which are the crash contributing factors. Note that the ratio of the odds of a severe crash pertaining to one value of a predictor to another is the associated coefficient exponentiated if there are no interaction terms and all other predictor values are fixed. For instance, the ratio of the odds of a severe motorcycle crash, in which the motorcyclist is negotiating a horizontal curve, to the odds of the severe crash of the motorcyclist not navigating through the horizontal curve, assuming all else is fixed, is equivalent to the exponentiated coefficient associated with the horizontal curve-related crash variable. In the presence of interaction terms, the odds ratios vary depending on the values of the interaction variables. The results of the model parameters are interpreted in terms of odds ratios.

TABLE 1 Descriptive Statistics of Wyoming's Motorcycle Crashes at Low-Volume Roads

Variable	Description	Variable Value = 0		Variable Value = 1	
		Frequency	Percent	Frequency	Percent
Outcome Variable					
Severity	1 if KAB injury or 0 otherwise	124	14.54	729	85.46
Crash Characteristics					
Speeding-related crash	1 if yes or 0 otherwise	680	79.72	173	20.28
Lane-departure crash	1 if yes or 0 otherwise	221	25.91	632	74.09
Turning-related crash	1 if yes or 0 otherwise	811	95.08	42	4.92
Impairment	1 if yes or 0 otherwise	616	72.22	237	27.78
Animal crash	1 if yes or 0 otherwise	721	84.53	132	15.47
Single-motorcycle crash	1 if yes or 0 otherwise	624	73.15	229	26.85
Roadway Characteristics					
Paved roadway	1 if yes or 0 otherwise	115	13.48	738	86.52
Major collector	1 if yes or 0 otherwise	483	56.62	370	43.38
Principal arterial	1 if yes or 0 otherwise	529	62.02	324	37.98
Minor arterial	1 if yes or 0 otherwise	743	87.1	110	12.90
Interstate highway	1 if yes or 0 otherwise	822	96.37	31	3.63
Minor collector	1 if yes or 0 otherwise	840	98.48	13	1.52
Local road	1 if yes or 0 otherwise	850	99.65	3	0.35
Undivided roadway	1 if yes or 0 otherwise	36	4.22	817	95.78
Horizontal curve-related crash	1 if yes or 0 otherwise	575	67.41	278	32.59
Wet road surface	1 if yes or 0 otherwise	797	93.43	56	6.57
Environmental Characteristics					
Time	0 if daylight or 1 otherwise	750	87.92	103	12.08
Weather	0 if clear or 1 otherwise	771	90.39	82	9.61
Rural	1 if yes or 0 otherwise	8	0.94	845	99.06

NOTE: Number of crashes = 853.

Goodness-of-Fit Measures

Two goodness-of-fit measures are used. They are the log likelihood and area under the receiver operating characteristic (ROC) curve (23). Each measure is elaborated subsequently.

The log likelihood (LL) is used to describe the deviation between each motorcycle crash severity's predicted probability and the outcome, which is 0 if the crash is non-severe (CO) or 1 if the crash is severe (KAB). For instance, a severe motorcycle crash, having an outcome of 1 and a high predicted probability, will have a high LL . The LL is computed for all crashes and is formulated as follows.

$$LL = \ln\left(\prod_{i=1}^n P_i^{y_i}(1 - P_i)^{1-y_i}\right) \quad (2)$$

For every crash, i , the outcome, 0 or 1, is denoted by y_i . Note that the coefficients, β 's, are obtained by maximizing the LL function and hence are known as maximum likelihood estimates. Typically, negative twice the LL value, $-2LL$, is reported as the model's fit statistic. In addition, the LL ratio test is conducted to assess the influence of all parameters in the model. The null hypothesis is that all parameters have no influence on motorcycle crash severity while the alternative hypothesis is that at least one parameter influences motorcycle crash severity. The LL ratio's χ^2 statistic is computed as negative twice the difference of the model's LL and that of a model estimated with the constant only. The degrees of freedom are the number of parameters in the former model excluding the constant.

The area under the ROC curve is a measure that describes how well the model distinguishes between severe and non-severe motorcycle crashes. Specifically, a large sample of pairs of motorcycle crash records is selected. In each pair, one crash is severe while the other is not. The model is applied to compute the probabilities of both crashes in the pair of being severe. The crashes are classified as either severe or non-severe by the model. The proportion of correctly classified pairs is the area under the ROC curve. An area of 0.5 indicates that the model correctly classified 50% of the pairs and hence the model is a poor classifier. That is similar to the scenario of classifying the pairs based on outcomes of coin tosses. On the other hand, an area of 1 indicates that the model is a perfect classifier.

RESULTS AND DISCUSSION

A logistic regression model is run successfully on the data of the motorcycle crashes at low-volume roads. The confidence interval (CI) chosen for assessing variable significance is that of the 90th percentile. The forward selection, backward elimination and stepwise selection procedures are attempted to select the set of final model parameters in the preliminary analysis. Based on judgment, the backward elimination procedure is chosen for model building. The model results are presented in Table 2 and the odds ratios of the parameters are presented in Table 3. The 90th percentile confidence limits of the odds ratios are presented as well. Interaction effects between each significant parameter and the other are attempted. However, they are not found to influence motorcycle injury severity.

TABLE 2 Logistic Regression Model Results of Severities of Motorcycle Crashes at Low-Volume Roads

Parameter	Estimate	Standard Error	Wald's χ^2	P-Value
Constant	1.262	0.145	76.258	< 0.001
Crash Characteristics				
Speeding-related crash	0.831	0.314	7.007	0.008
Lane-departure crash	—	—	—	—
Turning-related crash	—	—	—	—
Impairment	0.865	0.269	10.364	0.001
Animal crash	0.924	0.330	7.846	0.005
Roadway Characteristics				
Undivided roadway	—	—	—	—
Horizontal curve-related crash	0.518	0.229	5.114	0.024
Wet road surface	-0.947	0.320	8.766	0.003
Environmental Characteristics				
Time	—	—	—	—
Weather	—	—	—	—
Rural	—	—	—	—
Goodness-of-Fit Measures				
-2LL		665.632		
LL ratio χ^2		41.6612		
LL ratio degrees of freedom		5		
LL ratio P-value		<0.001		
Area under ROC curve		0.674		

NOTE: statistically insignificant parameters, at the 90th percentile CI, removed from the model are represented by “—”.

TABLE 3 Logistic Regression Model's Odds Ratio Results of Severities of Motorcycle Crashes at Low-Volume Roads

Effect (1 versus 0)	Odds Ratio Estimate	Lower 90 th Percentile Confidence Limit	Upper 90 th Percentile Confidence Limit
Crash Characteristics			
Speeding-related crash	2.295	1.370	3.846
Lane-departure crash	—	—	—
Turning-related crash	—	—	—
Impairment	2.375	1.527	3.696
Animal crash	2.520	1.464	4.335
Roadway Characteristics			
Undivided roadway	—	—	—
Horizontal curve-related crash	1.679	1.152	2.447
Wet road surface	0.388	0.229	0.657
Environmental Characteristics			
Time	—	—	—
Weather	—	—	—
Rural	—	—	—

NOTE: statistically insignificant parameters, at the 90th percentile CI, removed from the model have odds ratios of 1 and are represented by “—”.

As shown in Table 3, the odds ratios of the variables are computed by simply exponentiating the associated coefficients in Table 2. Regarding the crash characteristics, the odds of severe speed-related motorcycle crashes at low-volume roads are 2.30 times those of severe motorcycle crashes in which over-speeding is not a factor if all else is fixed. On the other hand, lane-departure crashes are not more hazardous than crashes not involving lane departure if all other conditions are unchanged. An example of a lane-departure crash is one in which a vehicle veers off its lane and collides with a motorcycle in the adjacent lane. Similarly, turning-related crashes are not more severe than crashes in which turning is not a factor if all else is the same. A vehicle at an access point failing to yield the right-of-way of an oncoming motorcycle on the main road is an example of a turning-related crash. On the other hand, impairment is found to be an influential factor that contributes to the severity of motorcycle crashes. The odds of a severe crash involving impairment are 2.38 those of crashes not involving impairment or intoxication assuming all other conditions are the same. Motorcycle–animal crashes are likely to be more severe than other motorcycle crashes not involving animals if all else is unchanged (odds ratio = 2.52). Wyoming is known for its elks which may pose serious hazards on the roads.

When it comes to the roadway characteristics, the absence of a roadway median does not influence motorcycle crash injury severity on low-volume roads if all other conditions are fixed. On the other hand, crashes on horizontal curves are more likely to be severe (odds ratio = 1.68) than crashes on tangent roadway segments if all else is controlled. On the contrary, wet road surfaces reduce the risk of motorcycle crash severity (odds ratio = 0.39) if all else is unchanged plausibly because motorcyclists are more cautious when riding on wet roads.

The environmental characteristics which are the time of the crash, weather conditions and area type, rural or urban, have no impact on the injury severities of motorcycle crashes on low-volume roads. That is conditional on the premise that all else is unchanged.

CONCLUSION

Motorcyclists are more at risk of being involved in severe crashes than any other vehicle occupants and hence motorcycle safety is a critical component of road safety. Road safety researchers investigated the various factors that influence motorcycle crashes including human factors, motorcycle configuration factors, roadway features, and environmental features. The reduction in motorcycle crashes due to the use of crash prevention technologies installed in commuter vehicles was investigated as well. However, research regarding the influential factors that affect motorcycle crashes and the severity of those crashes on low-volume roads is limited. In this study, the impacts of the crash contributing factors on the severities of motorcycle crashes on low-volume roads is investigated. The data used are those of Wyoming belonging to the crash years 2008–2017. The ordinary logistic regression modeling technique is employed for modeling the probabilities of the severities of the motorcycle crashes.

As per the modeling results, conditional on the motorcycle crashes on the low-volume roads, speed-related crashes, crashes in which impairment is a factor, animal-related crashes, and horizontal curve-related crashes are likely to be severe motorcycle crashes assuming all other conditions are unchanged. Therefore, more-stringent enforcement policies are recommended when it comes to over-speeding and RUI to reduce severe motorcycle crashes. Contradictory to the effects of the other influential parameters is that of wet road surfaces. Wet road surfaces reduce the risk that the motorcycle crash is a severe crash if all else is the same.

This study is not without limitations. Yet it sets the foundation for future studies. Human factors including the demographic characteristics were not investigated. Parameters such as the rider's history of past violations, age, and gender may be incorporated in the model. The severity levels modeled may be divided into KA, B, and CO. Furthermore, random parameters may be incorporated to take into consideration unobserved heterogeneity effects which are characteristics that describe variables not incorporated in the model (24). Parameters such as helmet use policies and whether the motorcyclists were wearing helmets may be investigated as well. All such topics are recommended for investigation. The effects of RUI and speed enforcement policies on motorcycle crash severity are also recommended for investigation. Also, the combination of speeding, wet road surfaces, and negotiation of horizontal curves, not necessarily on low-volume roads, is worth investigating using simulation software.

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A Tool for Improving the Geo-Coordinates of Crashes for Local Roads

Crash Location Improvement Program

MARIO A. ROMERO

ANDREW P. TARKO

LAURA SLUSHER

ETIENNE ATISSO

Purdue University

INTRODUCTION

Crash location information is a necessary component of safety analysis and decision-making. Unfortunately, its quality is frequently limited, which poses a major problem in road safety management, project development, and safety research. Without reliable location information, crash data may lead to incorrect safety management decisions and to biased research results (1).

According to the 2012 Indiana Five Percent Report, 74% of reported crashes were assigned to the state road network and, consequently, nearly 50,000 crashes could not be used to analyze safety at individual road locations (2). It is also important to mention that the rates of unassigned crashes are different for different types of crash and roads (3). This situation may lead to incorrect prioritization and selection of safety improvements.

Many crashes on Indiana local rural roads included in the state crash database have missing or incorrect geo-coordinates due to the incorrect or incomplete location information. The most frequent cases of faulty location information are

1. An incomplete description of the crash location (missing name of road, distance to the reference intersection, etc.);
2. Missing geographical coordinates;
3. Insufficient precision of reported coordinates;
4. Complete but incorrect crash location description; and
5. Complete but incorrect geographical coordinates.

Cases 1 through 3 are easily detected. Cases 4 and 5 are detected if the associated road information collected by police officers at the crash scene is inconsistent with the road information found in a roadway database for the location with the recorded geo-coordinates.

Currently, a manual checking of geo-coordinates are performed to correct the crash geo-coordinates or to add missing ones. This procedure is time- and labor-consuming and needs to be automated. To help mitigate the problem, Steiner et al. created a custom interface to enter and geocode crash location by extending the capabilities of ESRI geocoders (4). Bigham et al. used a multistep process that included preprocessing the street name information, geocoding using ArcGIS plus StreetMap Pro 2003 and Google Earth Pro, and VBA to incorporate the offset and the direction (5). Unfortunately, these tools are not utilized in Indiana, and probably in many

other states, because local agencies and other users often do not have knowledge or access to GIS tools.

This project aims to develop a convenient method that addresses the needs of local agencies. The developed convenient self-contained computer application supports both fully automated and supplemental semimanual checking and correcting of the crash coordinates without reaching for GIS tools. It uses the police crash reports, available online geocoders, interactive maps, and a database supplemented with street names, addresses, and coordinates.

The developed tool allows agencies to correct location information of most of crashes in the database and to use them in safety analysis. Once implemented, the developed method dramatically reduces the amount of time and effort needed to correct crash coordinates and to assemble data for analysis.

CURRENT METHOD

An Indiana police report provides the crash location information in two formats: numerical latitude and longitude, and (more convenient for human interpretation) a text version that includes the area where the crash has happened, such as county, township, or city, and the nearby reference point such as a mailing address, an intersection, or a milepost. The distance and the direction from the reference point to the crash location are provided to have a complete description of the crash location. Latitudes and longitudes are acquired in Indiana in three different ways—by assigning based on the reported information; from GPS; or by point and click on a base map. Unfortunately, the geo-coordinates source is not recorded in the crash report system so the recorded coordinates cannot be used for consistency checking.

If the crash coordinates are included in the police report, and if the textual information about the crash location found in the police report sufficiently matches the road element information included in the database, then the recorded crash geo-coordinates are considered correct. The sufficient match is concluded if the matching score is greater than a preset threshold. The point-based matching method was optimized for rural local roads to minimize false positives and false negatives.

If crash coordinates are missing or rejected, then the developed tool uses the location description reported by police to find all promising candidate crash locations from both the road network geodatabase and from an online geocoder. As in the previous step, the found candidate crash locations receive number of points that reflects how well the crash location descriptions reported by police match the description included in the road network database, or the description returned from the geocoder. The candidate location with the highest score is selected if the score exceeds the mentioned preset threshold. If multiple candidate reference points receive the same highest score, then the decision is left to the user.

Method Performance

The tool reduces the time of checking and correcting crash location for a typical Indiana county with about 2,000 crashes; from 3 weeks needed in the old manual process to 2 days. Figure 1 shows the difference between the original and the corrected data for local rural road crashes in an example Indiana county. This example clearly demonstrates that any decision based on the original data would definitely lead to an incorrect result.

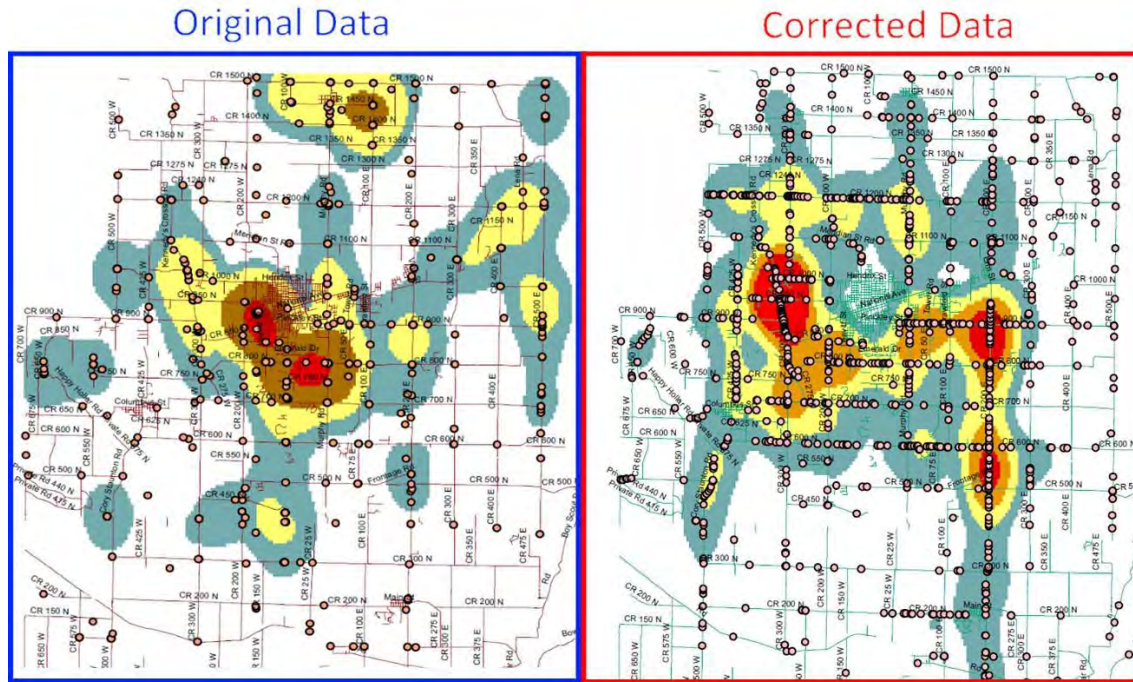


FIGURE 1 Crash location comparison before and after correcting data.

The developed tool was applied to the crashes reported on rural local roads during 3 years in eight selected Indiana counties. The percentage of missing geo-coordinate varies from 3% to 55% from county to county in Indiana while the percentage of incorrect crash geo-coordinates varies from 34% to 55% as shown in Table 1. It is clear that the quality of crash location data is different in different counties.

The percentage of the tool fully automatic matching in rural environment is between 65% and 75%, compared to 20% and 40% in urban areas. This result reflects both the optimization of the point-based matching for rural local roads and the complexity of road naming in urban area. In the second phase of correcting the location data, the user interactively searches for the

TABLE 1 Percentage of Correct, Missing, and Incorrect Coordinates on Rural Local Roads

County	% Correct	% Missing	% Incorrect
County 1	7	55	37
County 2	39	24	36
County 3	40	5	55
County 4	40	14	45
County 5	42	18	40
County 6	52	3	45
County 7	54	11	34
County 8	57	9	34
Average	41	17	41

locations of both the reference point and the crash, and assigns coordinates to these locations with help of visualization maps available via the tool.

In some cases, the lack of information in the crash report makes it impossible to properly assign geo-coordinates to the crash.

Method Improvement

The Bayesian probabilistic approach is implemented to assign crashes to roads based on the one-to-many linking technique used by Tarko et al. (1). The new methodology replaces the arbitrary points with the databased linkage probability, which is expected to improve the automated correction rate, particularly in urban areas.

CONCLUSION

The tool was used to correct crash geo-coordinates in eight sample counties. The results have confirmed that the percentage of incorrect location data differs across counties.

Increasing the completeness of the geodatabase improves the results of the matching. Therefore, transferring road names and aliases plus other data from different sources to the database is a critical condition to improve the crash geo-coordinates assignment quality.

Since it is not possible to automatically locate all crashes, the use of interactive tools with human involvement was implemented. The tool is suitable for users who do not have expertise in GIS.

The point-based approach to estimate the quality of matching is sensitive to the geographic area type and it does not estimate the probability of incorrect crash-road assignment. A probabilistic linkage is considered to estimate the matching quality and to better control the level of uncertainty in the crash assignment to roads.

ACKNOWLEDGMENTS

This work was supported by the Indiana Local Technical Assistance Program and the Indiana Department of Transportation.

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Developments in Low-Volume Roads Pavement Design and Management

DEVELOPMENTS IN LOW-VOLUME ROADS PAVEMENT
DESIGN AND MANAGEMENT

**Applying Pavement Preservation Principles in
Unpaved Road Management with Specific Reference to
Fines Preservation and Dust Control**

DAVID JONES

University of California, Davis

Unacceptable levels of dust are generated on unsealed road networks in most countries. The loss of fines associated with road dust contributes to increased gravel loss and the need for more frequent grader maintenance. By controlling this dust, the rate of gravel loss and maintenance expenditure can be reduced significantly. Research into the performance and benefits of using chemical treatments as part of unsealed road management programs led to the development of multiplier factors that can be used with gravel loss and blading frequency prediction models in unsealed road pavement management systems to determine where these treatments can be used cost effectively. The factors were first validated in a 2-year pilot study before being implemented in a road agency's computerized road management system. Output from the system indicated that chemical treatments could be cost effectively used on at least 20% of the agency's road network, with considerable savings accruing to both the agency and the road user. Subsequent analyses of county road projects in the United States, where chemical treatment programs had been in effect for several years, verified that considerable savings resulted from reduced rates of gravel loss and longer intervals between required grader maintenance. Based on this experience, it is clear that chemical treatments, as part of longer-term unpaved road management programs, can be considered preservation treatments, with additional benefits of dust control leading to safer driving conditions, reduced vehicle operating costs, and improved health and quality of life for people living and working adjacent to the treated road.

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DEVELOPMENTS IN LOW-VOLUME ROADS PAVEMENT
DESIGN AND MANAGEMENT

**Application of New Technologies in Mining
Industry to Low-Volume Roads**

ALEX T. VISSER

University of Pretoria, Pretoria, South Africa

ROGER J. THOMPSON

Mineravia Consulting, Kalgoorlie WA, Australia

RODRIGO PERONI

Federal University of Rio Grande do Sul, Porto Alegre, Brazil

New technologies have been introduced or are being planned for introduction which will have a dramatic impact on-road management and operations on open-pit mines. This initiative is in parallel to many smart road activities on public roads. A mine is a microcosm where outside new initiatives are tested under controlled conditions, and if successful, are applied worldwide. The objective of this paper is to present these new technologies in the mining environment and to discuss the potential implementation for managing low-volume roads networks. Issues that will be discussed are autonomous haulage and road design challenges, premium or long-life low-volume roads, drone-based condition monitoring, drone or smart phone real-time mapping of roads and fleet management systems recording instantaneous vehicle responses during operations. Autonomous vehicle technology will have a potential effect on the future geometric and structural design. Major advances in additives for premium pavements have indicated that this type of wearing course may be cost effective and feasible in future; no unique solution is yet available. Mapping and monitoring technologies such as drones and smartphone data capturing will allow the collection of road geometry, visual records of road defects and maintenance records and provide the road manager with updated information that will allow better decision-making. Data collected by fleet management systems is already available to identify in near real-time road sections that are defective in terms of riding quality.

INTRODUCTION AND BACKGROUND

The approach for managing mine-haul roads consists of dealing with drainage and geometry, the structural capacity, functional performance and maintenance management, which is a standard procedure for public roads as well (1). However, a number of new technologies have been introduced or are being planned for introduction, which will have a dramatic impact on-road management and operations on open-pit mines. This initiative is in parallel to many SMART road activities on public roads. The mining industry is constrained by decreasing resource grades; energy and labor cost increases; stringent safety and environmental controls; and capped capital and working-cost considerations. However, it is also looking into a future of increasing

global demand for many of its products. A mine is a microcosm where new initiatives from outside can be tested under controlled conditions, and if successful, applied worldwide.

The objective of this paper is to present these new technologies in the mining environment, some of which are disruptive (new ways that overturn traditional methods and practices), and to discuss the potential implementation for managing low-volume roads networks. Issues that will be discussed are autonomous haulage and road design challenges, premium or long-life low-volume roads, drone-based condition monitoring, including during emergencies such as flooding, real-time mapping of roads using smart phones and their location and camera capabilities and fleet management systems recording instantaneous vehicle responses during operations.

From the start of the 21st century, mining has evolved from the idea of a “modern” mine, to that of a “real-time” mine and, ultimately, will evolve into an “intelligent” mine (2, 3). Figure 1 shows this evolution and the accompanying development of autonomy, from simple user-interface and monitoring development, with minimal data and analytics, through to more process and analytically complex and data-rich aspects of perception, position, navigation and mission planning and independent equipment collaborative processes; the mining equivalent of the “industry 4.0” concept (4).

MANAGEMENT PROCEDURES OF MINE-HAUL ROADS

The operating requirements of a mine-haul road system can be subdivided into the following four distinct design categories (1), which are also generally applicable to low-volume unpaved roads:

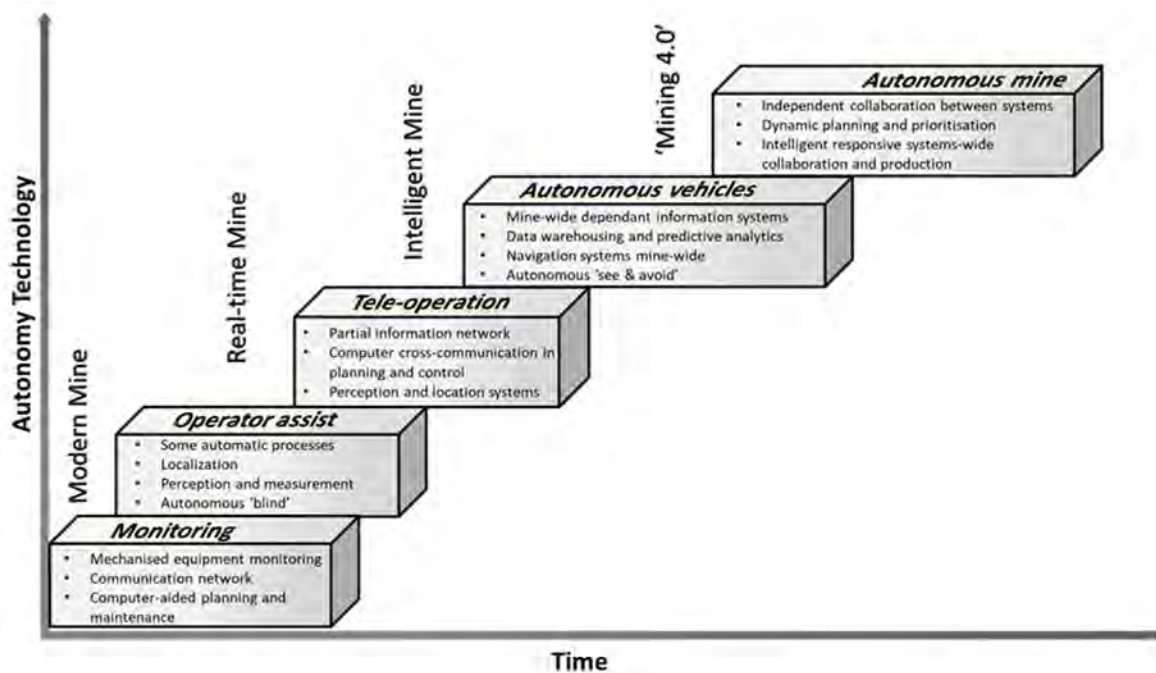


FIGURE 1 Evolutionary phases of autonomous haulage and mining systems [modified after Pukkila and Sarkka (2)].

- Geometric design. This refers to the layout and alignment of the road, in both the horizontal (curve radius, etc.) and vertical (incline, decline, ramp gradients, cross-fall, camber, super-elevation) plane, stopping distances, sight distances, junction layout, berm walls, provision of shoulders and road width variation and should accommodate optimal vehicle speeds, stability, road performance and safety. Drainage design for removal of surface water from the road surface, as well as the control of water in the mine pit is also associated with the geometric design.
- Structural design. This concerns the ability of a haul road to carry the imposed loads without excessive pavement deformation and consequently the need for excessive maintenance.
- Functional design. This refers to the ability of the haul road to perform its function, i.e., to provide an economic, safe and vehicle friendly ride. The selection and management of wearing course materials (or surfacing) primarily controls the functional performance. The use of additives to improve the performance of a wearing course is part of this design category.
- Maintenance design consists of the management and scheduling of maintenance (blading, watering and re-gravelling) of the gravel-wearing course according to the expected and actual road performance.

In the mining environment specifications and requirements of the geometric, structural, functional and maintenance designs exist for the various classes or importance of a road, together with the appropriate construction guidelines to enable the mine to build roads that minimize total (construction, vehicle operating and road maintenance) road-user costs. Any funds allocated to road maintenance (including dust suppression) can only be optimally utilized when the optimal structural, functional and maintenance design have been determined and implemented; the use of any form of dust palliative (including water) on substandard roads would typically lead to poor palliative performance and excessive user cost.

AUTONOMOUS HAULING OR DRIVERLESS VEHICLES

In mining autonomous haulage (AH) applications, the mine truck is best described as semi-connected–fully autonomous; they are autonomous (do not require a human driver) and computer-driven (but without the need for fully connected vehicle technology). This is because the trucks can independently navigate the assigned mine road network, but also operate with (limited) connected vehicle technologies too.

Many mine sites operate mixed (manned and AH) fleets, the autonomous haulage system (AHS) architecture is comprised of both managed machine operation and guidance, together with a fleet management, location tracking and production optimization system. This enables mixed fleet interoperability. To navigate, trucks sense objects around the vehicle using radar and lidar, assess location with GPS–gyroscope combination and create an overall picture of a location, speed, and obstacles. The trucks communicate using auto interaction (especially at dump, load, and intersections) and permission lines monitor speed while interacting with other trucks. This creates a digital safety envelope around all vehicles and a warning when a truck permission lines are projected to cross other vehicle safety envelopes, or obstacles are detected. Currently, no road sensor technology is applied specifically to recognize and adjust vehicle path around a road defect, although all trucks are equipped with suspension pressure monitoring which has been used to infer road conditions (5).

Vehicle–Road Interaction

As the concept of AHS moves from prototypes to production-ready applications, the operating performance of the haul road will become crucial to the overall success of autonomy in mining. In theory, an autonomous truck could operate on a much poorer road and would not be constrained by issues such as operator comfort and the potential for strain injuries associated with a poor road surface; the truck nevertheless rapidly accumulates mechanical strain damage and suffers from premature component failure as a result. Sensing road condition may be the key to minimizing truck damage as speed could be adjusted to limit mechanical damage. Currently, AHS are based on conventionally design trucks and only as trucks evolve could consideration be given to operability and the option of accommodating harsher and significantly rougher road conditions.

As discussed by Thompson (3), with AHS, rapid deterioration in road performance will require costly remediation, human intervention and significant, albeit temporary, changes to operating procedures, to accommodate these types of events. As an example, with autonomous trucking, vehicle path wander is minimal and the road will be subject to high, channelized wheel loads over a limited area. There is no wander as with conventional trafficking, as can be seen in Figure 2, which is what is experienced currently with trucks operating under a trolley assist system where wheelpath wander is minimal due to the requirement to position the pantograph under the power lines. This effect, coupled with the need for reliable and predictable performance requirements, presents challenges in mine road structural design, materials selection, performance specifications and construction. In the mechanistic structural design procedure presented by Thompson and Visser (6), the concentrated loading is handled by using a high expected-performance index that defines the design vertical compressive strains.



FIGURE 2 Example of channelized loading on a trolley assist ramp (reproduced with permission <http://www.metalsnews.com>).

However, with autonomous trucks and channelized traffic comes the opportunity for instance, to reduce road construction and operating width, and therefore generate potentially significantly reductions in stripping ratios and improvements to mine economics. This is only feasible if the design of the road, and its associated deterioration rate, is predictable and manageable, based both on the materials used to construct the road and the maintenance (if any) required to be carried out on the road. Furthermore, high-quality and strength-pavement materials can be placed in the wheel tracks and poorer-quality material outside the wheel tracks that are not trafficked, with concomitant construction and maintenance savings bearing in mind wind and water erosion outside the wheel tracks. Autonomous trucking has many potential advantages over conventional trucking, and to fully leverage these benefits, mine road design and management needs to develop to address the requirements of autonomy in mining.

Looking further into the future of haulage units specifically, Albanese and McGagh (7) anticipate an unconstrained driverless vehicle, uncoupled from the requirements to house and inform a driver of the truck's location, operations, and interactions in the mining environment. A truck that is more symmetrical, allowing multidirectional travel, all-wheel steering, and battery-electric drive systems with power- and energy-storage systems on-board under-body. This ideal combination of market-pull and manufacturer-push is leading to some of the new concept vehicles being built currently, such as the Komatsu IAHV as shown in Figure 3.

Experiments are underway at mines with autonomous vehicles which will allow testing of general GPS guidance systems, including evaluation of vehicle space interference and vehicle to vehicle communication. Smart car research is also testing this technology. This is important for defining road width specifications, including single lanes and passing bays, as there could be significant cost savings in road construction and maintenance. Furthermore identification of road defects would permit reduction in speed to avoid loss of vehicle control and mechanical damage to the vehicle. Smart car technology is already able to identify a tortoise on the road.



FIGURE 3 An example of an autonomous haul truck (reproduced with permission <http://www.awesomearethmovers.com>).

Tire Considerations

One of the major vehicle associated costs on open-pit mines is that of tires. Invariably tires do not wear out, but are damaged by rocks cutting the tread or sidewall, or because of exceeding the limiting tire tonne-kilometer per hour. Inflatable tires require pressures to be maintained to minimize damage to the truck and the tire itself and under certain circumstances, failure of hot, pressurized tires can be violent.

Non-pneumatic tires are being patented to match the capabilities of the vehicle and Figure 4 shows an example of a U.S. patent application, where the impacts applied to the tire are absorbed by a rubber lattice sidewall (U.S. Patent Application 13/954,504 dated July 30, 2013). The potential implication of a tire of this nature on a haul road is fortunately minimal. If the tire contact area is the same as the current pneumatic tires, then for the same load the contact stress between tire and road surface will be the same, and there will be no unexpected road damage although tire pattern may affect the contact stress distribution and wearing course performance.

Improvements in tire technology, particularly moving to non-pneumatic tires, would have minimal effect on the unpaved low-volume road structure design provided the contact stresses are similar to tires currently in use. Tire contact pattern may change and this may cause local higher stresses. Note that tire pressure affects primarily the upper 100 mm of a pavement, especially the surfacing, whereas wheel load primarily affects the lower pavement layers.

Road Geometry Considerations

One of the benefits of mine AHS is associated with the highly channelized traffic which is accomplished using control systems that place the truck location accurately on the road. This leads to a potential reduction in road width, such that the road width geometrics can be re-specified to accommodate this greater positional accuracy. Initially, road width can be reduced to



FIGURE 4 Non-pneumatic tire for use on haul trucks and loaders.

approach $2W$ (W is vehicle width) for two-way traffic, from the current $3.5W$. The validity of this approach would require reference to the truck control systems, such that an oncoming vehicle is not seen as entering another trucks exclusion zone when it approaches to pass in the opposite direction. A benefit of reduced road width is that the pit size can be reduced and the economic performance of a mine improved.

This modification can be further refined by designing the layer works of the road specifically for laden and unladen routes, with a thinner structure being applied to the unladen segments of the haul network, as shown in Figure 5. Again, with the facility to exclude specifically all laden vehicles from the unladen carriageway, the adoption of this design approach is feasible. Invariably laden trucks have priority, and unladen trucks could be scheduled to minimize waiting time using existing fleet management systems.

In the mining environment experience with channelized traffic without wander on trolley assist ramps has shown that it is possible to structurally design a pavement by reducing the design vertical compressive strains in the pavement layers. Taking this finding to public roads would mean that for low-volume roads, the focus of this paper, there would be minimal impact on the unpaved or even paved road structure. On roads carrying large numbers of truck traffic the degree or width of wander can be reduced or the design requirements for stresses and strains be adjusted. This approach has been successfully used on LVRs carrying truck and more than two semi-trailer combinations (Australian road trains) where the wheels follow in the same track. A single lane with passing bays shown in Figure 5 may not be practical on public roads since movements along a road are random, unless some form of platooning is applied. Platooning would mean that groups of vehicles would move together.

PREMIUM OR LONG LASTING PAVEMENTS

Open-pit mines have historically used unpaved roads, even for life-of-mine roads, as rocks falling from the haul truck would damage and pothole a road. Spillage is normally removed by

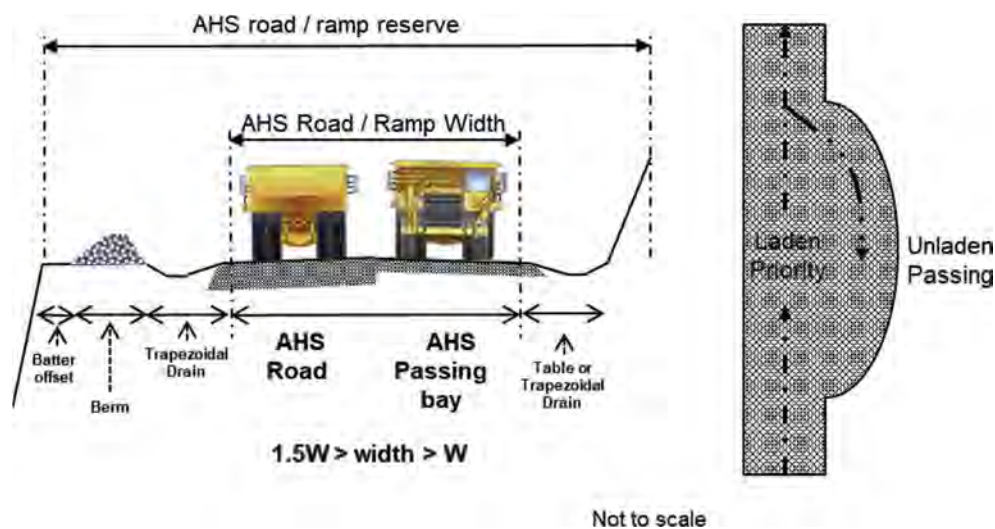


FIGURE 5 Adoption of single-lane roads and passing bays for AHS.

motor grader and this would result in damage to a permanent surfacing such as asphalt. However, there are significant financial benefits in providing a good structure and an appropriate wearing course, as this will reduce the maintenance costs such as grading and watering in addition to improving productivity of the haul trucks themselves.

There is an international trend to provide a premium pavement through mixing in various chemical additives, with a view to realising the ultimate goal of any unpaved mine road: 24–7 traffic ability. There is no general solution as the premium pavement has to be constructed with locally available materials and the chemical additive has to be compatible and locally available. The premium pavement is durable and produces a minimum of dust. All motor-grader maintenance is stopped as such activity would damage the hard crust, as well as water sprays, which cause erosion and potentially leaching of some additives. Since there is a minimum dust, a rotary broom is effective to sweep the dust off the road. Alternatively, a truck mounted industrial vacuum cleaner as is used on alluvial diamond mines can be used. An occasional diluted rejuvenation spray is applied to reinforce the surfacing and counter degradation. A careful loading procedure is applied to minimize spillage, and any spillage is removed either by rotary broom or by means of a hit-squad with a small rubber tired loader.

Figure 6 (upper left side) shows a 10% grade ramp road wearing course that has withstood three rainy seasons (1,400 mm per year average rainfall) without any degradation or defects and a rolling resistance [obtained from a visual inspection procedure (8)] of 2% after



FIGURE 6 Close up of a premium pavement (upper left) after three rainy seasons, compared to an untreated pavement (lower right) with considerable unbound material because of loss of fines after rain.

3 years which is excellent. Traffic is about 100 kt per day or about 400 haul trucks. Minor abrasion has taken place. In Figure 6, the premium pavement (upper left) is compared to an untreated pavement (lower right) with considerable unbound material because of loss of fines after rain even after a few months. Although the capital expenditure in constructing the road initially may be daunting, the mining company has recouped the initial construction costs within a year through savings of routine maintenance and water spraying, besides the increase in productivity and reduction in vehicle maintenance. After the first year the savings become profits.

Roads engineers, like alchemists, have been searching for the pot of gold. The ultimate solution would cheaply produce an all-weather unpaved road providing desired service with minimal maintenance and no dust or mud. Materials technology is advancing rapidly, and the example of a ramp road lasting for 3 years without any motor-grader maintenance or water spraying shows that this is becoming feasible. The only maintenance that was necessary was sweeping spillage off the road by rotary broom and an occasional surface rejuvenation. In the mining environment, the mine receives the direct benefit of the investment, and it was found that the capital costs were recuperated within 1 year. In the public roads environment the costs are incurred by the road authority whereas the benefits accrue to the road user. At present costs of additives are still expensive, although there are reductions in price that are becoming evident as for example nanotechnology reaches the market. It is not yet possible to provide a single solution for all types of wearing course as preliminary research with nanomaterials has shown that clay chemistry, which is not normally tested, is becoming important (9).

DRONE-BASED CONDITION MONITORING

Drones or unmanned aerial vehicles (UAVs) are used in a wide range of commercial roles ranging from search and rescue, surveillance, monitoring, firefighting, photography, videography and even delivery services. Originally, drones were unmanned aircraft systems that were used to access locations that would be difficult or dangerous for a human. More recently, this has expanded to replacing or supplementing data gathering, irrespective of ease of access, as in many circumstances, their data gathering potential and efficiency is superior to survey teams. By integrating UAVs with on-ground equipment, Komatsu (10) use UAV data and artificial intelligence to manage construction sites and equipment almost fully autonomously. The system combines Komatsu's intelligent Machine Control (iMC) technology with drone-generated data to automate (initially foundation) and other civil construction work, but could easily be expanded to mine-haul road construction and, ultimately, maintenance. The UAV is also capable of performing aerial surveys.

The aerial survey produces, with photogrammetry, an ability to generate a dense point cloud after processing the data captured by the onboard camera. Figure 7 presents the results of the survey in a projected view of the point cloud coloured from the orthophoto created.

Peroni discusses how digital elevation models (DEM), contour plots and cross-sections (longitudinal and transverse) are used to analyze road profiles and general geometrical compliance (11). Figure 8 shows a cross section of a haul road. A 2.2-m-high safety berm (lower right side) is a suitable size for the largest truck in operation at the surveyed mine (Komatsu 730E with 3.42-m diameter tires). Berm height must be at least two-thirds of the tire diameter. The actual berm height in this case corresponds to the recommended berm height; however, as seen in Figure 8, the road width of 12.4m is below the recommended minimum 3.5W and is a potential safety hazard.

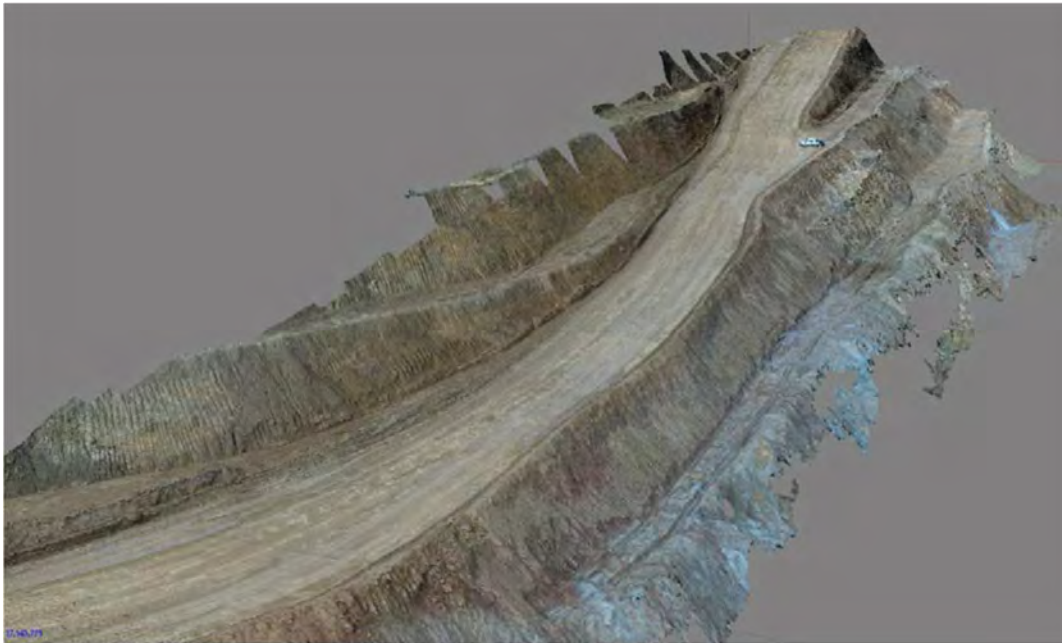


FIGURE 7 3D view of the road surveyed with photogrammetry.

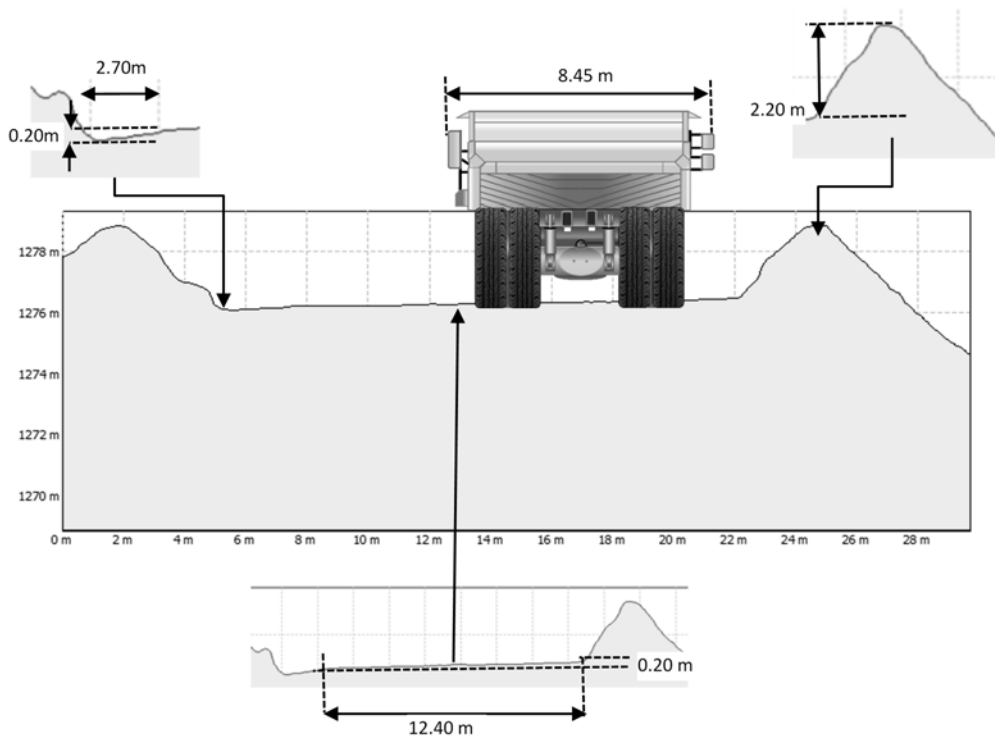


FIGURE 8 Details of the geometrical elements at transverse section.

The transverse slope directing the drainage to the inner side of the road has a slope of approximately 1.6%. The recommended slope is between 2% and 3% to promote rapid drainage of surface water to the roadside drains with greater efficiency and speed (but still minimizing scour), especially during periods of higher rainfall.

After analyzing the main geometric elements detected in the survey, the actual measured conditions are compared with the mine's recommended standards for this type of project, as presented in Table 1. This provides ready feedback to management.

There are two ways in which the quality of the road surface can be evaluated by means of a drone survey, namely by visual or machine-learning evaluation of the photographic evidence, or by means of computing the riding quality (*I*). This information may be used to do the following:

- Assess the performance of a road; and
- Assess the impact of the road surface condition on the operating costs of vehicles using the road.

Both techniques are still in their infancy, but they show potential. Visual assessment of road quality is a slow process and not yet suited for real-time implementation. The International Roughness Index (IRI) is used on public roads and laser elevations are processed by a computer package (12) that simulates the vertical accelerations of a passenger vehicle with standardized characteristics, the so-called golden car. It was possible to generate the IRI from drone data, but currently the resolution and repeatability of drone measurements in the vertical axis is about 30 mm, whereas the profile resolution has to be less than 2 mm. Once more precise and higher resolution data becomes available this approach can be applied.

Drone technology has made it possible to rapidly survey a mine. For example, the flight over an 80-hectare mine was achieved in a morning, and processing took an afternoon. A land survey team would take at least 3 months to do the same survey. The drone does not affect operations and does not interfere with production. This would be a useful application on public roads, and recent applications have seen visual inspections from the photographs being performed on heavily trafficked freeways where traffic volumes do not permit the use of even a camera van. Legislation regarding the use of drones in public space, as well as insurance requirements in case of causing an incident are issues to be remembered. Future developments regarding precision of elevation measurements will make road roughness surveys feasible.

TABLE 1 Description of the Geometric Elements Taken from a DEM at Selected Transverse Section

Geometric Design Checklist	Measured	Recommended	Checklist Finding
Berm height (m)	2.20	2.26	✓
Road-trafficked width (m)	12.40	3.5W = 29.6	✗
Longitudinal grade	6%	Max. 10%	✓
Transverse slope	1.6%	2% to 3%	✗
Cross-sectional area of drain (m ²)	0.27	0.40	✗

REAL-TIME MAPPING USING SMART PHONES

The mining environment is highly dynamic with changes in the haul road network taking place on almost daily basis in the production areas. It is normally a fulltime task of a survey or safety team to keep up with developments. In cases where road conditions, alignments, drainage and similar on-road features are sought, smartphone hosted open-source applications such as Mapillary (13) can be usefully applied to map and record road conditions for further processing, image or feature recognition or simply for road maintenance diagnostics. Figure 9 shows a typical application in which a section of a road network has been recorded, geolocated on a map of the location or mine site and then subsequently used in this case to assess variations in elevation and thus grade of the road. The approach can have further application in road condition monitoring by applying the functional assessment methodology discussed previously in conjunction with machine-learning image recognition techniques to both identify critical functional defects and, ultimately, determine rolling resistance. Smartphone evaluation of riding quality is also possible as a Class 4 instrument (14) as defined in the IRI documentation (15) and needs refinement as in the case of drones.

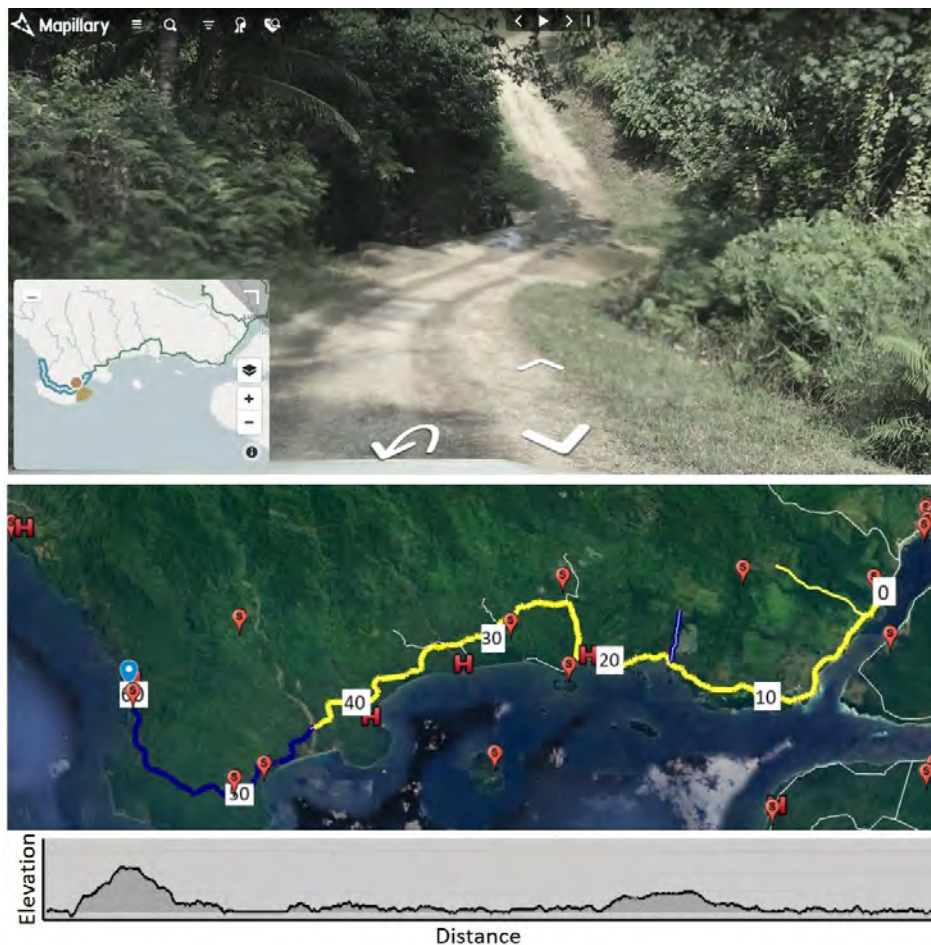


FIGURE 9 Example of road diagnostic recording by smartphone and analysis using Mapillary software.

The capabilities of smartphones are being used in a number of road management applications. Maintenance teams photograph maintenance performed and this is linked to the asset management data base through GPS location. In the mining environment the use of this smartphone capability allows routine monitoring of the haul road network, as well as for interim positioning of short-term roads. On the public road network this facility is useful in areas where road locations have not been surveyed, or where heavy tree cover does not permit use of aerial survey techniques. It is particularly useful for pre-feasibility evaluation of road improvements in developing countries.

FLEET MANAGEMENT

Fleet management systems, commonly known as dispatch systems in mines, evolved from the 1970s using manual batch allocation and radio communications, to today's wireless systems which direct and control equipment from source to destination using real-time positioning, speeds and traffic volumes all relayed to a data center (16). The data center might be located remotely and the entire haulage operation managed by a few key staff enabling immediate communicating and real-time interventions back on site.

A fleet (vehicle) management system can include a range of functions, alerts and triggers according to the specific need of each operation. Fleet management data can include vehicle operating hours, vehicle telemetry (tracking and diagnostics), driver performance, speed control, fuel consumption and health and safety management. The system facilitates identifying improvements in efficiency, productivity and the opportunities for risk reduction associated with vehicle investment, for haulage fleet operators. Data collected by a dispatch system can be used to improve the performance not only of the truck fleet and mining equipment in general, but it can also generate information to detect haul road problems and indicate the need and prioritization of correction actions for the maintenance crew, as part of an integrated haul road management system. Besides, for operations the data systems can also be used for construction.

Information on truck speed, suspension vertical accelerations and unplanned stops during the operating cycle can indicate problems with the road or can even be used to help to back-calculate rolling resistance for sections with homogenous geometry and speed. The number of events from a single shift of operation can be enormous. Consider for example a fleet with 20 trucks recording events at every 10 s which represents around 57,600 events registered in a single 8-h shift across all mine roads, origins, and destinations, etc., as can be seen in Figure 10. Often the number of events is so large that the practicality of analyzing this data to extract information and then recognize road problems, react and correct the road defects in real time can be difficult without mature big-data analytics. Furthermore, highly trained and experienced staff are required to both recognize the appropriate and relevant data, compile the analytical routines and provide suitable feedback to the operation, as often the analytical processes themselves are limited to predefined outputs. This remains a challenge.

Recording the responses of vehicles in a mining environment is theoretically feasible but nevertheless challenging as haul truck manufacturers are reticent in sharing some commercially sensitive performance data. Slow progress is being made in this regard. For highway trucks this is a challenge as there are many manufacturers. Limited access to information on the onboard computer is legislated in the United States through a specified output plug, which can be connected to a laptop computer. The manufacturer of a well-known German brand of trucks



FIGURE 10 Example of events collected from the truck fleet by the dispatch management system.

maintains a cloud database of all the operating information of their vehicles worldwide. Owners may request access to information for their vehicles. This information on a real-time basis has been successfully used to identify locations on unpaved roads requiring maintenance to ensure the integrity of tomatoes transported from the farm to the market (17), as well as for some contractor-mining operations too. There is thus significant capability for using fleet management data to enhance operations.

CONCLUSIONS

There are a number of smart technology initiatives in the mining environment that hold valuable implications in the public road sector, particularly low-volume roads. These include autonomous vehicle technology that will have a potential effect on the future geometric design, and even potentially single lane roads with passing bays. Structurally limited wander would require a refocus on channelization and the potential for non-pneumatic tires on the stress and strain development within a pavement structure. Fortunately, indications from similar limited wander situations are that this would not be a major issue, as it has been catered for in trolley assist lanes. Major advances in additives for premium pavements have indicated that this type of wearing course may be cost effective and feasible in future.

Mapping and monitoring technologies such as drones and smartphone data capturing will allow the collection of road geometry, visual records of road defects and maintenance records and provide the road manager with updated information that will allow better decision-making. Data collected by fleet management systems is already available to record road sections that are defective in terms of riding quality and damage vehicle and goods carried. Reactive and ultimately predictive maintenance can then be focused on defective roads, which will result in the best application of scarce resources.

AUTHORS' CONTRIBUTION STATEMENT

The authors have worked in the field of mining haul roads for a number of years. The joint experience was pooled. Alex T. Visser produced the draft document to which Roger J. Thompson and Rodrigo Peroni contributed.

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Education for Low-Volume Road Engineers

Training and Education for Forest Roads

Use of Experiential Learning with Forestry Students within Coursework at a Land Grant University

W. MICHAEL AUST
Virginia Tech

Forest roads are a vital component of forest products transport, yet forest roads have potentially significant environmental and economic costs. Thus, solid training regarding design, layout, construction, and maintenance of forest roads is an important asset for many forest managers. Although the importance of forest roads is well-documented, forest road training is often minimized during undergraduate training, and may be relegated to brief overviews as part of forest harvesting or silviculture courses. The Department of Forest Resources and Environmental Conservation at Virginia Tech has a forest operations course—Forest Boundaries and Roads—that is dedicated to basic surveying and road planning and construction. Portions of the roads-related curriculum also are offered as workshops to forest managers tasked with overseeing and using forest roads, who commonly seek postgraduate continuing education opportunities. The purpose of this manuscript is to outline topics and techniques used to deliver forest road curricula to undergraduates, graduate students, and continuing-education postgraduate students with a variety of traditional classroom lectures combined with field exercises. The course relies heavily on experiential learning techniques including hands-on road layout, collaborative learning, cooperative learning, field trips, and service learning. The use of experiential learning requires considerable resource inputs as compared to traditional lecture formats, including increased time for grading, increased travel costs, and increased personnel needs. Student, employer, and workshop feedback indicates that the experiential learning component allows many students to acquire a solid grasp of materials.

INTRODUCTION

Forest operations apply silvicultural manipulations to achieve specific objectives, thus forest operations include site preparation, harvesting, and consideration of forest transportation infrastructures such as roads, decks, skid trails, and stream crossings (1). Activities such as planning, design, layout, construction, and maintenance of forest roads, skid trails, and stream crossings are necessary to ensure that forest roads provide desired access within budgetary constraints, while minimizing potential problems (2, 3). Roads, decks, and skid trails are vital components for forest operations; however, these access infrastructures have significant potential to affect both environmental and economic costs of silvicultural activities disproportionately (4).

Potential environmental costs of forest roads have been documented by Laurance et al. (5) and include factors such as loss of habitat, wildlife endangerment, invasive exotic species expansion, increase of unauthorized trespass, and potential degradation of water quality and instream wildlife habitat. For many forestland managers, potential water quality degradation is a

pressing environmental concern associated with forest transportation infrastructure because of state and federal water pollution policies and regulations (6). Forestry water quality best management practices (BMPs) are used to minimize water quality influences and state developed BMP guidelines focus on forest roads, skid trails, and stream crossings (2, 7, 8).

Potential costs of forest operation infrastructures have not been documented extensively and costs vary dramatically with road class, permanence, geology, topography, stream crossings, desired traffic, and other factors. A “typical” Class I permanent forest roads with high standards might cost over \$62,000 per kilometer (\$100,000 per mile), while temporary bladed skid trails may cost \$3,100 per kilometer (\$5,000 per mile). Potential economic costs of roads, skid trails, and stream crossings can be estimated with a variety of techniques, including machine cost, case studies, and modeling (9). A variety of road costs models exist than can be used to estimate costs (10, 11). In some situations, road costs, including expenditures for BMPs, can be prohibitive to profitable harvesting operations (12).

The literature clearly indicates that forest infrastructures are important considerations for forest management from access, environmental, and economic perspectives, yet relatively few current forestry curriculums have courses devoted to forest roads (13). Sample et al. conducted a survey of forestry educators, employers, and graduates and found that water resources were considered as being the most important consideration for a forester, however water was also found to be an area that received lower rankings for student preparedness (14). Planning for forest roads and use of BMPs during and after construction are primary techniques that can be used to minimize impacts of forest operations on water quality (7). The importance of water quality protection, combined with minimal training in forest roads explain why foresters often attend postgraduate continuing education workshops or short courses on topics such as forest roads and stream crossings.

Forest roads, as with many forestry and related natural resources curricula, have long traditions of being taught with some form of experiential learning, extending back to the early 1900s with Carl Schenk at the Biltmore School of Forestry (15). Forest educators have called these exercises by a variety of names including field exercises, field labs, summer camps, spring camps, etc., but experiential learning is the term more widely used in pedagogy literature. Kolb generally is acknowledged as being one of the leading authorities on experiential learning and he characterized experiential learning as “learning from life experiences” and described somewhat related terms including internships, field projects, service learning, problem-based learning, action learning, team learning, and life-long learning (16). Kolb cautioned that there is considerable confusion and perhaps misuse of the terms and emphasized that experiential learning included multiple stages beginning with experience, and subsequently progressing to reflection, abstract conceptualization, and experimentation. Millenbaugh and Millspaugh (17) used an experiential learning process to provide wildlife majors with field experiences. They outlined advantages of experiential learning as including greater retention of material, higher enthusiasm for the subject, knowledge of professional skills, exposure to multiple learning styles, and real-world situations in a safe environment. Another advantage is the students may learn critical thinking and problem-solving skills more readily than in a traditional classroom setting. They cautioned that experiential learning exercises may be interrupted by unexpected events, such as weather, yet such events can be used as teaching opportunities and confidence builders. Another potential disadvantage of experiential learning is that it may be difficult to complete all planned exercises as anticipated.

Millenbaugh and Millsbaugh (17) concluded that multiple advantages exist for use of experiential learning for certain wildlife courses, but also reflected that experiential learning classes require additional preparation and planning, logistical support for travel and equipment, and require additional personnel. Brown reviewed forestry education in Europe and advocated the incorporation of additional problem-based learning exercises to supplement and strengthen traditional lecture curricula (18). Hsiung evaluated incorporation of cooperative or team learning exercises for homework and exams within engineering curricula and initially found that students' grades did not immediately improve as compared to individual assignments (19). However, over time, grades improved as students became more comfortable with protocols. Ramson explained the use of service learning projects, which linked students in a law class to community service projects and requires reflection on the experience (20). Evaluations of student feedback indicated that such opportunities allowed students to apply and master information from the classroom, allowed the students to learn and develop workplace skills, and enhanced students' appreciation for volunteerism. DeGiacomo evaluated potential benefits of experiential learning for forestry students and concluded that opportunities such as cooperative education could provide students with alternative learning opportunities, better understanding of workplace duties and expectations, better employment opportunities, mechanisms to finance education, and chances for students to improve both soft and hard skills (21). Additionally employers may gain a pool of partially trained personnel, reduced recruiting costs, providing bridges to academia, and opportunities for professional mentorship.

Quesada-Pineda incorporated experiential learning into a course for natural resource students (water, wildlife, forestry majors) with a combination of lectures, field trips, student journals, student presentations, and student papers (22). Generally, they concluded that complexities of natural resources and natural resources problems require multiple teaching approaches, including both traditional and experiential learning. Walker et al. also emphasized that natural resources problems and issues are messy, complex, and controversial, thus are best approached through collaborative learning, which includes experiential learning, ecosystems thinking, negotiation skills, and participatory communication (23). Sample et al. (14) and Sharik et al. (24) evaluated natural resource education and employers needs and found that employers are seeking creative and innovative employees with solid professional training, ethical grounding and soft skills such as communication and team work. Felder concluded that the optimum teaching style for engineering students would include classic classroom lectures and memorization (deduction) combined with problem-based learning, discovery learning, and inquiry learning (induction) (25). Felder explained that a balance of concrete information and facts might be more readily absorbed by sensory students, while problem-solving skills are more appealing to students having intuitive–reflective learning styles.

The goals of this paper are (1) to provide a general overview of a forest roads course curriculum provided by the Virginia Tech Department of Forest Resources and Environmental Conservation and (2) to explain the experiential learning techniques that have been used to enhance student learning experiences. Such information could be useful to instructors or organizations wishing to develop similar courses and this manuscript presents one course as an example of how experiential learning techniques have been incorporated into forestry education for decades.

METHODS

Course Delivery and Logistics

The undergraduate curriculum related to forest roads represents 8 weeks of a 14-week semester course entitled *Forest Boundary and Roads* (FREC 3724), whereas the first 6 weeks are devoted to forest boundary surveying. The skills learning in the first 6 weeks (e.g., use of surveying equipment) are subsequently applied to the forest roads portion. The course is designed for juniors and seniors and is occasionally taken by graduate students as a special or individual topics course. The portion of the course focused on forest roads contains two 50-min lectures per week (16 h total on forest roads) and eight 4-h (32 h total on forest roads) field exercises.

Student performance is gauged by performance on exams, homework assignments, field lab memorandums, group lab exercise reports and presentations, and participation in field exercises. There are three exams. The first two exams are take-home open-book exams that are due 1 week after assignment. Students may use any resource at their disposal, but they cannot communicate with any persons other than the teaching assistant and instructor. The Virginia Tech Honor Pledge is in effect at all times during exams. The take-home open-book exam format provides students with additional time to consider and reflect on questions and applications that represent “real world” situations. This exam format also provides students with extra time to synthesize information from field labs, lectures, and readings in order to provide thoughtful answers. On the first exam, some students perform poorly simply because they are not accustomed to the exam format, synthesizing data, and developing detailed answers. However, subsequent grades improve and overall grades are similar to those in other junior senior level classes. The final exam is based on a PowerPoint presentation in which approximately 100 images of forest roads are presented with an associated applied question. For example, a slide might depict a stream crossing for a log truck and ask what type of additional best management practices are needed. Homework assignments are used to supplement lecture presentations. For example, following presentation of road curve calculation formulas and one example calculation might be followed with a curve calculation homework assignment. This allows students to acquire the basic information before applying it during a road layout field exercise.

Eight field exercises (4 h each) use a variety of experimental learning techniques. Some are field trips to active logging roads that allow the student to see newly constructed forest roads and BMPs used for water quality. Others field exercises are designed so that student crews evaluate an existing road and collect data such as soil erosion, and BMP implementation rate. Other exercise allow students to layout a forest road. Some exercises will allow each crew to work independently while others may consolidate all crews’ data. Experiential learning requires contemplation; therefore students are required to write a post-exercise memorandum that outlines the major purposes and methods of the lab and to explain the relevance of the exercise to the forestry profession. At the beginning of the semester, students’ level of experiences are used to place them in three-person crews that provide some diversity of education (e.g., seniors and junior) and backgrounds (e.g., forest resource management and forest operations). These crews work as a team and build team skills within their crew and between the other crews (cooperative learning). Student teams develop and present final road plans and profiles, erosion control measures, and cost estimates. Much of the work for the final report requires calculations and analyses of data they collected (various distances, directions, volumes, depths, and costs) for development of the final plan and profiles, road construction plan, and cost estimate.

Service learning is included as a component of the course as opportunities become available. To date, 10 classes have designed and located private roads that were eventually constructed. The students are motivated to perform to high standards when they see the products of previous classes. Additionally classes have located boundaries for 12 private landowners. Previous class designed roads are visited and critiqued by current classes.

Graduate students who opt to focus on forest operations commonly select this class as an independent study. In order to achieve graduate credit they must complete all undergraduate class exercises (field exercises, homework, exams) and they must additionally complete a smaller assigned project. During 1 year, four graduate students were enrolled so they were assigned a larger project of updating an earlier road cost method. They did such professional work that the revisions were published as a peer-reviewed journal article on forest road costs (9). More recently, another group of graduate students conducted a survey of loggers regarding road BMP costs, which was published as an extension paper (26). Although the graduate expectation changes yearly as opportunities change, all graduate students must collect data and analyze some type of real-world road issue.

The continuing education programs have been designed to serve a variety of clientele, but the shortest workshops have been delivered in 1 day, with a morning indoor lecture session followed by an afternoon outdoor experiential learning session. More commonly, continuing education activities have been delivered in three 4-h indoor sessions (mornings) and three 4-h (afternoon) outdoor sessions. Continuing education workshops have been specialized for various offerings to personnel with the U.S. Forest Service, Virginia Department of Forestry, Tennessee Department of Forestry, Georgia-Pacific, MeadWestvaco, and other organizations. All continuing education programs were individualized to reflect education and experiences of the attendees. One advantage for these types of continuing education experiences is that attendees have wide experiences that they share with each other and the instructor.

Course Topics

The topics covered by the course are similar to other proposed curricula or manuals for forest roads (27–30). Major topics typically involve two indoor lectures and one field exercise. Indoor and outdoor sessions are planned so that indoor lectures provide the background and principles that will facilitate more efficient application of theory in outdoor field sessions. The following sections provide justification, lecture topics, and lab exercise for weekly modules. The continuing education exercises can be modified to fit various class or workshop times and emphases can be changed to fit educational levels and interests. Table 1 provides examples of topics and time used for traditional lectures and experiential learning exercises for a traditional university offering and two workshops of shorter duration. For workshops, a ratio of 1:1 h of indoor versus field exercises has worked well, where undergraduate students may benefit from additional traditional preparation prior to field exercises (1:2 ratio).

Topic 1: Road Planning Choices: Road Classes, Road Standards, And Road Templates

During the first week, students are exposed to the necessities of forest road infrastructure and factors that should be considered during road plans (road purpose, road traffic, road longevity,

TABLE 1 Example of Classroom Topics and Experiential Learning Field Exercises Adapted from a University Course to Two Different Continuing Education Workshops

	Indoor Topic <i>Field Lab Topic</i>	University Course (hours)	3-Day Continuing Education Workshop	1-Day Continuing Education Workshop
1	Road Planning Choices: Road Standards and Classes <i>Road Classes and Standards</i>	2 4	1 0.5	0.5 0.25
2	Road Planning and Reconnaissance <i>Grade-Line Installation</i>	2 4	2 4	0.5 0.75
3	Grade-Line Adjustments and Construction Techniques <i>Grade-line and switchback layout</i>	2 4	1 0.5	No No
4	Road Curves, Choosing and Appropriate Curvature and Methods of Curve Layout <i>Calculation of Curve Components and Installation of Curves</i>	2 4	1 1	No No
5	Stream Crossings and Water Control Structures <i>Examination and Evaluation of Multiple Crossing Types</i>	2 4	2 2	1 1
6	Erosion Control (Vegetation and Surfacing) and Best Management Practices <i>Field Estimates of Erosion Rates Relative to BMP Implementation</i>	2 4	3 3	1 1
7	Road Construction and Road Costs <i>Examination and Cost Estimation for Existing Roads</i>	2 4	2 1	1 1
8	Road Design <i>Develop Road Plan and Profile, Construction Plan, Cost Estimate</i>	2 4	No No	No No
	Total hour of indoor instruction (traditional classroom)	16	12	4
	Total hours of experiential learning field exercises	32	12	4
	Ratio of traditional lecture hours to experiential learning hours	1:2	1:1	1:1

road costs, road impacts, etc.). Students are exposed to basic road categories, ranging from high standard, permanent forest roads to low standard temporary skid trails. Road standards that are emphasized include road entrances, road grade, road width, road template (prism), road subgrade, road cut materials, road fill materials, road drainage and water control structures, road surfacing, stream crossing options, daylighting, and road closure techniques. The first field exercise is used to expose the students to a variety of road classes and road standards where students have the opportunity to see and discuss road entrances, road grades, cut and fill slopes, drainage structures, road surfacing, stream crossings, closure techniques, and contrast different road classes and the rationale of selected classes (Figure 1).



FIGURE 1 Students examine the template for a newly installed forest log truck access road.

Topic 2: Road Planning and Reconnaissance: Indoor and Outdoor Tools

Basic planning tools are the next major topic. The use of topographic maps, soil surveys, aerial photo–satellite imagery, and property plats–deeds for road planning are discussed. Students are trained to identify potential problems identified from topographic maps and soil surveys. Students are required to identify appropriate road grades for topography, needs, and environmental protection and to locate proposed grade-lines on topographic maps. Methods of installing grade-lines in the field are discussed in the classroom. During the field lab exercise students propose a grade-line on a topographic map and then reconnaissance the field site and subsequently use a clinometer (or hand level) to flag a proposed grade-line (Figure 2). The class subsequently discusses the resultant grade-lines and the positive and negative control points encountered.

Topic 3: Grade-Line Adjustments and Construction Techniques

Commonly encountered situations and techniques for grade-line adjustments are described, including intermediate obstacles, deep and narrow hollows, long narrow ridges, and switchback locations (Figure 3). Examples of switchback characterization and calculations (side-slopes, desired grades radius, length, cut depth and length, fill depth and length) required for uniform grade-line installations are provided. Students work through several grade-line adjustment exercises in the classroom. Subsequent field exercises require layout of a second grade-line and a switchback is designed and marked in the field. The advantages and disadvantages (including costs) of the location of the different switchbacks are discussed.



FIGURE 2 Students use simple hand levels or clinometers for grade-line installation.



FIGURE 3 Topographic maps and soil surveys are used for several exercises regarding road location.

Topic 4: Road Curves

The terminology and basic calculations of simple road curves are introduced. Standard terminology and calculations (curve radius, degree of curvature, change in direction, point of interception, point of curvature, point of tangency, external distance) are provided. Various methods of calculating an appropriate degree of curvature (desired radius, desired length, desired tangent distance, and desired external distance) are discussed. Advantages and disadvantages of three methods of curve installation (deflection angle method, tangent offset method, chord offset method) are discussed. The associated field exercise requires each student crew to install three curves using combinations of the three different methods for calculating an appropriate degree of curvature and three curve layout techniques (Figure 4). This exercise is often simplified for continuing education workshops and the simplest method of curve layout (chord offset) is discussed if time is limited.



FIGURE 4 The curves and switchbacks on this road were designed and laid out by students in the class.

Topic 5: Stream Crossings and Water Control Structures

Stream crossings are expensive and can be a primary entry point for sediment into streams. The advantages, disadvantages, BMPs, and costs of various types of crossings (designed stringer bridges, portable panel bridges, culverts, pole crossings, and fords) are described and discussed. Example calculations of appropriate culvert diameters are provided (Talbot's formula, TR-55). The second lecture is devoted to discussion of water control structure (ditches, culverts, broad based dips, rolling dips, water bars), surfacing techniques (stone, geofabrics, slash), and vegetation (seed, mulch, fertilizer, lime, hydroseed, etc). The linked field exercise involves examination of forest roads and stream crossings with discussion of the appropriateness of the crossings and potential best management practices that might be used to improve the roads and stream crossings (Figure 5).

Topic 6: Erosion Control and Best Management Practices

Sediment is the largest nonpoint source pollutant in the United States and the highest rates of erosion are from forest operations typically associated with forest roads. Lectures during this segment provide background on using the Universal Soil Loss Equations as modified for Forest Land (USLE-Forest) (31) and the Water Erosion Prediction Project–Road (WEPP-Road) (32) option for estimating erosion from forest roads. BMPs and advanced erosion control techniques also are presented. The field exercise consists of estimating erosion on roads having different levels of BMP compliance (Figure 6) with subsequent discussion regarding how BMP implementation levels and estimated erosion varied for the different crews.



FIGURE 5 Continuing education workshop participants discuss advantages and disadvantages of different stream crossings and BMPs for erosion control.



FIGURE 6 Students can examine and monitor a variety of erosion control techniques used on roads and skid trails such as this seed and mulch treatment used on a bladed skid trail.

Topic 7: Road Construction and Road Costs

The first lecture regarding road construction and costs provides an overview of standard equipment used to for pioneering, clearing and grubbing, road template establishment, installation of drainage structures, stream crossings, surfacing, and closing roads. Typical road costs for these construction activities are provided and road cost estimation techniques are discussed (machine rates, models, personal experience). During the second lecture, a simple estimation model (Virginia Tech Road Cost Method) (9) is introduced, used, and discussed during an in-class exercise. The field exercise during this week consists of estimating the costs required for construction of a new road similar to the standards of an existing road designed by a previous class (for which costs are known) and cost of installing needed BMPs to an existing road (Figure 7). Cost estimates developed by different crews are compared and reasons for differences are discussed.

Topic 8: Road Design

During the final week the class road design project is planned. Sometimes the task is hypothetical and sometimes an actual road will be constructed based on the class design. The task typically requires use of provided road objectives to design and layout the grade-line for the desired road standard, develop a plan and timeline for construction, and to estimate road costs.



FIGURE 7 Example of road designed by students for private landowners who kept cost records so that students can compare their cost estimates to the actual costs.

To date, the class has designed 10 roads that were actually constructed for a combined length of approximately 6 mi. The students prepare and present their plan and profile of their road, estimate total earthworks, estimate costs, provide drainage, surfacing, and construction plans, and estimate total costs for the entire road (Figure 8).

CONCLUSIONS

Forest Boundaries and Roads is a course that is similar to many other forestry courses with an experiential learning component, such as forest ecology, silviculture, forest fire, harvesting, and other courses related to the professional application of forestry. Overall, student perceptions regarding this class have been very positive. The class has been offered 29 times and has an overall student perception of teaching score that ranges from 5.6 to 6.0, with an average score of



FIGURE 8 Permanent forest access road that was planned and laid out by students taking the Forest Boundaries and Roads course.

5.7, where 6.0 is the maximum score. The vast majority of student comments supported the field exercises as teaching tools. Thus, as indicated by the literature, the experiential learning format does appear to provide a positive and applied learning format. Multiple former students are in leadership positions within companies and natural resource agencies where they use forest road skills frequently. However, class logistics are challenging, particularly for larger classes. For most field exercises, we must coordinate multiple vans and drivers in order to travel from campus to nearby forest roads. The vans cost approximately \$75 to \$100 per student per semester, so additional transportation fees are levied to cover the costs. In addition to the actual exercise, teaching assistants and the instructor acquire, drive, and return vans to a motorpool, which will usually require approximately 5 h total for an afternoon lab, which is a significant commitment. It is also expensive to maintain equipment for such courses. Students are required to supply a personal hand compass, clinometer, and loggers tape, but the department provides GPS units, total stations, and other related road layout equipment, as well as specialized software packages. Maximum enrollment is limited to 33 people, so 11 sets of equipment are required. Teaching workloads are considerably heavier for this type of class. For example, a traditional 3-credit class will have 150 min of student faculty contact per week, but this lecture and lab course has 340 min of student–faculty contact per week. The grading requirements also are heavier due to the increase time commitment for grading take home open book exams and weekly field experience memorandums and project reports. Additionally, exercises are often conducted in very inclement weather conditions, which typically hamper teaching exercises, yet

do prepare students to work in challenging conditions. As a young forestry student in the 1970s, the author was a beneficiary of experiential learning during formal forestry education and he remains a lifelong advocate for such experiences. However, such courses are more expensive in terms of time and finances. The long term sustainability of such courses will be determined by administrative support and supply of necessary resources combined with the continued willingness of future faculty to devote additional time required for experiential learning exercise in natural resources.

AUTHOR CONTRIBUTION STATEMENT

The author confirms sole responsibility for the following: study conception and design, data collection, analysis, and interpretation of results, and manuscript preparation.

ACKNOWLEDGMENTS

Logistical and financial support were provided by the Department of Forest Resources and Environmental Conservation, College of Natural Resources and Environment at Virginia Tech, Blacksburg. The author also wished to acknowledge the multiple graduates students who have served as teaching assistants in courses and the forest roads mentorship provided by the late Dr. Thomas A. Walbridge, Jr.

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Pavement Evaluation

PAVEMENT EVALUATION

Use of Unmanned Aerial Photogrammetry for Monitoring Low-Volume Roads After Hurricane Harvey

SURYA S. C. CONGRESS

ANAND J. PUPPALA

ARITRA BANERJEE

The University of Texas at Arlington

NAVID H. JAFARI

Louisiana State University

UJWALKUMAR D. PATIL

University of Guam

AASHTO's definition of low-volume roads (LVRs) includes paved roads that receive less than 1 million equivalent single-axle loads (ESALs) during their service period. In the United States, 60% of all roads are LVRs, as both urban and rural communities rely on them. In a recent event, a Category 4 hurricane made landfall in the Houston, Texas, metropolitan area, inflicting serious damage on civil and transportation infrastructures. Low-volume arterial roads were affected by the flooding and by the high volume of trucks that collected the enormous amount of debris left by the massive destruction of homes and their contents. The University of Texas at Arlington (UTA) conducted research to investigate safe and efficient methods for monitoring LVRs with unmanned aerial vehicles that are equipped with a camera for close-range photogrammetry (UAV-CRP). A visible range camera was mounted on an unmanned aerial platform to collect images that were processed into 3D models to provide metric and qualitative information. By analyzing these models, different attributes of the LVRs were obtained, including surficial cracks, deformations, elevation profiles, etc. Information pertaining to the volume and location of debris piled up on areas adjacent to roads was also provided. UAV-CRP technology has proven to be cost-effective and efficient for inspections. It is also an important reconnaissance tool post-natural disasters, when agencies are insufficiently staffed to conduct LVR infrastructure monitoring tasks while also performing rescue and recovery tasks.

INTRODUCTION

The American Association of State Highway Transportation Officials (AASHTO) defines LVRs as paved and unpaved roads that receive less than 1 million ESALs during their service period. Unpaved LVRs are often built out of necessity, rather than as planned engineering structures (1–3), and combined with paved roads, they represent 60% of total roadway miles in the United States (4). For those living in rural communities, they provide access to essential health, education, and civic services, and outdoor recreational facilities. Even though they carry only 20% of the traffic, they are considered the primary link for public mobility and for transporting materials to markets. This is key for improving the gross domestic product (GDP), which is an indicator of

the economy of any nation (*I*). Recently, there has been an increase in the frequency of occurrence of hurricanes that cause enormous destruction to properties and lives. Hurricane Harvey, the 500-year flood that struck Houston, is a prime example of such a disaster.

Inundated layers of pavement affect its service life, making it vitally important to monitor them immediately after a hurricane event. However, agencies frequently lack the personnel to collect information pertaining to the conditions of the roads while coordinating rescue and recovery operations. Aerial technology, with its quick turnaround time for data collection, offers a perfect solution for collecting data on pavement conditions and the volumes of debris generated by hurricanes. This research study identifies the need to monitor LVRs post-hurricane, using innovative UAV-CRP technology, and provides two case studies with data that was collected immediately post-Hurricane Harvey. Comprehensive data analyses showed how this technology can provide valuable information that will aid cities and regional transportation agencies in their infrastructure restoration and resiliency efforts.

Hurricane Harvey

Storms can easily form over warm bodies of water that are located near the equator, as the surrounding air swirls in to compensate for the lower air pressure created by the rise of warm air above the water. The wind flow creates an eye, and gradually grows into a storm. When the speed of the outer rotating winds of a storm reach a minimum of 39 mph, it is considered a tropical storm. When the wind speed exceeds 74 mph, it is considered a tropical cyclone or hurricane (5). Tropical cyclones originate frequently from the Atlantic basin, and, less frequently, from the central part of the North Pacific Ocean (6). Hurricanes are classified by their maximum sustained wind speed, defined as the speed that occurs for a minimum of 1 min at a standard meteorological altitude of 10 m (33 ft), in an unobstructed area above the ground. Hurricanes are classified into five types, based upon the maximum sustained wind speeds shown in Table 1 (7).

Hurricanes feed on warm moist air, as shown in Figure 1, and die down as they approach land, due to the loss of energy that was provided by the warm ocean water. Unfortunately, the intensity of the hurricane allows it to travel a great distance over land, causing great damage before it ceases. Satellites monitor the occurrence of these natural disasters to warn the public potentially affected by them and reduce the enormity of loss. Two Geostationary Operational Environmental Satellites (GOES), jointly managed by the National Aeronautics and Space Administration (NASA) and the National Oceanic and Atmospheric Administration (NOAA), assist meteorologists in observing and predicting the weather (5).

TABLE 1 Saffir-Simpson Hurricane Wind Scale (5)

Category	Wind Speed (mph)	Damage Level	Storm Surge (feet)
1	74-95	Minimal	4-5
2	96-110	Moderate	6-8
3	111-129	Extensive	9-12
4	130-156	Extreme	13-18
5	157 or higher	Catastrophic	19+

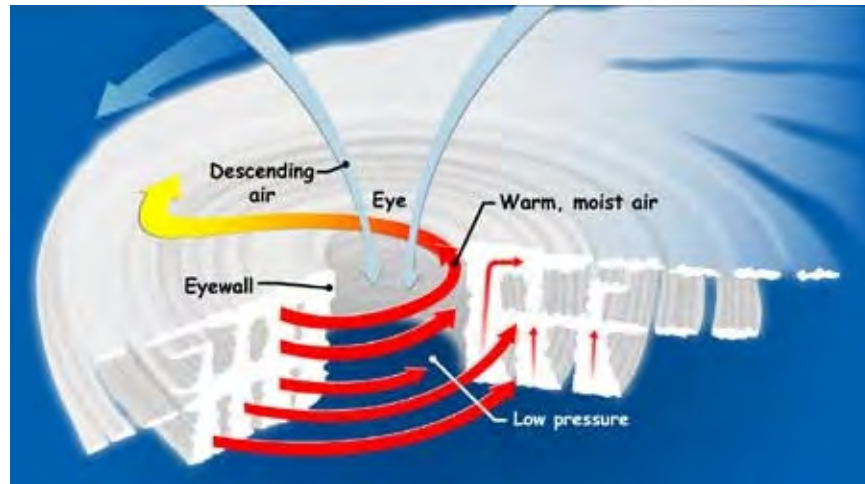


FIGURE 1 Mechanism of hurricane from the slice of eye (5).

Hurricane Harvey, a Category 4 hurricane with a diameter of 280 mi and wind speed of approximately 130 mph, made landfall in the Houston metropolitan area on August 25, 2017, and inflicted serious damage. Although the winds died down in few days, the rainfall-triggered flooding caused catastrophic damage due to inundation (8). Some areas had more than 50 in. of rain, according the National Weather Service (9), and two reservoirs exceeded their limits and breached, causing most parts of Houston to be inundated, and creating huge amounts of debris. Hurricane Harvey was recorded as being the second most-costly hurricane to make landfall in the United States since 1900.

Low-Volume Roads

The durability of LVRs depends upon many factors, including the strength and stiffness of underlying layers, traffic, environment, frequency of condition monitoring, and others (9–13). Many studies have been conducted on treating low-volume road subgrades and embankments composed of problematic soils (2, 10, 11, 14, 15). The amount of funding allocated for construction and maintenance often depends upon the number of road users, hence the funding available for LVRs is limited, even though the vehicular loads are higher than those of high-volume roads and they require frequent monitoring and rehabilitation. The need for pavement condition data increases exponentially immediately after a disaster. The quality of traditional visible inspections of LVRs varies with the experience and expertise of the inspector, thereby yielding subjective evaluations that cannot be used for making metric assessments. Inspection methods using laser scanners obtain high-quality information about pavement conditions, but are not feasible due to their high cost. Therefore, a need exists for a technology that can provide quality data at a reasonable cost, within a short timeframe. The UAV-CRP technology offers one such approach that was evaluated in the present research. More details of this technology are provided in the following.

UAV-CRP Technology

Photogrammetry is a remote data collection technique that can record or capture information, using imaging sensors to make measurements without coming in direct contact with the inspected element (16). It is also referred to as an art, science, and technology designed to obtain reliable information about physical objects and their surrounding environment through the process of recording, measuring and interpreting patterns (17–19). Under most conditions, and particularly in large areas, photogrammetric techniques have proven to be inexpensive; they have even proven useful in land surveying (18–23). Scans or images taken from a distance of less than 1,000 ft (~300 m) between the sensor platform and the inspected object are usually classified as close-range photogrammetry (CRP). Employing UAVs for photogrammetry comes under the category of close-range photogrammetry (CRP) (19, 23–25), referred in this study as UAV-CRP.

The two types of UAVs (or unmanned aircraft systems) units are rotary wing and fixed wing. A fixed-wing UAV has a single rigid wing across its body that allows it to fly at high speeds and for long flight distances, similar to manned airplanes (26). A rotary-wing UAV uses lift from the continuous rotation of its blades, and has the ability to take off and land vertically, similar to manned helicopters. The main advantages of these systems are that they can access remotely located areas and confined spaces, and can hover at a fixed altitude, allowing sensors (such as digital cameras) to collect precise data from hard-to-reach areas. Due to their versatile nature, rotary UAVs have become a popular means of remotely gathering information and assessing infrastructure damage in the past decade. Photogrammetry software, using UAV-collected data, can provide a 3D dense point cloud model, an orthomosaic, a digital elevation model (DEM), and a good-quality digital terrain model (DTM) in a short period of time (27, 28). The main advantages of UAVs over traditional surveying techniques are their ability to capture detailed images of the study area inexpensively and with rapid deployment.

Infrastructure Health Monitoring

Federal government initiatives and research conducted by NASA in building military drones paved the way for using UAVs for various civil engineering applications. Modern image-capturing equipment provided the impetus for conducting real-time mapping, surveying, and monitoring of assets. The ability to remotely monitor and detect physical features in infrastructures is of great value to civil engineers. The UAV can identify distress in pavements, roads, rail bridges, and movement of slopes and embankments, etc. Many researchers have identified the potential of using UAV-CRP technology.

Frew et al. (2004) used a modified Sig Rascal radio-controlled airplane to demonstrate vision-based autonomous following of roads. Vision systems are small and lightweight, due to their passive type of data collection and processing. They process the natural scenes in the field and measure the relative distance and orientation between an aircraft and the road. This approach was planned to help in collecting road data in areas where GPS is unavailable. They encountered disparities in vision-based and GPS measurements of the UAV, due to their assumption of zero roll (29).

Rathinam et al. conducted fixed-wing monitoring of linear structures, such as roads, pipelines, bridges, and canals. Linear structures were detected by visual recognition techniques controlled by a closed loop algorithm (30). Irizarry et al. used a small-scale drone equipped with a video camera that used image capturing, as well as real-time videos, at construction sites (31).

They concluded that a high-resolution camera, vocal interaction, and autonomous navigation were some of the ideal features of a drone system that allow safety inspections at construction sites (31). Congress et al. (32) conducted a comprehensive error analysis of a UAV and camera as a total system. They obtained infrastructure models with accuracy values that are necessary to make engineering judgements. A pavement case study was provided to inspect the presence of heaving on the pavement (28, 32). Puppala et al. (33) emphasized the role of aerial data collection by providing 3D visualization models of infrastructures. They concluded that navigable 3D models assist in a better understanding of infrastructure health (33). Recent studies also utilized the advanced aerial platforms in validating the effectiveness of the ground stabilization techniques using the aerial data collected in the field (34, 35).

The Beaumont-area locations of the two case studies had received significant amount of rainfall, as shown in Figure 2. The subsequent sections cover the works performed by the UTA research team as a part of the National Science Foundation's Rapid Response Research study, along with data analysis and observations of LVRs' performance after a hurricane event.

POST-HURRICANE HARVEY: LOW-VOLUME ROAD DATA COLLECTION

Case Study I

The first case study data was collected on San Anselmo Street in Beaumont, Texas. There was no traffic during the data collection; however, due to the location of debris in close proximity to the T-intersection with Willis Lane, an additional visual observer was employed for spotting the traffic around the site. The UAV was flown manually due to excessive obstructions posed by the trees. The flight path was planned in such a way that the pavement and three debris stockpiles, as shown in Figure 3, were covered in single flight mission to assess the infrastructure condition

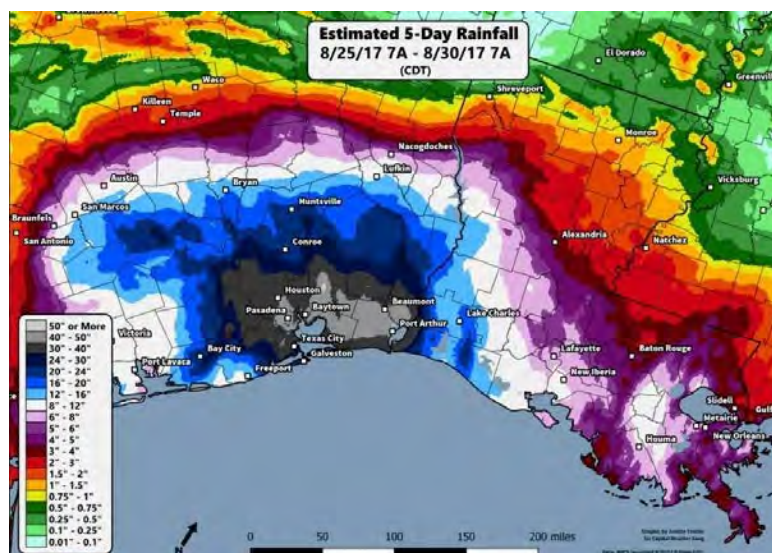


FIGURE 2 Estimated rainfall in Beaumont area during Hurricane Harvey in 2017 (36).

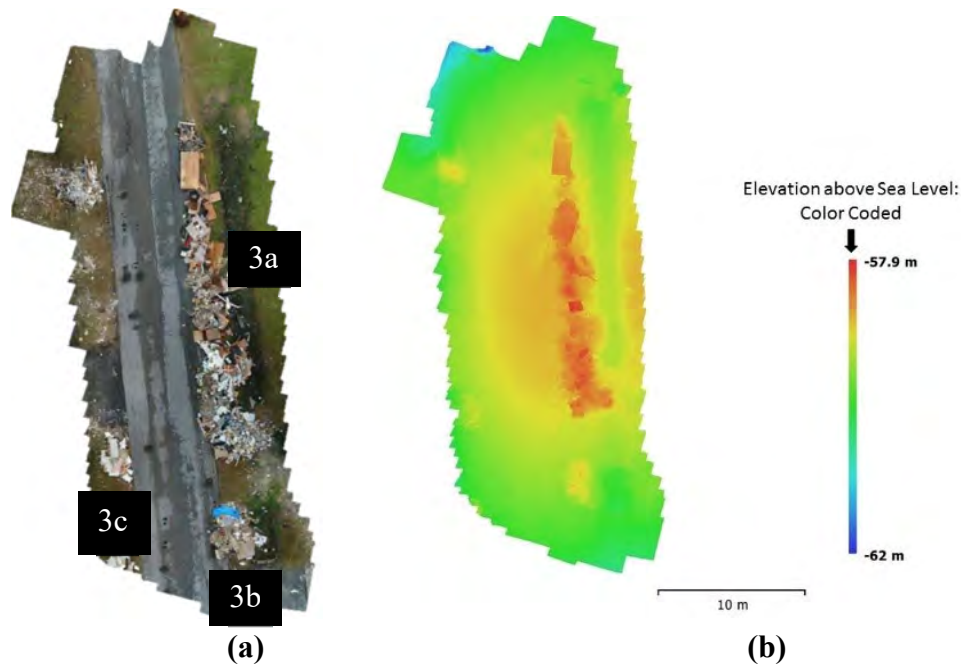


FIGURE 3 Case Study I mapping data (a) orthomosaic and (b) digital elevation model.

after the natural disaster. All of the captured images were geotagged and processed to generate a 3D dense point cloud model, an orthomosaic, and a DEM, as shown in Figure 3. The color-coded bar in the DEM of the collected data shows the highest elevation point of the debris pile in red and the lowest elevation point in dark blue. This gives an estimate of the relative elevations of the surrounding areas, i.e., the debris was stockpiled over an area that sloped downwards, away from the adjacent pavement.

Pavement Deformation

Rutting, or excessive permanent deformation, is a common type of flexible pavement distress that is caused by the deformation of deteriorated material properties in the asphalt surface or underlying base and subgrade layers. The deformation can be observed immediately after a rainy season, when floods inundate the whole pavement right-of-way. The stagnant water permeates into the underlying pavement layers and reduces the shear strength, causing the deformation of the underlying layers.

Excessive rut depths can also contribute to water stagnation that impedes the friction necessary for skid resistance, and they need to be measured to ascertain whether the pavement is suitable for vehicular traffic. Traditional methods use a straightedge, placed over two pavement contact areas across the rut, and a gauge to measure the rut depth (the distance between the bottom surface of the straightedge and the deformed pavement surface) (37). This is a tedious procedure, as it involves changing the position of the instrument frequently, and requires additional personnel to perform the traffic control operations and follow other safety protocols. UAV-CRP technology offers a quick way of assessing the rut depth with no or minimal traffic restrictions.

The research team used the UAV-CRP technology to map the pavement infrastructure condition in Beaumont immediately after the occurrence of Hurricane Harvey in 2017. The hurricane-triggered flooding inundated Houston and many other cities along the coastal region. Figure 4 depicts one of the inspected infrastructure sites that had the pavement sides piled up with debris generated from the destruction of residences and their contents.

The 3D view presents the color-coded elevation of pavements and surrounding areas covered under the outer cyan-colored rectangle in the top view, as indicated by the curved yellow arrow in Figure 4. The crest of the debris pike is red in 3D view, and the pavement is indicated by the green and yellow areas on the left side of the pile of debris. The lowest point in the collected data, the sloping ground on the other side of the pavement opposite the debris pile, is indicated in dark blue in the 3D view, as shown in Figure 4. Color-coded elevations can also be seen in the profile view of the pavement provided in Figure 4.

While collecting data, the research team physically identified permanent deformation conditions at the pavement location marked within the red rectangle in the top view and red ellipse in the profile view of Figure 4. After the images and building models were processed, they were analyzed to determine the extent of distress. A sectional view of the pavement in the small white rectangle shown in the top view was provided in the profile view of Figure 4. All of the points in the generated point cloud possessed their respective XYZ coordinates, which helped in assessing the extent of distress. The depth of the rutted portion from the dense point cloud was estimated as 12.20 cm (4.80 in.), which matched the depth measurements taken from field surveys. The UAV-CRP technology provided a quick and accurate way to estimate the rutting depth, in addition to providing other pavement-related information.

A detailed elevation profile of the entire pavement surface provided a comprehensive understanding of the conditions, as shown in Figure 5. The elevation profile of the centerlines of lanes was obtained by placing points at 1-cm intervals, as shown in Figure 5a, to understand the extent of distress on the 3-m pavement stretch that is depicted by red rectangles in Figure 4. The profile of the left lane is shown in red and the right lane in green. The plot between the actual elevation of points on the pavement and their corresponding longitudinal distances from the initial point along the centerlines of two lanes of the pavement are shown in Figure 5b. As can be



FIGURE 4 UAV-CRP technology data identifying rutting of pavement section immediately after Hurricane Harvey.

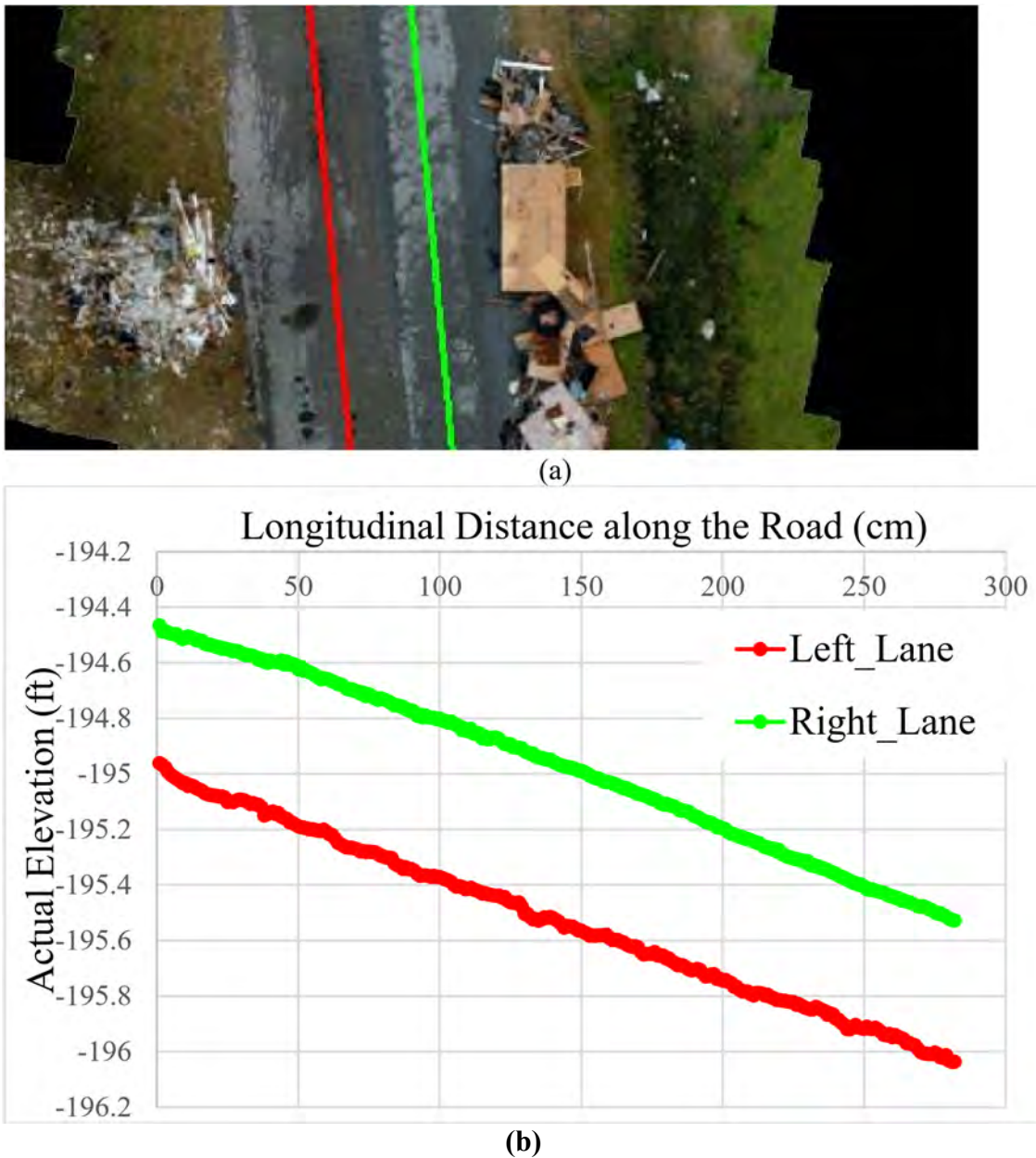


FIGURE 5 Pavement deformation identification (a) elevation profiles of left and right lanes at 1-cm interval and (b) plot between actual elevation and corresponding longitudinal distance along the centerlines of two lanes of the pavement.

observed in Figure 5b, the right lane has a smoother elevation profile than the left lane. This indicates more distress over the left lane, which can also be observed in the 3D view of Figure 4. A difference of approximately 12.7 cm (5 in.) between the elevations of the right and left lane points, at the same longitudinal distance from the initial point shown in Figure 5b, confirms that a permanent deformation occurred on the 3 m stretch of the left lane, making it unsafe for road users until it has been rehabilitated.

Aerial technology assists in estimating the condition of an infrastructure and the volume of debris generated by hurricane-triggered rainfall events. The two stockpiles on the right side of the pavement in Figure 3a are marked 3a and 3b, and the debris pile on the left side of the pavement is marked 3c. Using the 3D dense point cloud model and orthomosaic, the volume of each of the three stockpiles (3a, 3b, and 3c) was obtained as 19 m³ (676 ft³), 3 m³ (104 ft³), and 0.8 m³ (28 ft³), respectively. Cities are responsible for removing the debris and properly planning the number of truck loads is vital so that it can be removed in a timely manner, without exerting undue stress on the infrastructure with the use of heavy trucks.

Case Study II

The data for the second case study was collected at 11080 Sherwood Drive in Beaumont, which is located parallel to a railway line without any obstructions by trees. The debris was spread around the pavement and the adjacent areas, in an area with moderate traffic. Therefore, visual observers were present as an added precaution while performing the UAV surveys and data collection operations.

All of the images were geotagged and processed, according to the procedures explained above, to generate the 3D dense point cloud model, orthomosaic, and DEM shown in Figure 6. The color-coded bar in the DEM of the collected data represents the highest elevation point on the debris pile, shown in red, and the lowest elevation point in the collected data is shown in dark blue color. This gives an idea of the relative elevations of the surrounding areas. The debris was piled over an undulated pavement, caused by excessive cracking, which is shown in light yellow in the DEM.

Pavement Cracking

The aerial data of the pavement distress, shown in Figure 6, was collected immediately after the hurricane. Prior data of the pavement condition was not available to the research team. Hence,

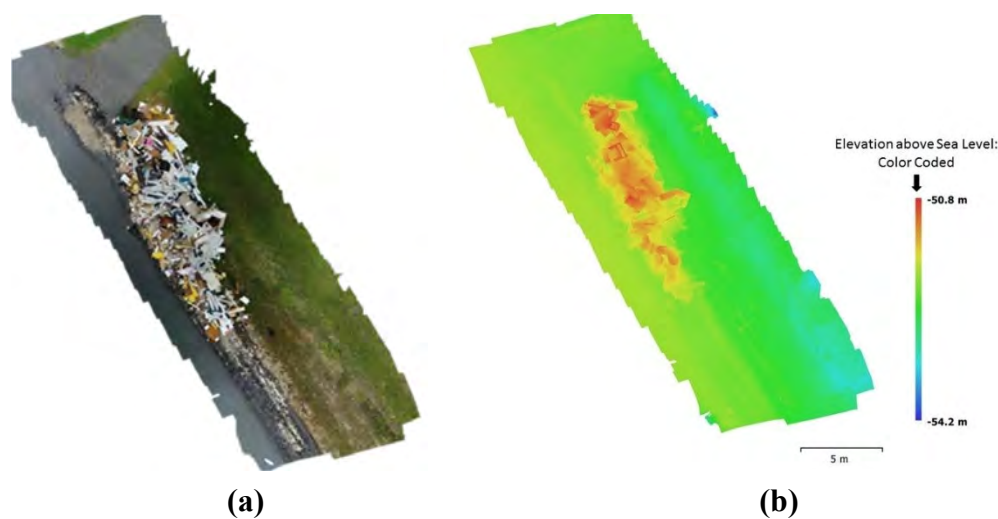


FIGURE 6 Case Study II mapping data (a) orthomosaic and (b) DEM.

Google maps was accessed to obtain images of the exact same location before the hurricane. Unfortunately, the most recent image was captured in January 2013; nevertheless, it was studied to understand the reasons for the excessive cracking failure of the pavement that is shown in Figure 6. Few cracks were identified, but a barely noticeable sealed crack and developing crack were identified from the google images and are shown in Figure 7.

The aerial data was analyzed to estimate the present condition of distress, as shown in Figure 8. The maximum depth of distress, calculated as 6 cm (2.4 in.) along the 24 m (80 ft) length, was identified quickly from the aerial data shown in Figures 8a and 8b.



FIGURE 7 Google map street images of Sherwood Drive, looking towards Tram Road in Beaumont, captured in January 2013: (a) sealed cracks and (b) developing cracks. (Map data © 2018 Google.)

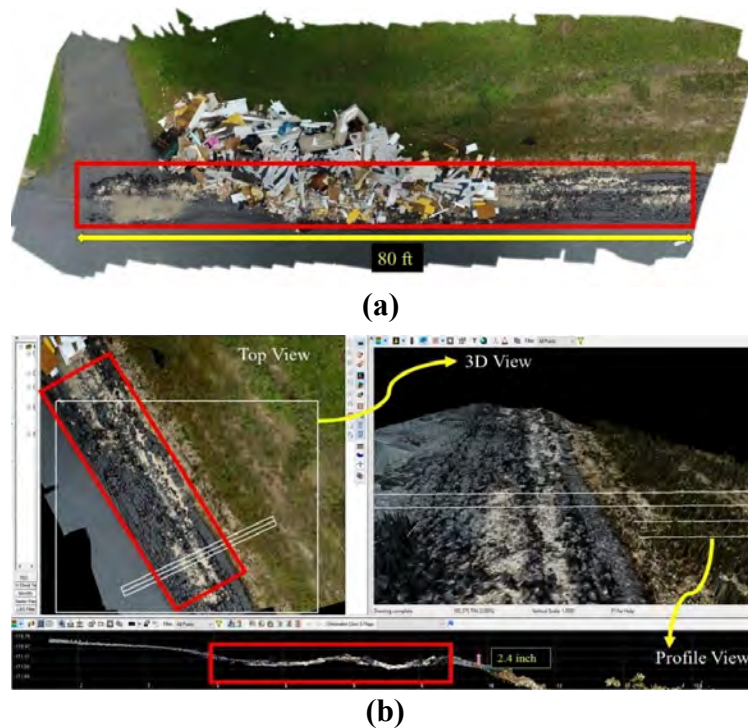


FIGURE 8 Pavement condition data (a) distress highlighted in the orthomosaic of the debris site and (b) different views of the UAV-CRP data displaying the extent of distress.

The authors opine that even though new cracks had developed (Figure 7*b*), the site had experienced earlier cracking problems, as evidenced by the Google images captured in 2013 (Figure 7*a*). The seepage of water through the new cracks, the traffic volume, and other factors affecting pavement stability might have triggered the failure, but the proactive use of an affordable monitoring tool could have mitigated the damage by assisting with planning preventative maintenance. This highlights the importance of maintaining pavement before it deteriorates to the point that the entire pavement structure needs rehabilitating. UAV-CRP technology offers high-quality data images of pavement and would help in monitoring the distress propagation.

In addition to the pavement condition, a stockpile of sheetrock waste, sodden furniture, and carpets was also inspected at this location. Using the 3D dense point cloud model and an orthomosaic, the volume of the stockpile was obtained as 25.7 m³ (907 ft³).

Aerial surveys provide data that can be utilized for multipurpose applications, as they do not require separate procedures for estimating the condition of pavement infrastructure and debris volumes, as traditional methods do. Photogrammetry data collected from UAVs can be analyzed for multiple attributes, as shown in the above case studies. Overall, the UAV-CRP tool can be utilized for health monitoring of low-volume-related pavement infrastructure.

SUMMARY AND CONCLUSIONS

The recent increased frequency of higher-intensity natural disasters warrants the use of new technology to inspect pavement conditions rapidly, efficiently, and safely, immediately after a hurricane. UAV-CRP technology can be used to assess the extent of various pavement distresses, such as permanent deformation and cracking.

Photogrammetry data collected from UAVs can be analyzed for multiple pavement distresses. UAV-CRP technology offers the capability of collecting more data with less boots on the ground, which is important immediately after a disaster when a limited number of personnel have a wide variety of pressing tasks to perform. Proactive monitoring of pavement facilitates prioritization of maintenance activities and allocation of funds for rehabilitation tasks.

During these studies, it became apparent that aerial technology is multipurpose. It not only helps in estimating the condition of the pavements, but also in assessing the volume of debris generated by hurricane-triggered rainfall.

ACKNOWLEDGMENTS

The authors sincerely thank the National Science Foundation and Program Director, David Corman, for funding the award 1760715. They would also like to acknowledge the UTA NSF IUCRC CICI Site and its research team; Texas Department of Transportation; the Center for Transportation, Equity, Decisions and Dollars (CTEDD); and the Transportation Consortium of South-Central States (Tran-SET) for their support.

AUTHOR CONTRIBUTION STATEMENT

The authors confirm contributions to the paper as follows: study conception and design: Surya S. C. Congress and Anand J. Puppala; data collection: Surya S. C. Congress and Anand J. Puppala; analysis and interpretation of results: Surya S. C. Congress and Anand J. Puppala; draft manuscript preparation: Surya S. C. Congress, Anand J. Puppala, Navid Jafari, Aritra Banerjee, and Ujwalkumar Patil. All authors reviewed the results and approved the final version of the manuscript.

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PAVEMENT EVALUATION

Integrating Unmanned Aerial System-Based Road Corridor Canopy Mapping with Real-Time Dust Monitoring

ERIC H. CHASE
BRENNAN HOLDERMAN
MARC MCDILL
DOUG MILLER
MOHAMED ABUZAIID
Penn State University

INTRODUCTION

Studies on vehicle-generated dust are well documented in the arid regions of the United States (1–3), but data are lacking in eastern temperate regions where unpaved roads and dense canopy cover are common in rural areas. Due to this data gap, the Center for Dirt and Gravel Road Studies at Penn State University (the Center) began an investigation of dust production from unpaved roads in central Pennsylvania in 2016. The focus of the initial study was the comparison of dust emission levels from gravel road surfaces composed of Driving Surface Aggregate (DSA), and commonly used Pennsylvania Department of Transportation specification 2A aggregate. DSA was developed by the Center specifically for use as a wearing course on unpaved roads and has been used extensively in Pennsylvania since 2000 (4, 5).

Previous dust-monitoring studies have found a number of factors that influence dust generation such as vehicle speed, wind speed, vehicle make, days since rain, present of silt and material type (1–3, 6, 7). Additionally, the study done by the Center in 2016 showed that canopy cover above the road corridor is another significant variable driving dust production (5). In that study, higher dust values were correlated with higher road surface temperatures and the response was attributed to canopy cover and the amount of shading on the road surface. When the data was stratified by visual canopy observations, the sections with full canopy cover had road temperatures slightly cooler than ambient air temperature, while sections in full sun were generally 10°C to 15°C warmer. However, a more detailed analysis of the relationship between canopy cover and dust production could not be conducted due to lack of high-quality canopy cover data available.

Realizing the significant role that canopy cover played in dust generation in forested temperate environments, the Center partnered with the Mobile GeoSpatial Systems Group (MGSG) in the Penn State Department of Ecosystem Science and Management to help provide high-quality canopy mapping above road corridors. In 2018, the MGSG developed a low-cost “mapping grade” (better than decimeter accuracy in ideal conditions) Global Navigation Satellite System (GNSS) that was combined with a small unmanned aerial system (sUAS) to provide an inexpensive solution for high-resolution and spatially accurate aerial imagery in forested regions. Initial testing done in 2018 showed success with combining the dust-monitoring data with the georeferenced aerial imagery.

METHODS

Unpaved road dust production was measured along Red Rose Road in the Penn State Experimental Forest in Huntington County, Pennsylvania, using a vehicle-mounted particulate monitoring system with integrated GPS tracking. The primary dust-monitoring instruments consist of two TSI DustTrak 8530 aerosol monitors. During monitoring runs both aerosol monitors were mounted on the rear of the vehicle with the inlets approximately 1 m above the road surface to monitor PM_{2.5} and PM₁₀ simultaneously. The integrated dust-monitoring system developed by the Center captures dust generation from both instruments as well as GPS location every 1 s, and the data are then recorded on a laptop. In fall 2018, an infrared road surface temperature instrument was integrated into the system to capture road surface temperature every 1 s.

Prior to aerially mapping the Red Rose Road corridor, the MGS installed 15 ground control points (GCPs) with 8-in. aerial markers at clearings along 2 km of the road. GCPs were occupied for 20 to 25 min each with low-cost Emlid GNSS L1 receivers mounted to a Leica survey tripod and also by a survey-grade TopCon Hiper Lite L1/L2 GNSS receivers as a control. Data were post-processed in a radial network against a continuously operating reference station located 15 km north. High-quality geotagged aerial imagery was then collected at 250 to 300 ft above ground level with a DJI Phantom 4 advanced sUAS with a built-in 20-MP camera. The flight imagery was then processed to produce an orthomosaic of the Red Rose Road corridor from 993 photos and the raw location data of each GCP was extracted for comparison to the stationary points collected by the GNSS receivers. The dust-monitoring data and the aerial imagery were then combined in ArcMap for analysis of canopy cover and to explore its relationship to dust production.

FINDINGS

Due to a record-breaking rainfall during the summer of 2018 (>55 cm in 3 months), dust measurements were limited to a short window in late July. The weather limited not only the number of dust-monitoring runs but also the number of sUAS flights of the road corridor. The wet weather also precluded the use of the road surface temperature instrumentation, which was not fully integrated into the data collection system until late summer.

An analysis of the performance of the aerial mapping system showed that horizontal positions of the GCPs extracted from the raw orthomosaic collected from the sUAS are within 2 m of the post-processed Emlid GNSS L1 receivers GCPs, and both datasets are typically within 2 m of survey-grade data that was collected as a control. The vertical positions have >20-m offset between the Emlid data and the orthomosaic; the raw orthomosaic shows little topographic change while the Emlid shows nearly 50 m of change, in good agreement with the survey-grade data. Nine of the Emlid points are within 1-m elevation of the survey data; three are within 10 cm. Using the three GCPs with highest-quality satellite observations to georeference the orthomosaic dramatically improves the topographic representation over the entirety of the road segment. Photo resolution was typically in the 2- to 3-cm-per-pixel range (Figure 1).



FIGURE 1 Photo showing edge of road vegetation shading the road surface. Note the high-quality image resolution of 2 to 3 cm per pixel.

Since both the dust-monitoring data and the aerial imagery are spatially located they were overlain and mapped together in ArcMAP (Figure 2). Analysis of the full data set showed dust reductions of 50% to 75% when the monitoring vehicle traveled through road sections with full canopy cover. These results are similar to the reductions found in 2016 when using road surface temperature as a surrogate for canopy cover.

As shown in Figure 2 there is a lag time before dust production decreases when entering shaded areas. During monitoring the typical lag time before dust reductions were observed was 2 to 3 s after entering shaded sections and the same lag was observed for increases when leaving shaded sections. This is related to the speed of the vehicle (~40 km/h) and the fact that the instruments are a flow-through system that needed to pull a sample of air through the instrument before recording the dust value. Additionally the data are output as 1 s averages which can contribute to the observed lag.



FIGURE 2 Segment of dust-monitoring data points overlain on georeferenced aerial imagery. The monitoring vehicle was traveling from left to right. Note lag in the dust production decrease as vehicle enters road section with full canopy. Dust values in mg/m³.

CONCLUSIONS

This research is the first step in assessing the performance of a low-cost sUAS for mapping canopy above forest roads and preliminary results indicate that a combination of low-cost L1 GNSS and geotagged aerial imagery can yield high-resolution and spatially accurate orthomosaics. Combining the imagery with real-time dust monitoring shows promise in assessing the significance of road surface canopy shading on dust production. To date the analysis has been limited to unpaved road segments with full sun or full canopy cover.

Currently an algorithm is being developed to process percent canopy cover from aerial imagery on road segments where the canopy above the road is a mix of sun and shade. A full assessment of percent canopy cover on road segments with mixed canopy cover will be needed to determine if there is a canopy threshold value at which dust significantly decreases. This information would be valuable to road and natural resource managers to help guide road corridor canopy management.

The work done in 2018 has set the stage for more significant exploration of the effect of canopy on dust production in summer 2019. Both the dust-monitoring system and the sUAS canopy mapping system are being refined based on the knowledge gained to date. Future work will include continued dust monitoring with the addition of an integrated road surface temperature sensor and additional aerial imagery collection with an expanded Emlid GNSS receiver network to improve the spatial accuracy of the processed images. Additionally, a sUAS lidar mapping system is currently being developed and tested to provide canopy structure and additional information to assess canopy cover and shading.

ACKNOWLEDGMENTS

The authors would like to thank the Pennsylvania Department of Conservation and Natural Resources Bureau of Forestry and the Pennsylvania State Conservation Commission for supporting the project and providing equipment support. Additionally we want to thank the Penn State University Experimental Forest and Rothrock State Forest for allowing us use of their roads for this study.

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PAVEMENT EVALUATION

Use of Cell Phone Apps to Evaluate Low-Volume Roads in Illinois

MOHAMMAD HOSSAIN

NIKITA KAPIL

COLE GRIMM

Bradley University

EROL TUTUMLUER

University of Illinois Urbana–Champaign

Cell phone apps can be used to measure low-volume road roughness as a cheaper alternative to using a high-speed profilometer and the app can be made user-friendly. This study evaluated two Android operating system-based apps, Roadroid and RoadBump, both commercially available, to measure pavement roughness, specifically, International Roughness Index (IRI) of low-volume roads. The measured IRI values were validated with a high-speed laser profiler operated on both smooth and rough local roads. Results indicate that Roadroid provides better IRI data when compared to the RoadBump app that is in reasonably good agreement with the high-speed profilometer data. A consistent IRI data on low-volume roads could be achieved if the vehicle was driven within 30 to 50 km/h speeds on both smooth and rough pavements. Dashboard mount with Roadroid app offered the most reliable IRI values. A definite conclusion cannot be made regarding the most suitable vehicle among a sedan car, a van, and a truck on which to mount the cell phone, although the sedan car option often provided somewhat better results in measuring IRI with Roadroid app.

INTRODUCTION

Low-volume roads are defined as two-lane urban or rural roads, paved or unpaved roads, and the vehicle per day ranges from 400 to 2,000 (1). Low-volume roads play crucial roles by providing access from regional highways or collector roads to cities, villages, townships, and farms, and other necessary places like health, education, and outdoor recreational facilities (2, 3). In this paper, low-volume and rural roads are used interchangeably.

Illinois Department of Transportation (DOT) oversees 146,890 mi of total road network, of which the paved roads are 61,669 mi, and unpaved roads are 85,221 mi; accordingly, unpaved roads comprise of 58% of total road mileage in Illinois as of 2015 (4). The paved roads are defined as a block, brick, portland cement concrete pavement, and asphalt concrete (AC) surface of 1.0 in. or more where the combined surfaced and base thickness is 7.0 in. or more. On the other hand, the unpaved roads are defined as AC surface where the combined surface and base thickness is less than 7.0 in., seal coat, and earth–gravel roads. Counties oversee 9,650 mi paved and 6,838 mi unpaved roads, and townships and municipalities manage 35,886 mi of paved and 78,252 mi of unpaved roads. Illinois has the highest number of local governments consisting of 102 counties, 1,457 townships, and 1,297 municipalities (5).

Illinois grows most corn and soybean in the United States, and for this reason, rural roads are often used by farm machinery, tractors, and haul trucks, in addition to common vehicle categories, to carry local produce. Like other paved roads, low-volume roads are exposed to damages and distresses, but the damages are often severe since the low-volume roads are used by slow-moving heavy vehicles such as a tractors. The construction materials of low-volume roads usually consist of native soil topped with full-depth AC, or single or multiple seal-coated layers on gravel or paved surface (6). Accordingly, low-volume roads have fewer structural layers compared to traditional AC and cement concrete pavements, and such few layers are not adequate to transfer vehicle loads to the subgrade, which results in higher and accelerated pavement damage. Also, often local materials, e.g., inferior quality aggregates such as gravel as opposed to crushed stone, are used in low-volume roads. As a result, many local agency roads show excessive damage in early spring due to the weakening of subgrade conditions caused by the melting of snow and thawing of soil, and the roads become inaccessible to vehicles. Figure 1a shows an agricultural vehicle with wide tire and Figure 1b shows the exposed edge cracking in the seal-coated road due to heavy tire pressure. Figure 1b also indicates excessive bleeding in the roads caused by heavy truck traffic.

In Illinois, rural roads are maintained by townships and highway commissioners are often in charge. Each county has often several townships and a county engineer, who provides technical support and coordinates rural roads maintenance plans and finances with Illinois DOT. Often a highway commissioner is not an engineer but has experience on rural road maintenance and management. Highway commissioners use their expertise to access road damage or distress by evaluating roads while driving a truck and looking around through the windshield. This method is known as a windshield survey, which is a subjective measure.

The degree of road damage can be assessed by measuring specific information related distresses, such as cracks, potholes, patches, and ruts, with tapes and straightedge, and by recording them according to ASTM or other standards. Several graphs–charts and equations are



(a)



(b)

FIGURE 1 Typical heavy-axle-weight agricultural vehicle using seal-coated roads and associated pavement distresses caused by heavy vehicles: (a) an agricultural vehicle with wide tires that are covering the full width of seal-coated roads and (b) edge cracks in seal-coated road due to the frequent passage of oversized agricultural vehicles and excessive bleeding in the roads.

often used to normalize pavement distresses and determine the condition of the road by expressing the result through quantifiable condition indices such as the Pavement Conditioning Index (PCI) (7). Several other indices are also available to express road condition through Pavement Serviceability Rating, Ride Number, Pavement Surface Evaluation Rating, Unsurfaced Road Conditioning Index and many others (8–11). The physical distress measuring methods are time-consuming as well as labor-intensive. Also, the lane closure is needed to collect the data and safety of surveyors should be assured during the data collection time. Surveyors need to attend safety training for flaggers before conducting the distress survey.

A standard road roughness measuring scale known as the IRI is commonly used by transportation professionals. According to the World Bank, “The IRI is based on simulation of the roughness response of a car traveling at 80 km/h. It is the reference average rectified slope, which expresses a ratio of the accumulated suspension motion of a vehicle, divided by the distance traveled during the test” (12). IRI represents the longitudinal profile of a road. A higher IRI value means rougher pavement and a lower IRI value indicates smoother pavements. A pavement becomes rough when it shows distresses such as cracks and potholes; on the other hand, a smoother pavement represents newly constructed or rehabilitated pavement. A variety of equipment, such as profilograph, response-type devices, walking profilers, and inertial-laser profilers, are available to measure IRI. These road roughness assessment devices are expensive, require a professional operator, and the operation is complicated, time-consuming, and delicate. However, an accurate measurement of roughness and pavement condition assessment can be achieved using those devices.

Most local engineering offices such as small counties and townships cannot afford an expensive profilometer. Also, a high-speed profilograph cannot always be operable in low-volume roads since the rural roads have often lower posted speeds due to geometrical design restrictions, such as narrow lane width and not having enough shoulder space. For this reason, relatively cheaper and user-friendly technology is needed to access the low-volume road conditions. More recently, cell phone-based apps have been introduced to measure pavement condition in terms of IRI. With the advent of advanced technology, cell phones with various sensors and GPS are used for road condition assessment. Apps can compute IRI values with the accumulation of vertical displacements of vehicles with the respective distance. Cell phone apps allow users to map road surface; the method is less time-consuming, and easy to operate.

Only a few studies can be found in the literature on measuring IRI using cell phones (13–16). A built-in accelerometer in a cell phone can measure road roughness and the relationship between the accelerometer data and road roughness was found to be linear (17). A recent study conducted in Illinois showed that cell phone-based measured IRI and inertial profilometer-measured IRI showed good correlations with the data obtained on smooth pavements at an operating speed of 80 km/h (13). Another study in Wyoming indicated that the cell phone measured IRI data varied with speed and the most variation was seen while the vehicle was driven at 80 km/h (16). In Michigan researchers conducted a similar study to measure IRI using their cell phone app; the data at the initial state showed low accuracy. However, a much higher accuracy was achieved when the driver took repeated data on the same road segment (18).

Most of the previously completed research studies reported on cell phone app development in relation to specific research projects and did not make their app available for professional use. A limited number of studies has measured IRI using cell phone apps and provided them as available to purchase from online stores (14, 19). The cell phone app developers also performed a few studies that showed promising results with their apps (20). This

study was intended to measure IRI of low-volume roads in Illinois using two commercially available apps and present results for comparison.

OBJECTIVE AND SCOPE

This study aims to measure low-volume road roughness in terms of IRI using cellphone apps. Two commercially available Android operating system-based apps Roadroid and RoadBump were used in this study. One cell phone and four different cell phone mounts were used to check the variability of the IRI data for various phone mounts. The cell phone was mounted on a truck and a sedan car to observe the variability in roughness results between two vehicles type. Also, on a few low-volume roads, the IRI values recorded with two apps are compared with the inertial-laser profiler measured IRI data to validate the app results.

METHODOLOGY

High-speed inertial-laser profiler and cell phone apps are used to assess low-volume road roughness. The truck hosting the laser profiler with assisting equipment is shown in Figure 2. The profilometer was first calibrated concerning its displacement transducer, accelerometer, and laser. The calibration distance for the profiler was 200 m. After calibrating the device, the truck was driven at a speed of 30 to 80 km/h. The output data was recorded through the attached laptop.

The cell phone was mounted on the truck as well as in the sedan car, and the apps were calibrated according to the procedure for data collection. Figure 3 shows the cell phone mounted in the vehicle. Only Roadroid needed calibration to set the 3-D accelerometer to its zero coordinates. The vehicle was driven at speeds in the range of 30 to 90 km/h. For a specific target speed, the vehicle has driven both directions, i.e., northbound and southbound or eastbound and westbound, and the data were averaged for both directions. Each time the vehicle was driven approximately 200 m to collect the IRI data as it was done similarly for the profiler.

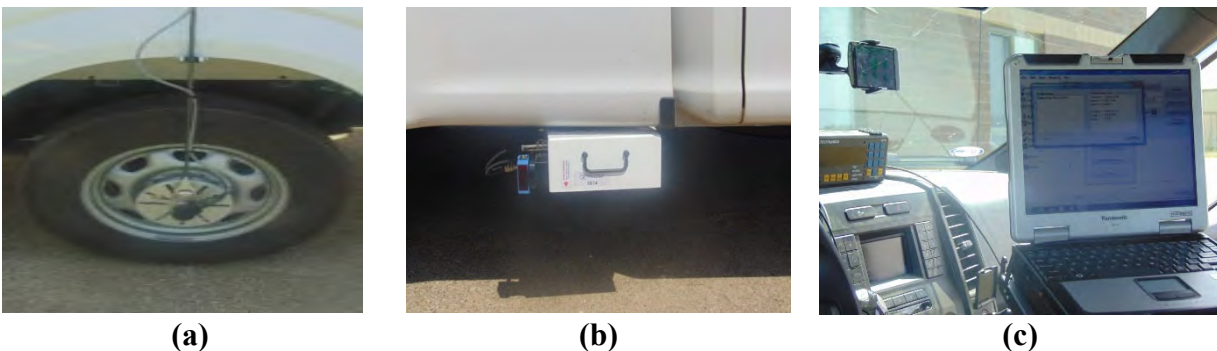


FIGURE 2 High-speed inertial-laser profiler: (a) displacement transducer; (b) accelerometer and laser source; and (c) recording IRI using laptop.



FIGURE 3 Cell phone apps used in this study: (a) Roadroid app on cell phone and (b) RoadBump app on cell phone

For the Roadroid app, the data were recorded and uploaded on the developer's website from a registered mobile phone IMEI number. The website provided the roughness values in terms of IRI with respect to average speed and length of the run. Also, the website displayed the Google map of the site and labeled the IRI with the color code on the map. Roadroid provides both subscription and free limited version, but for research purposes, the provider gives access to full subscription for a limited time, and that can be renewed by the developer. For RoadBump, the IRI data were captured instantly after each run. RoadBump also provides GIS map and the variation of IRI on a graph, but the app needs to be purchased from the store to receive that added service.

Cell phone apps collect data based on vehicle response while driving. There are different setups to be studied for recording the IRI data. The IRI data collection might vary in different cell phones, cell phone mount, and the vehicle itself. This study was conducted with one Samsung S3 type cell phone and four different mounts, i.e., dashboard, windshield with short arm, windshield with long arm, and air vent mount. Figure 4 shows the four mounts used in this study.

DATA COLLECTION

The IRI data were collected for low-volume roads in the summer seasons of 2017 and 2018. All the low-volume roads surveyed were chip-sealed roads. Figure 5 shows the 12 counties where the IRI data were collected using cell phone apps. IRI data were collected in three counties using a high-speed laser profilometer, and the locations are highlighted in the map in Figure 5.

Roadroid app was used in 2017 summer and fall seasons to collect the IRI data along with the high-speed laser profilometer to check the consistency of the data. Later in the summer of 2018, Roadroid and RoadBump apps were used with four different mounts to test the variability of the IRI data collected. Also, limited data were collected with both an SUV and a van to compare the results with those from the sedan car and truck to check the differences in the IRI data. For brevity, this paper presents only limited results of the total collected data.

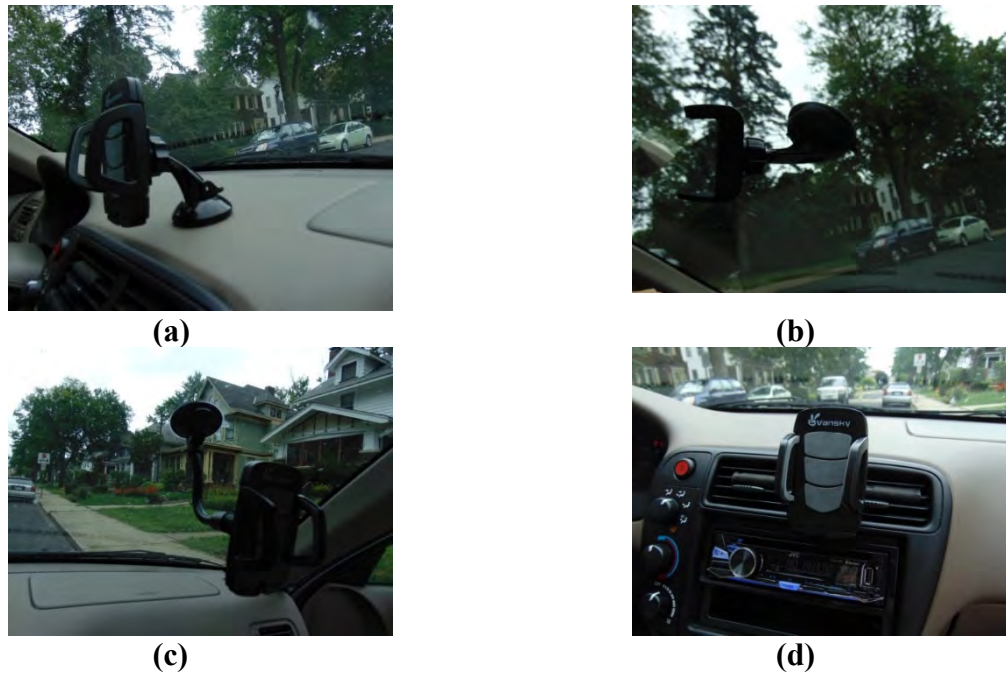


FIGURE 4 Different types of cell phone mounts used to collect IRI data: (a) dashboard mount; (b) windshield mount with short arm; (c) windshield mount with long arm; and (d) air vent mount

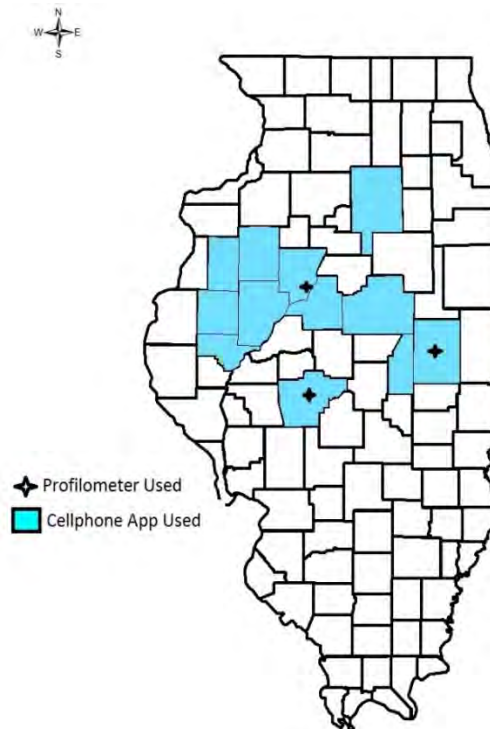


FIGURE 5 GIS map shows the location of counties visited to collect IRI data using cell phone apps and profilometer.

RESULTS AND DISCUSSION

Establishing Driving Speed Range to Collect IRI data on Low-Volume Roads

The profilometer could record IRI data at a speed of 80 km/h. However, the low-volume roads have lower driving speed, and for this reason, it is necessary to check if the cell phone apps can provide quality data at lower driving speed. IRI data were collected at a variable speed using Roadroid app and profilometer. Figure 6 shows the IRI value collected by Roadroid app and profilometer in a seal-coated road located in Peoria County. The cell phone was placed in the truck that was carrying the profilometer, and then the cell phone was placed in a sedan car. The cell phone as mounted using a long-arm cell phone mount and the mount was recommended by the Roadroid. For each speed, two data were taken, one for northbound and other for southbound and the average was calculated. According to Figure 6, profilometer-measured IRI values are in general the lowest, and they are consistent at different vehicle operating speeds. Data measured with the Roadroid app in sedan car and truck show an increasing trend with an increase in speed. Note that the variation in app data was higher when the cell phone was mounted on the truck. Also, for speeds above 70 km/h, Figure 6 shows a more significant difference in the IRI data. It was experienced that driving above 70 km/h did not feel safe since the roads were narrow, and there was no paved shoulder along the seal-coated roads.

The second set of IRI data was collected using profilometer mounted on a truck and the cell phone with Roadroid app mounted in the sedan car. The driving speed was less than 70 km/h for all cases. For each speed, two data sets were collected in both directions, and the average was calculated. Figure 7 shows the collected IRI data, and the profilometer data are again consistent at different vehicle operating speeds, as indicated previously. The Roadroid IRI data measured in the sedan car are closer to the profilometer-measured IRI data at lower speeds (Figure 7).

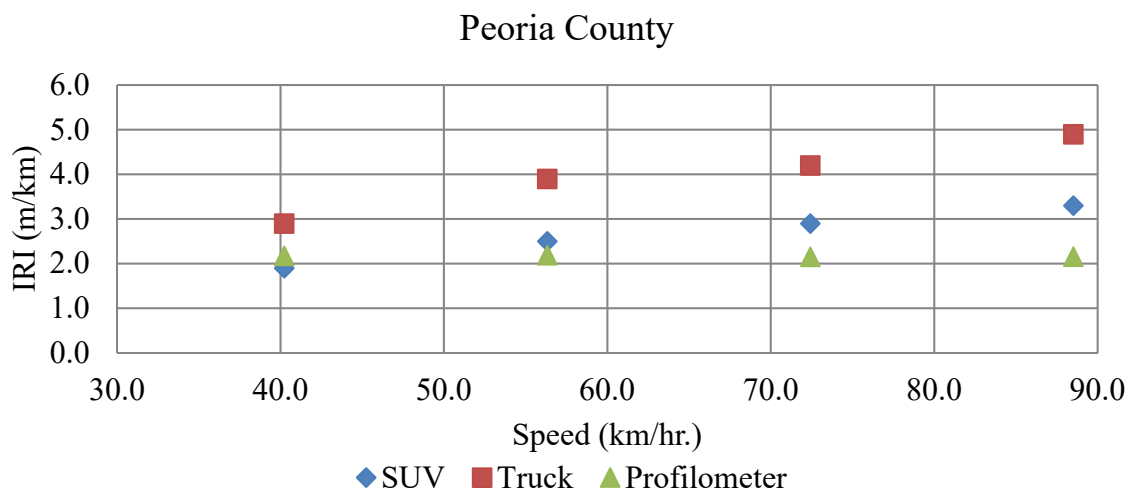


FIGURE 6 Roadroid- and profilometer-measured IRI data.

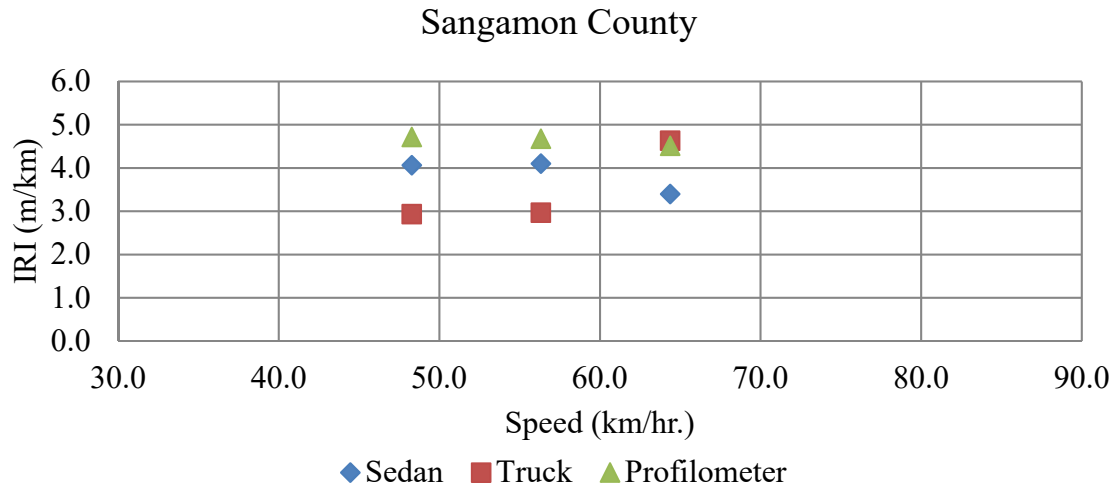


FIGURE 7 Roadroid- and profilometer-measured IRI data.

The third set of IRI data was collected using Roadroid, RoadBump, and the profilometer on a low-volume road that has both smooth and rough lanes. The lane designation as smooth or rough was determined using the profilometer. For each speed, two data sets were collected in both directions, and the average was calculated. As shown in Figure 8 and Figure 9 using the profilometer, the IRI values measured were 1.5 m/km for the smooth lane and 1.8 m/km for the rough lane, respectively. Comparing Figures 8 and 9, Roadroid-measured IRI values are closer to the profilometer-measured IRI values. For both the smooth and rough lanes, the Roadroid underestimated the IRI data when compared to the profilometer data.

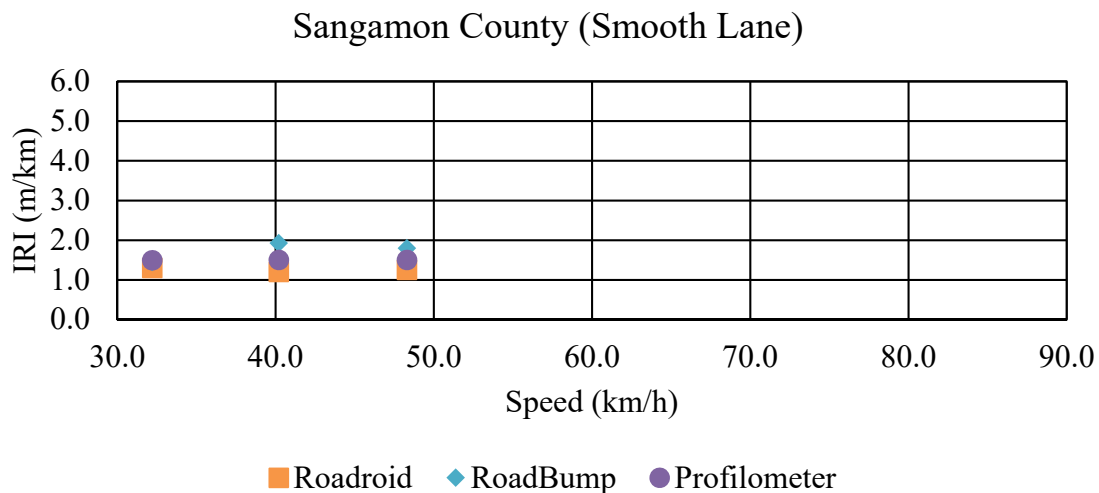


FIGURE 8 Roadroid-, RoadBump-, and profilometer-measured IRI data in smooth lane.

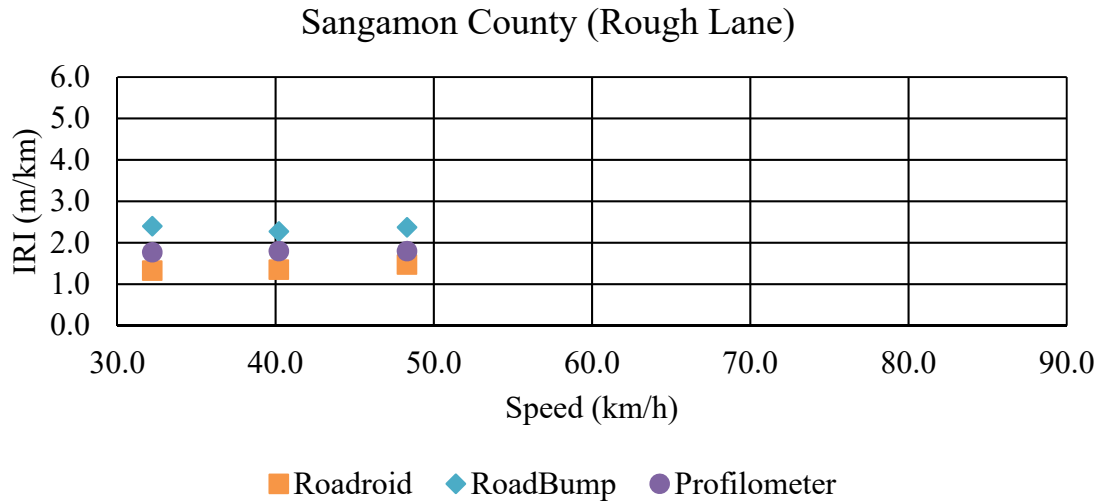


FIGURE 9 Roadroid-, RoadBump-, and profilometer-measured IRI data in rough lane.

Comparing Figures 6 through 9, using the cell phone apps, IRI data are in general consistent if the data were collected at 30 to 50 km/h driving speeds. Note that 50 km/h is a more acceptable speed to obtain the IRI data without slowing down the existing traffic. For this reason, the IRI data were collected with the apps at the recommended driving speeds.

Effect of Vehicle Type in Measuring IRI

Roadroid and RoadBump apps were used with the profilometer, an SUV, and a van with light and heavyweight, and a sedan car. The results are tabulated in Tables 1 and 2. The individual IRI value for each vehicle and the average IRI value of all vehicles are compared.

Using the Roadroid app in the smooth lane, sedan car gives IRI values comparable with the profilometer-measured IRI data. Light and heavyweight vans do not show appreciable differences at the same operating speed. An SUV gives lower IRI values compared to both the sedan and the van at low speeds. Using RoadBump in the smooth lane, sedan car gives higher IRI values than the profilometer-measured IRI data. Van with low and heavyweight combinations do not show consistent results when compared to the results obtained from the sedan car and SUV.

Using the Roadroid app in the rough lane, the sedan car under-predicted the Profilometer IRI data. However, van and SUV show higher IRI values on a rough surface when compared to the values obtained on the smooth surface. Using RoadBump in the rough lane, the sedan car shows higher values compared to the profilometer-measured IRI data. In general, the lightweight van shows higher IRI values when compared to those obtained from the heavyweight van. Comparing the results from the SUV and van, lower IRI values were reported from the SUV when compared to the heavy and lightweight van combinations.

In most cases, RoadBump showed higher IRI values when compared to those reported by the Roadroid app. In both smooth and rough lanes, the sedan car gave more consistent results than other vehicles. However, it is difficult to make a definitive conclusion from the limited data.

TABLE 1 Roadroid and RoadBump Use in Different Vehicles and IRI Comparison with Profilometer in Smooth Lane

Speed (km/h)	IRI (m/km)					
	Roadroid (Smooth Lane)					
	SUV	Van (5 Persons)	Van (2 Persons)	Sedan	Average	Profilometer
32	1.2	1.3	1.3	1.4	1.3	1.5
40	1.1	1.2	1.2	1.3	1.2	1.5
48	1.2	1.2	1.2	1.4	1.2	1.5
RoadBump (Smooth Lane)						
32	1.0	1.2	1.7	1.9	1.5	1.5
40	1.1	2.5	2.1	2.0	1.9	1.5
48	1.1	2.1	1.9	2.1	1.8	1.5

TABLE 2 Roadroid and RoadBump Use in Different Vehicles and IRI Comparison with Profilometer in Rough Lane

Speed (km/h)	IRI (m/km)					
	Roadroid (Rough Lane)					
	SUV	Van (5 Persons)	Van (2 Persons)	Sedan	Average	Profilometer
32	1.3	1.4	1.3	1.3	1.3	1.8
40	1.4	1.4	1.3	1.3	1.4	1.8
48	1.4	1.5	1.4	1.6	1.5	1.8
RoadBump (Rough Lane)						
32	1.4	2.4	3.6	2.2	2.4	1.8
40	1.6	2.6	2.4	2.5	2.3	1.8
48	1.4	2.7	2.8	2.6	2.4	1.8

Effect of Cell Phone Mounts in Measuring IRI

Four different phone mounts were used as shown in Figure 4 with both Roadroid and RoadBump apps. Data collected in two sites are presented in this paper. Figure 10 and Figure 11 show the variations of IRI data measured by Roadroid and RoadBump apps, respectively, at various speeds and for all four mounts. Comparing Figures 10 and 11, RoadBump gives higher IRI values in all cases compared to Roadroid. Furthermore, comparing results from the four mounts, the dashboard mount is associated with more consistent results for both Roadroid and RoadBump apps.

In Fulton County, four different mounts were used with Roadroid and RoadBump apps. The results are shown in Figure 12 and Figure 13. Again, RoadBump shows higher IRI values compared to those from the Roadroid app. More consistent IRI values were obtained using the Roadroid app for all four mounts.

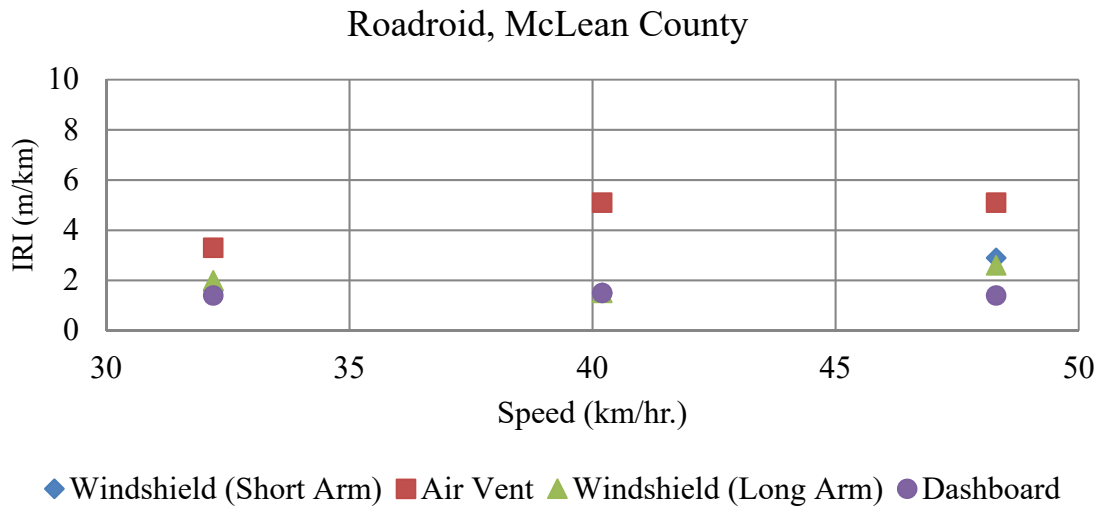


FIGURE 10 Effect of cell phone mounts on Roadroid-measured IRI data.

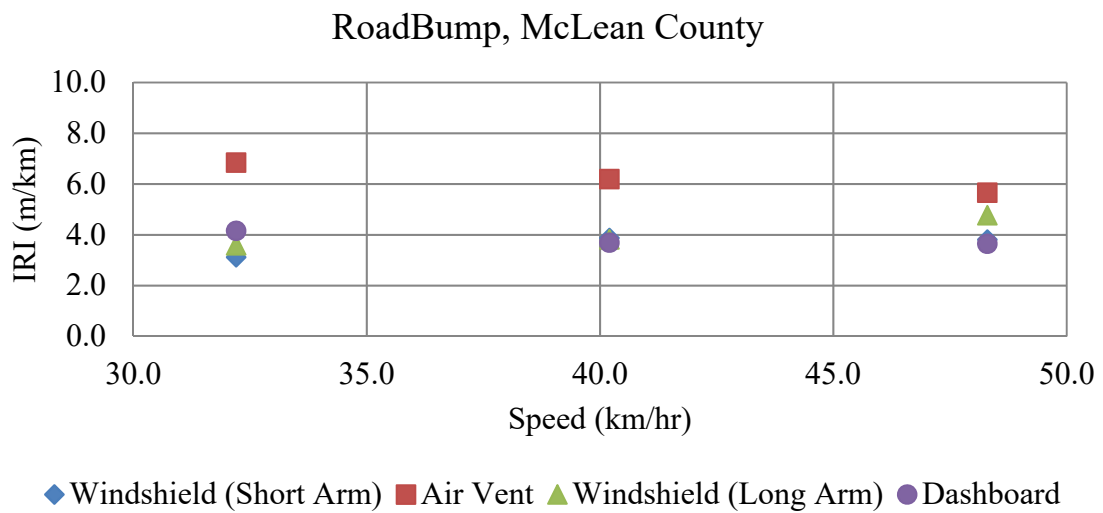


FIGURE 11 Effect of cell phone mounts on RoadBump-measured IRI data.

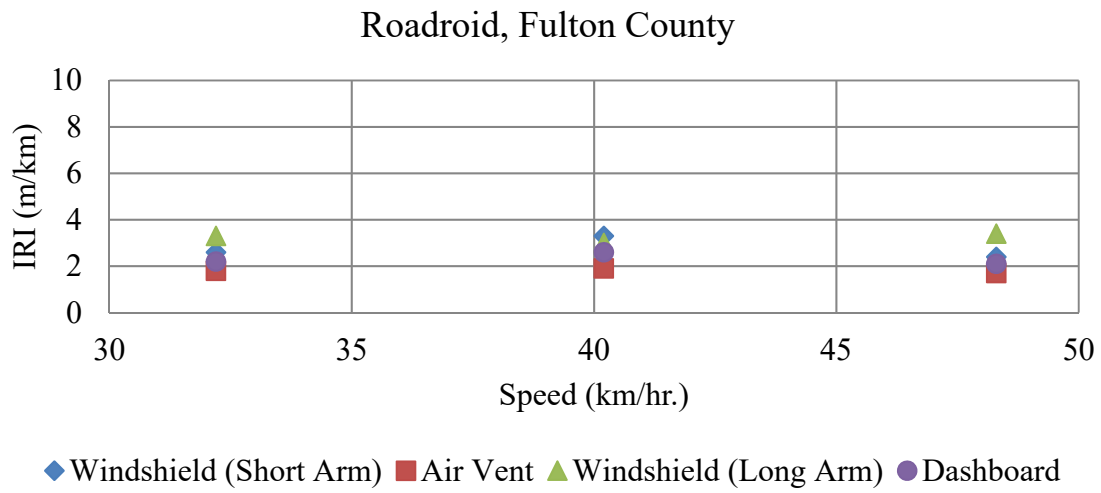


FIGURE 12 Effect of cell phone mounts on Roadroid-measured IRI data.

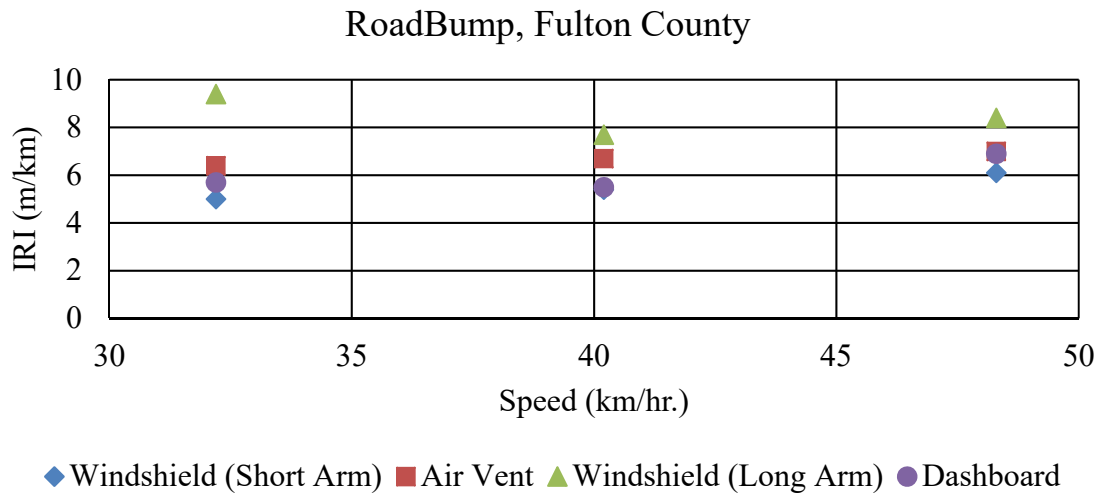


FIGURE 13 Effect of cell phone mounts on RoadBump-measured IRI data.

Four different mounts were further evaluated in Knox County and the results are presented for Roadroid and RoadBump apps in Figure 14 and Figure 15. As expected, RoadBump app shows higher IRI values compared to those obtained using the Roadroid app. More consistent IRI values were obtained again using the Roadroid app for all four mounts.

SUMMARY AND CONCLUSIONS

Cell phone apps can be used to measure low-volume road roughness. This paper presents a recent research study conducted at the Illinois Center for Transportation to evaluate two Android operating system-based apps, Roadroid and RoadBump, to measure the road roughness

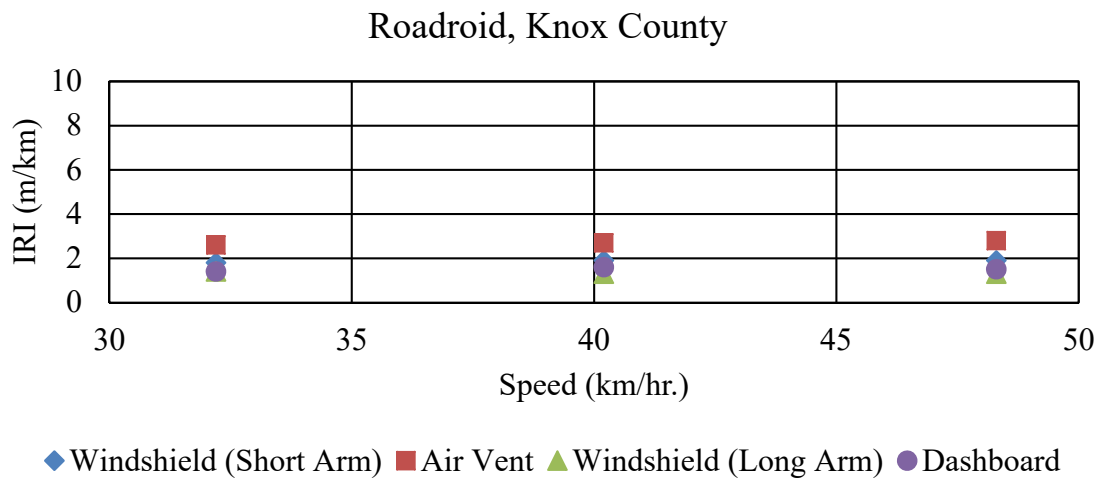


FIGURE 14 Effect of cell phone mounts on Roadroid-measured IRI data.

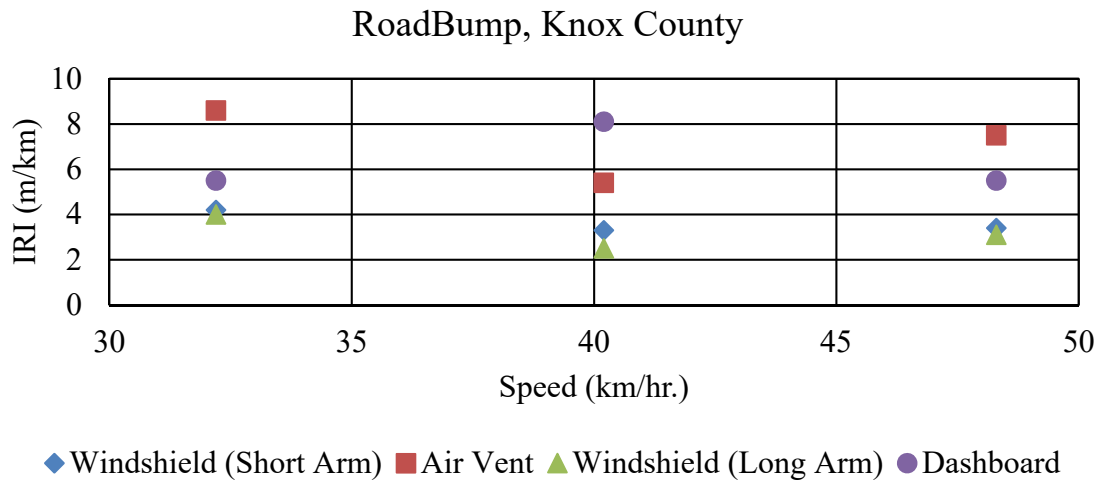


FIGURE 15 Effect of cell phone mounts on RoadBump-measured IRI data.

conditions of low-volume roads in Illinois through IRI). The study results indicate that Roadroid app provides somewhat better results in measuring IRI values when compared to the IRI data obtained by using RoadBump. The Roadroid IRI results were reasonably close to the high-speed profilometer-measured IRI data. A consistent set of IRI data could be obtained from different vehicles operated at 30- to 50-km/h speeds on the surveyed low-volume roads. Roadroid app also provided consistent results for both the smooth and rough pavements. Dashboard mount with Roadroid app was found to offer the most reliable IRI values. Since this study included limited data and research scope, a definitive conclusion is often difficult to make regarding the most suitable vehicle among a sedan, van, and truck utilized herein to mount the cell phone and test the cell phone app performance. The sedan car, however, provided better results in general for measuring IRI with Roadroid app.

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Utilizing Smartphones and Image Processing Analysis for Evaluating Dust on Gravel Roads

OMAR ALBATAYNEH

KHALED KSAIBATI

University of Wyoming

LARS FORSLÖF

Roadroid

Dust is one of the inherent problems related to gravel roads. Dust generated from gravel roads can cause health issues and safety concerns. Data collection of dust level is one of the main challenges for local agencies. For several years significant efforts have been devoted to the use of smartphones in data collection of roads' conditions. This paper introduces an advanced image-processing algorithm to detect and classify the dust amounts on gravel roads. Image-processing approaches have been adopted by using images taken from a smartphone application, Roadroid. The images are taken at a frequency of 100 ms, then they are sent to a web server to be analyzed. The simple dust classification algorithm (SDCA), which is the first phase of this study, has been conducted and validated against actual "dustometer" measurements. The results are promising; they showed that there was no statistically significant difference between the SDCA and the dustometer measurements. A future phase will develop the SDCA with an advanced machine learning modules to include surrounding roadway features to have a more-comprehensive algorithm.

INTRODUCTION

Low-volume roads, gravel roads, and access roads have a significant influence in various countries' economies. Generally, gravel roads are considered inexpensive and affordable roads to most local agencies due to the low cost of the materials (stone, sand, and fines) used in the construction process. However, they have a greater demand than paved roads for rehabilitation and maintenance. Currently, there is no easy and inexpensive comprehensive objective method (numerical based) for determining the dust level on gravel roads. Many technologies are contributing to identify an objective method for rating and evaluating gravel roads' dust. Smartphone technologies are taking the lead towards finding the best methods and solutions to identify gravel roads' dust by providing more-accurate condition data even though gravel roads' conditions change frequently due to traffic and environmental conditions.

In recent years, the development of smartphones and their applications have impacted the transportation industry. In many instances, smartphones are used by engineers and developers to collect data and measure roads' conditions by built-in sensors and cameras that have the ability to capture a high-resolution GPS-linked images (or videos) of the roads. This study focuses on how to use the images taken from an Android application (Roadroid) to analyze the inherent problem of the dust resulted from gravel roads. The smartphone application used in the study captures the

images at set intervals of 100 m; sends them to a web-server (cloud); and then processes by developed image-processing algorithms to analyze the content of dust in the images.

Monitoring the road networks and their conditions is a great challenge for engineers; especially local agencies with limited budgets (1). Roadroid system technology is one of the pioneering and innovating technologies providing engineers and decision-makers with a road survey and data about the road conditions whether they are paved roads, unpaved roads, or footpaths. This system will allow the users to conduct extremely cost-effective road surveys and to collect a larger amount of data than the traditional data collection methods. Moreover, this system can automatically take images at fixed intervals then upload them via web server, by which the users can have a huge database to view and analyze without leaving the office. Thus, the number of the tested gravel road can be easily expanded.

METHODOLOGY

This section will clearly state and describe the general framework of this study. Furthermore, the research practices will be also highlighted.

Roadroid System

Roadroid is a system used to monitor road conditions by using the built-in vibration sensors in smartphones. This system consists of two main parts: the Android application and Internet service. The Roadroid application picks up the vibrations coming up through the vehicle from the built-in accelerometer in the smartphone and sample them at 100 samples per second (100 Hz) (2). The data provided is GPS-linked so that the road coordinates can be identified easily. Roadroid is considered as an innovative and simple method to collect data to be used in road asset management.

Image-Processing Algorithms

The current state-of-art is that this method will provide the users with realistic images from the application to the point that they have more information and data of what a human eye can capture at the same conditions. In order to achieve fast processing times, the study first deals with one of the simple image processing algorithms. The Simple Dust Classification Algorithm (SDCA) was first developed based on a AForge.NET class library. This SDCA provides extracted information and data from images taken Roadroid application from gravel roads. This developed algorithm works mainly based on the analysis of the image contrast and color spectra to detect white pixels (dust particles) or black (something else). Table 1 depicts the general steps of the SDCA. Furthermore, SDCA provides a final score to rate the dust amount in three categories (low, medium, and high). The general framework of this algorithm, was developed based on comparing the actual dust measurement (dustometer) with the SDCA results. A standard cropping size for all images was used to remove irrelevant road surroundings in order to focus on the dust pixels in the image.

Advanced image-processing algorithms have huge number of parameters and training data. The next phase in this study, Phase 2, is to improve the SDCA by using new advance

TABLE 1 The Five Simple Dust Classification Algorithm Development Steps

Steps	Preprocessing	Global Thresholding	Remove Noise and Smooth	Feature Extraction	Classification
Sub Tasks	Contrast Enhancement and Cropping	Binarization and Color Filtering	Image smoothing using morphological closing; Noise filtering based on Blob size	Extract biggest Blob using connected component algorithm	Restate White-Pixel count

image-processing algorithms and artificial intelligence (machine-learning modules) in order to enhance the image feature extraction capabilities and include more parameters affecting the dust amount detection such as wind shifts, vehicles, and horizontal and vertical alignments.

FINDINGS

The newly developed cost-effective method of collecting data and information on low-volume roads have been introduced in this first phase of the study. SDCA have been tested on 30 gravel roads in Wyoming to classify the dust amounts. The results matched the actual dust amounts obtained from dustometer measurements. Figure 1 shows the setup of the smartphone used in developing and validating the the SDCA.

After the collection process, the data is sent to a web server to be analyzed; after that, the image-process algorithm provides the users with the dust amount on each image taken of the tested road as a downloadable text file. This file contains information about the tested gravel road such as location, distance, speed, altitude, and dust rating. Figure 2 clearly illustrate the classification process on the web server.



FIGURE 1 Smartphone arrangement.

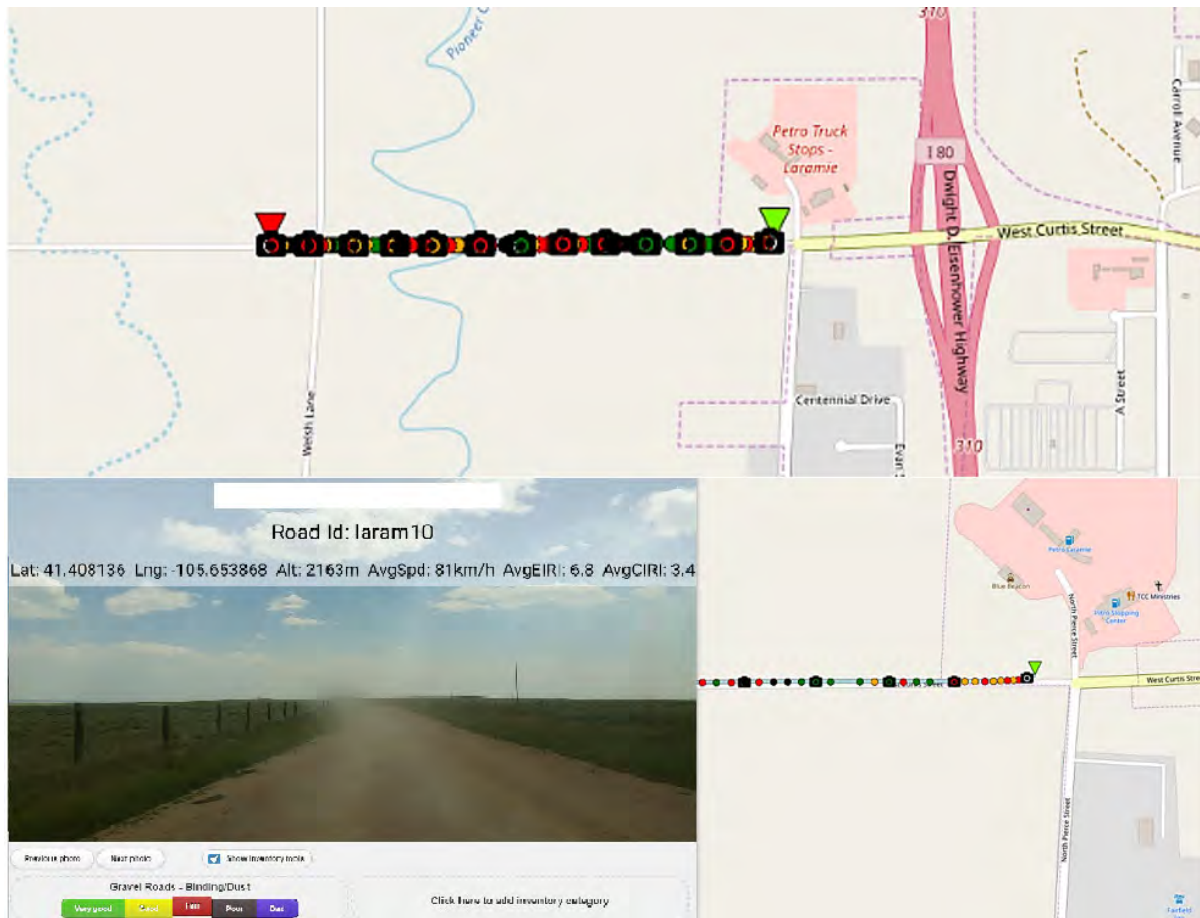


FIGURE 2 Dust classification process on web server.

CONCLUSION

This study deals with the issues of recognizing and classifying dust amount on gravel roads. The first phase of this study validated the developed algorithm for detecting dust roads' conditions. The process is easy and cost-effective for local agencies with limited budgets. The developed method demonstrated how modern technologies such as the smartphone applications can be used to help and assist the decision-makers in identifying dust conditions. Those conditions can be then utilized in allocating the most recommended and appropriate maintenance and rehabilitation activities to optimize dust conditions on gravel roads. Future studies will concentrate on enhancing the developed system by including other road features.

ACKNOWLEDGMENT

The authors gratefully acknowledge the generous financial support from the Mountain–Plains Consortium (MPC) for this study. All opinions, findings and results are solely those of the authors.

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PAVEMENT EVALUATION

Do-It-Yourself Street View

Using 360-Degree Camera Technology for Managing Low-Volume Roads

JOHN S. COBURN

Olsson

INTRODUCTION

Low-volume roads are the backbone to a rural state's economy. In Lancaster County, Nebraska, over 1,300 mi of roadway are maintained by the county and approximately 1,000 are graded, most of which could be classified as low-volume roads (1). Much of this rural roadway network have Google Street View imagery captured (by looking at <http://maps.google.com>), but much of it is nearly a decade out of date, with imagery taken back in 2007 (2). Although rural roads may not see the development booms that can be seen within or near urban cities, new culverts are installed, bridges repaired, or new access locations to fields may have been constructed over a decade's time, making that imagery out of date. And, much of that imagery in 2007 was a much lower quality than what Google Street View imagery can capture today.

Instead of waiting for updates to the Street View imagery by Google itself, rural counties and states can look to create their own updated street view imagery for their own uses, including asset management, field video surveying, and even uploading to Google's service for use by the general public. The objective of this paper is to show how it can be done easily and inexpensively using new camera technology.

What we know as Google Street View today began as a research project called the Stanford CityBlock Project back in 2001, which was funded by Google cofounder Larry Page. The goal of CityBlock was to summarize video with a few images. Five years later, the technology developed through this research project was folded into Street View and is still in use today (3). Competitors to Street View have surfaced, including Mapillary, Bing Maps, and Mapquest (4). As the last decade has progressed, imaging technology has improved, as can be seen in Figure 1 and Figure 2, showing Lincoln Nebraska's Haymarket District.

As Figure 1 and Figure 2 show, the quality has improved dramatically, so much so, that you are able to read text on manhole covers or see irregularities in the pavement. From the authors' antidotal analysis of Google Street View on rural segments of low-volume roads near Lincoln, Nebraska, it seems like most of the imagery has not been updated by Google since the first imagery runs of roadway back in 2007–2009 (2).

One of our first steps as a consulting engineer who work on roadway design projects is to receiving field data from our survey crews and performing an on-site visit. However, some of our projects are hundreds of miles away from our offices in remote locations, so we initially "visit" them using Google Street View, before we make the official on-site visit with our clients [department of transportation (DOT) or county employees] months later after our first plan submittal.

Our firm recently started a new project in Valentine, Nebraska, in the north-central portion of the state of Nebraska. We had our site visit scheduled 2 months after our project began, so we initially took a "visit" using Street View. After accessing the Google Maps website,



FIGURE 1 June 2009 Google Street View imagery of Lincoln's Haymarket District (2).



FIGURE 2 August 2017 Google Street View Imagery of Lincoln's Haymarket District (2).

we discovered that the Street View data was minimal (only half of our project had imagery available), and in the data that we were able to access the imagery was low in quality and was more than a decade out of date. Our team therefore invested in an Insta360 Pro 360° Camera and was able to take it along on the official site visit to record panoramic video imagery along the entirety of our project corridor, including potential detour routes on county highways. This video has been beneficial for a variety of staff in the firm, including from the firm's senior project manager to the student interns.

After obtaining this imagery, it was simple to process the data (using software supplied by the Insta360 company) and be able to output it in multiple video formats for the design process. This video is also able to be processed into a format used by Google and uploaded to their Google Maps service after an approval process, for not only our ease of use, but for our clients and the public at large.

Overall, the addition of this 360-degree footage has been a positive addition to our design process but could be utilized in many additional ways. Over a decade ago, as an intern for the Douglas County Engineers' Office in Omaha, Nebraska, the author was on a survey team that went out to document county-owned signs and pavement conditions on county-maintained roadways.

Although it may not fully replace the process to "grade" pavement conditions, 360-degree video capture could help augment and document deterioration over time by being able to compare side-by-side actual video footage of roads year over year. Sign surveys sometimes are used in the court of law, to decide fault of drivers in accidents (by finding out if a stop sign was present, for example). These sign surveys are time-consuming and may only be done once every 4 years (as budgets may prohibit an annual survey). Using 360-degree video can help provide more real-time inventory that could be useful in case that information is subpoenaed.

Other teams within our company have started to capture 360-degree video data as well, specifically for utility clients (who want to be able to view their power or fiber corridors), and our land development team, who want to be able to provide future lot buyers the ability to see new developments right after they have been graded and paved.

METHODOLOGY

The Insta360 Pro 360° Camera is a professional-quality camera but is as simple to use as many consumer-grade video devices. A sample setup for data capture includes

- Insta360° Pro Camera;
- Batteries, chargers, memory cards, or solid-state hard drives;
- High-quality vehicle mount;
- GPS capture device; and
- Cell phone, tablet, or laptop computer (to control the capture).

The cost of these devices varies between suppliers, but our set-up was approximately \$5,000 as of the writing of this abstract.

The capture of video is as simple as securely mounting the camera to your vehicle, turning on the camera, connecting the camera to your cell phone, tablet, or laptop, and clicking "record." The author has had success driving up to 40 to 50 mph with a high-quality professional vehicle

mount, but the manufacturer of the camera has suggested not traveling more than 38 mph, due to the possibility of degradation of video quality at certain frame rates.

After the video data is captured in the field, using the free Insta360 Stitcher application on your Windows PC or Mac is simple, and creates a 360-degree, 4-megapixel video file that can be played locally on your computer, or uploaded to YouTube. That captured video file can also be processed into a geolocated photo format that can be uploaded to Google Street View and Google Earth.

These raw video files are large, especially if you maintain the files in 8K format. One may have to work with their information technology professionals for their organizations to ensure the data is not a burden on their networks, or alternatively, invest in a small network attached storage device (with redundant backup capabilities) to be able to store the data.

FINDINGS AND CONCLUSION

The ease and quality of capture can bring many benefits to local agencies or consultants who need to manage or improve low-volume roads. A small investment in equipment and a little bit of time could provide local municipalities, counties, DOTs, consultants, and utility companies with more accurate data to use in their day-to-day practice.

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PAVEMENT EVALUATION

Evaluation and Performance of Low-Volume Roads

ALYSSA CLEMEN

HR Green

RICHARD A. REID

South Dakota State University

A majority of South Dakota roads are low-volume roads (LVRs). Simplified methods for LVR pavement thickness selection developed by the South Dakota Department of Transportation (DOT) allow users to obtain recommended asphalt surfacing and base course layer thickness based on the subgrade support and the average daily traffic (ADT). The main objective of this study is to relate the current surface condition of existing LVRs to their ADT, layer thicknesses, foundation material properties, and maintenance. A secondary objective was to determine if the surface condition of the roads is related to the level of compliance of the construction with LVR material and thickness selection methods.

Sixteen asphalt surfaced LVRs were evaluated and tested. The construction and maintenance histories of the roads were obtained and the performance of the roads was evaluated based on the surface condition of the pavement. Field tests were performed to obtain the layer thicknesses, dynamic cone penetrometer index, and material samples were obtained for laboratory testing. Laboratory testing included moisture content, liquid and plastic limit, and gradation tests of the foundation soils. Test results were used to compare the layer thicknesses and material properties to the performance of the roads and the suggested thicknesses and material specifications from the South Dakota DOT guides.

Based on the analysis, it was determined that layer thicknesses, material quality, and maintenance schedule have the greatest positive impact on a road's performance when all of these components satisfy the guidelines in the South Dakota DOT *Rural Road Design Guide*.

INTRODUCTION

A majority of roads in South Dakota are low-volume roads (LVRs) and field observations have determined the condition of these roads within the system can vary widely. In some cases, newly constructed roads exhibit significant distresses within a few years after construction while others appear to have minimal distresses after decades of service without significant rehabilitation. This study was undertaken to assess what characteristics distinguish the pavements performing with significant longevity from those that do not.

Simplified methods for LVR pavement thickness selection developed by the South Dakota Department of Transportation (DOT) allow users to obtain recommended asphalt surfacing and base course layer thickness based on the subgrade support and the average daily traffic (ADT). The main objective of this study is to relate the current surface condition of existing LVRs to their ADT, layer thicknesses, foundation material properties, and maintenance.

A secondary objective was to determine if the surface condition of the roads is related to the level of compliance of the construction with LVR material and thickness selection methods.

TEST METHODS, CONDITION ASSESSMENT, AND DATA COLLECTION

The test methods and protocols used in this study consisted of both the field testing and laboratory testing. The field testing consisted of the surface condition assessment, the dynamic cone penetrometer (DCP) test, and obtaining base course and subgrade samples. The laboratory testing included sample reduction and testing for moisture content, particle size and gradation, Atterberg limits, and soil classification. In addition to testing, information about the age, ADT, average daily truck traffic (ADTT), original thickness design, and maintenance history of each road was obtained, if it was available.

SURFACE CONDITION ASSESSMENT

The surface condition of the asphalt pavement was assessed to determine the performance of the pavement. The DCP test was performed on the base course and subgrade soil to obtain a penetration resistance of the soil. A boring was also done at each site so that the layer thicknesses could be measured and samples of the base and subgrade could be obtained for laboratory testing.

The different pavement conditions and distresses that were used to assess the pavement were weathering, oxidization, raveling, shoving, transverse cracks, longitudinal cracks, block cracks, alligator–fatigue cracks, rutting, patching, potholes, edge deterioration, and rideability. The frequency and severity of these distresses determined the rating that the pavement surface received. The pavement ratings were determined following the rubric presented in Table 1.

For the purposes of this study, the six different rating categories from the *Rural Road Condition Survey Guide* were condensed into three different rating categories as indicated in Table 1. The three resulting categories were

- Category 1:** poor and very poor to failed;
- Category 2,** good and fair; and
- Category 3:** excellent and very good.

Each pavement surface was rated following the guidelines provided by the *Rural Road Condition Survey Guide* for assessing the pavement surface (Beckemeyer, 1995).

FIELD TESTING

The DCP test was used to estimate the support of the base aggregates and the subgrade soils of the roads. The DCP test was performed at three locations for each test site: in the left lane outer wheelpath, in the right lane outer wheelpath, and on the centerline.

A boring of the base aggregate and subgrade soil was taken to obtain samples for laboratory testing. The boring was performed in the roadway near the location where the DCP tests were performed. Only one boring was performed for each test section to help minimize the

TABLE 1 Flexible Pavement Rating and Evaluation Scheme (Beckemeyer, 1995)

Rating	Surface Condition Description
100 to 86 (excellent)	The pavement surface is in excellent condition. The pavement appears to be very smooth and is generally free of any distress. As the pavement nears a rating closer to the lower end of this category, some oxidation of the pavement surface may be present, and minimal amounts of low-severity hairline cracks or depressions may be visible.
85 to 71 (very good)	The pavement surface is in very good condition, but surface deterioration is more evident. The pavement surface may be partially oxidized or weathered. Transverse and longitudinal cracks are visible, and crack widths are generally less than 3 mm (1/8 in) wide. Block cracking patterns may be appearing, but cracks have not deteriorated greatly. Some minor spalling or faulting may be present along the cracks. Additional types of surface deterioration may be present. Minor rutting may be noticeable in the outer wheel paths.
70 to 56 (good)	The pavement surface is generally in good condition. The surface is noticeably oxidized and raveling may be present. Transverse and longitudinal cracks are between 6 and 12 mm (0.25 and 0.50 in) wide and may exhibit some deterioration (spalling). Depressions in cracked areas or around utility repairs may be noticeable. Alligator cracking may be evident in the wheel paths. Rutting is becoming more pronounced, and some shoving may occur at intersections. Minor patching may be present as a result of surface distresses or utility settlements.
55 to 41 (fair)	The pavement surface is in fair condition. Pavement deterioration is much more advanced. Many reflective cracks are present on overlaid pavements. Block cracking is common and weathering is noticeable, with detrimental effects to the pavement. Some reflective cracks may be faulted or have medium- to high-severity spalls.
40 to 26 (poor)	The pavement surface is in poor condition with poor rideability. Alligator cracking is severe, and potholes may be present. Rutting is common and, in some instances, is greater than 20 mm (0.75 in). The pavement edge may be deteriorated, and over 60 m (200 ft) of cracking per 90 square meters (1,000 sq ft) of pavement is present.
25 to 0 (very poor to failed)	The pavement surface is in very poor to failed condition. The vast majority of the pavement surface is severely cracked and disintegrated. Traffic operations are severely affected.

Category 3

Category 2

Category 1

destruction of the pavement surface. A sealed bag of each the base aggregate and the subgrade soil was filled from the boring. These were the aggregate and soil samples brought back to the laboratory and used for the sieve analysis, Atterberg limits tests, in-situ moisture content and soil classification.

LABORATORY TESTING

Laboratory tests were performed on the samples of base course and subgrade soil that were obtained from the sites during field testing to determine the in-situ engineering properties of the materials. Tests were conducted to determine moisture content, particle size analysis, liquid and plastic limit tests. Based on these results the soils were classified by both the Unified Soil and AASHTO classification system. In each test, appropriate ASTM or AASHTO test standards were followed.

ROAD HISTORY

Information on each road and its history was obtained from the respective county or South Dakota DOT so that the analysis of this study would be as accurate as possible. For many roads, not all of the information is known, but the information that is known was obtained. The desired information included the age, original design, ADT, ADTT, and maintenance history for each road. All of these can affect the performance of a pavement.

SELECTED DESIGN GUIDES

Most local governments do not have information on or the ability to test in-situ materials for several parameters used in many design methods such as the resilient modulus and California bearing ratio (CBR). However, they are usually able to obtain ADT, ADTT, material gradations, and Atterberg limits. With only this available information to use in the design of their roads, they do not have the parameters needed for many design methods. However, two guides allow these local governments to obtain layer thicknesses with the information that they do have and can measure: the South Dakota DOT *Rural Road Design, Maintenance, and Rehabilitation Guide* (Beckemeyer and McPeak, 1995) and the South Dakota Local Roads Plan (South Dakota DOT, 2011). Therefore, only these two design guides will be used in this study.

SOUTH DAKOTA DOT RURAL ROAD DESIGN, MAINTENANCE, AND REHABILITATION GUIDE

The *Rural Road Design, Maintenance, and Rehabilitation Guide* provides suggested pavement and base layer thicknesses based on the ADTT and the quality of the subgrade support. Throughout the remainder of this study, the *Rural Road Design, Maintenance, and Rehabilitation Guide* will be referred to as “Rural Road Guide”. Typical layer thicknesses in South Dakota are 6- to 10-in. aggregate base for blotters and 2- to 6-in. asphalt concrete (AC) on top of 6- to 10-in. aggregate base course for AC pavements. The suggested layer thicknesses are shown in Table 2 and Table 3

TABLE 2 Suggested Gravel Layer Thicknesses for New or Reconstructed Rural Roads

Estimated daily no. of heavy trucks	Subgrade support condition ¹	Suggested minimum gravel layer thickness, mm (in)
0 to 5	Low	165 (6.5)
	Medium	140 (5.5)
	High	115 (4.5)
5 to 10	Low	215 (8.5)
	Medium	180 (7.0)
	High	140 (5.5)
10 to 25	Low	290 (11.5)
	Medium	230 (9.0)
	High	180 (7.0)
25 to 50	Low	370 (14.5)
	Medium	290 (11.5)
	High	215 (8.5)

for blotters and AC pavements, respectively. Although Table 2 is for gravel roads, it is also used for blotter-surfaced roads because the blotter provides no additional support for the traffic load. In this case, the suggested minimum gravel layer thickness in Table 2 would be the suggested minimum base course layer thickness under the blotter.

Table 3 presents the suggested corresponding AC surface layer thickness for a range of base course layer thicknesses at each ADTT range and subgrade support condition. These thicknesses were determined using the AASHTO method for rural roads with less than 200 heavy trucks per day, 20-year design life, design reliability of 75%, and structural coefficients of 0.10 for the aggregate layer and 0.36 for the AC layer.

When not determined from laboratory testing, the CBR used to determine the subgrade support conditions in Table 2 and Table 3 can be estimated two different ways: with the DCP, dynamic penetration index (DPI) and with the liquid limit (LL). The following equation may be used to estimate CBR from the DPI (Abu-Farsakh, 2004):

$$\log(\text{CBR}) = 0.84 - 1.26 \cdot \log(\text{DPI})$$

The approximate CBR values are listed in Table 4 for five different ranges of the LL (Beckemeyer and McPeak, 1995).

SOUTH DAKOTA LOCAL ROADS PLAN

The Local Roads Plan has recommended design minimums for layer thicknesses of asphalt surfaced roads for different ranges of ADT. These values shown in Table 5 are based on a life cycle of 20 years (South Dakota DOT, 2011).

TABLE 3 Suggested AC-Surfaced Pavement Thicknesses

Road classification and estimated daily truck traffic	Subgrade support conditions ¹	AASHTO structural number	Aggregate base thickness (in)	Corresponding AC layer thickness (in)
Light truck traffic (0 to 15 heavy trucks per day in design lane)	Low	2.89	6.0, 8.0, or 10.0	6.5, 6.0, or 5.5
	Medium	2.42	6.0, 8.0, or 10.0	5.0, 4.5, or 4.0
	High	1.88	6.0, 8.0, or 10.0	3.5, 3.0, or 2.5
Medium truck traffic (15 to 50 heavy trucks per day in design lane)	Low	3.44	8.0, 10.0, or 12.0	7.5, 7.0, or 6.5
	Medium	2.90	8.0, 10.0, or 12.0	6.0, 5.5, or 5.0
	High	2.27	8.0, 10.0, or 12.0	4.0, 3.5, or 3.0
Heavy truck traffic (50 to 200 heavy trucks per day in design lane)	Low	4.19	10.0, 12.0, or 14.0	9.0, 8.5, or 8.0
	Medium	3.55	10.0, 12.0, or 14.0	7.0, 6.5, or 6.0
	High	2.82	10.0, 12.0, or 14.0	5.0, 4.5, or 4.0

TABLE 4 Summary of Relationship Between LL and CBR of Typical South Dakota Soils

Liquid Limit, %	Approximate CBR, %
0 to 30	> 8.5
30 to 40	5.0 to 8.5
40 to 50	3.3 to 5.0
50 to 75	1.5 to 3.3
> 75	< 1.5

TABLE 5 Local Road Plan Surfacing Design Minimums for a 20-Year Life Cycle

ADT	Base	Surface
<400	8" to 10"	Asphalt Surface Treatment *
401 to 750	10"	3" Asphalt Concrete
>750	**	**
*Asphalt surface treatments need to be repeated @ every 4-5 years for optimum performance		
** Base and surface shall be designed according to current SDDOT Standards		

TESTING LOCATIONS

The selection of LVRs to be used for testing and sampling was based upon sites that would provide the necessary data for this study, as well as offer a collection of roads with varying surface qualities. Another consideration was that the locations needed to be in areas where the necessary equipment could be provided. Finally, the research team also needed to have access

and permission to cut out pieces of pavement surface to allow for DCP testing and the removal of base and subgrade samples.

All of the roads tested were located in the state of South Dakota, except for one road in Bowman County, North Dakota. An effort was made to select roads from a variety of counties and locations around the state. Only asphalt surfaced roads with low traffic volumes ($ADT < 2,000$) were included in this study. The specific sites were chosen in order to provide a variety of age and surface conditions among the different roads for the study. A limitation in the selection of test sites was that the county of the road undergoing testing had to be able to provide the auger and skid loader necessary to cut through the asphalt and sample the base, as well as traffic control during testing, in order for testing to occur.

For each of the selected test sites, the specific location for testing and sampling also had to be determined. If possible, the testing location was not on the top or bottom of a hill. This was done in order to avoid the possible fill sections at the top of a hill and sections that could be uncharacteristically weak due to excess moisture at the bottom of a hill. The testing location also had to be in an area that was a fairly average representation of the whole site in terms of the surface performance. At each of the testing locations, the field and laboratory testing was conducted as previously presented.

TESTING PARAMETERS

There were several parameters that were determined for each of the sample locations. Some of these parameters were obtained through research and the assistance of the county highway superintendents. This included ADT, ADTT, age of the road, and maintenance history. The other data was obtained through observation and measurements of the roads in the field and the results of laboratory tests. This data consisted of surface type, surface condition, surface and base thicknesses, base course gradation and plasticity index, base and subgrade moisture contents, and base and subgrade DCP penetration resistances.

PROCESS OF COMPARING ROAD PERFORMANCE ASSESSMENT TO RECOMMENDED DESIGN

The field and laboratory testing provides the necessary information to determine the required road thickness using the methods presented in the Rural Road Guide and in the Local Road Manual. This first design method uses the subgrade support and the ADTT to determine the required layer thicknesses. The latter design method depends only on the ADT. Both methods use the South Dakota DOT base course specification. The following steps describe how the data was used to validate the effectiveness of the design methods by determining the recommended design thicknesses and relating the design to the road performance.

1. Laboratory data was used to determine the material properties (gradation, penetration index, LL).
2. Base course material properties were used to determine if the materials met the state specification.

3. Subgrade material properties and DCP results were used to establish the subgrade support.
4. Subgrade support and traffic data were applied to the design methods to determine the required layer thicknesses.
5. Design layer thicknesses were compared to the in place layer thicknesses to determine the adequacy of the existing pavement layer thicknesses.
6. Surface condition data was assessed to determine the road rating.
7. Analysis was conducted to assess how layer thicknesses and material properties influence observed road performance.
8. The effectiveness of the design methods were validated based on how the determination of design thickness and material properties influenced observed road performance.

The result of this process will subsequently be used to evaluate the effectiveness of the LVR design methods.

SURFACE CONDITION RESULTS

The surface condition assessment was performed using the methodology presented by the Rural Road Guide (Beckemeyer, 1995). Table 6 lists each road under the category that indicates its pavement condition.

RESULTS SUMMARY

Table 7 is a summary of the field and laboratory evaluations along with the average daily traffic for each road. Most of the good performing roads have significantly greater pavement and base thicknesses than the other roads.

BASE COURSE QUALITY

The South Dakota DOT base course specification is presented in Table 8 and the gradations, LL, and PI for each base course sample determined during laboratory testing are shown Table 9. The

TABLE 6 Surface Condition Assessment Summary

Category 1	Category 2	Category 3
Harding – Hwy 867	Deuel – Hwy. 311 original	Lincoln – Hwy. 135
Bowman – 154th Ave.	Aurora – W. 262nd St. (Stickney)	Miner – Railroad St.
Beadle – Broadland Rd.	Pennington – Rockerville Rd.	Deuel – Hwy. 311 rehab
Aurora – E. 262nd St. (Stickney)	Brown – Hwy. 17	Codington – Old Hwy. 81
Pennington – Bombing Range Rd.	Aurora – 386th Ave. (Plankinton)	
Brown – Hwy. 14		
Clay – Saginaw Ave.		

TABLE 7 Summary of Results

Surface Condition	County	Rating	Average Pavement Depth (in)	Average Base Depth (in)	Pavement Type	ADT	Base Meets State Spec	Subgrade AASHTO Class.
Category 1	Harding	40-26; poor	0.625	7.25	Blotter	34	N	A-6
	Bowman	40-26; poor	1	6.25	Blotter		N	A-6
	Beadle	25-1; very poor	3.5	6.25	Mat	97	Y	A-7-6, A-6
	Aurora–E. 262nd St.	40-26; poor	1.25	9	Blotter	171	N	A-7-6, A-6
	Pennington	40-26; poor	3	8	Mat	663	N	A-7-6
	Brown–Hwy. 14	40-26; poor	10.5	1	Mat	1,400	N/A	A-7-6
	Clay	10-0; failed	1	7.625	Blotter	601	Y	A-6
Category 2	Deuel–Hwy. 311 orig.	55-41; fair	5	5.25	Mat	35	Y	A-6
	Aurora–W. 262nd St.	55-41; fair	2.75	6.25	Blotter	212	N	A-7-6
	Pennington Rockerville	70-56; good	6.75	8	Mat	530	N	A-4
	Brown–Hwy 17	70-56; good	1.75	4.375	Blotter	325	Y	A-7-5
	Aurora–386th Ave.	70-56; good	2.25	6.25	Blotter	113	Y	A-6
Category 3	Lincoln	85-71; very good	5.625	5.375	Mat	506	Y	A-7-5
	Miner	85-71; very good	6.5	15	Mat		N	A-7-6
	Deuel–Hwy. 311 Rehab	85-71; very good	3.25	8.25	Mat	370	Y	A-7-6, A-6
	Codington	85-71; very good	11.25	8.25	Mat	1,929	Y	A-6

TABLE 8 South Dakota DOT Specifications for Aggregate Base Course and Subbase (South Dakota DOT, 2004)

Requirement	Aggregate Base Course	Subbase
Sieve	Percent Passing	
2" (50 mm)		100
1" (25.0 mm)	100	70–100
¾" (19.0 mm)	80–100	
½" (12.5 mm)	68–91	
No. 4 (4.75 mm)	46–70	30–70
No. 8 (2.36 mm)	34–58	22–62
No. 40 (425 µm)	13–35	10–35
No. 200 (75 µm)	3.0–12.0	0.0–15.0
LL, max.	25	
Plasticity Index	0–6	0–6
L.A. abra. loss, max.	40	50
Processing required	Crushed	Crushed

TABLE 9 Subgrade Gradation, LL, Penetration Index, and AASHTO Classification

County–Road	Percent Passing				LL	PI	AASHTO Classification	
	No. 4	No. 10	No. 40	No. 200			Individual Sections	Road
Harding–Hwy. 867	100	97	79	58	37	23	A-6	A-6
Bowman–154th Ave.	89	83	77	47	32	14	A-6	A-6
	97	96	93	68	37	22	A-6	
Miner–Railroad St.	95	84	63	53	42	27	A-7-6	A-7-6
Beadle–Broadland Rd.	99	97	90	59	42	24	A-7-6	A-7-6
	100	99	94	65	42	23	A-7-6	
	99	97	90	60	40	24	A-6	
Deuel–Hwy. 311 Rehab	95	89	74	41	30	15	A-6	A-7-6
	98	94	81	53	41	21	A-7-6	
Deuel–Hwy. 311 Original	98	96	87	67	38	21	A-6	A-6
Aurora–E. 262nd St. (Stickney)	97	93	83	52	46	31	A-7-6	A-7-6
Aurora–W. 262nd St. (Stickney)	100	100	96	79	40	25	A-6	A-6
Aurora–386th Ave. (Plankinton)	97	93	85	60	34	20	A-6	A-6
Codington–Old Hwy. 81	98	95	81	61	41	23	A-6	A-6
	93	86	69	50	37	16	A-6	
	99	97	87	66	40	19	A-6	
Clay–Saginaw Ave.	99	98	91	76	38	20	A-6	A-6
Lincoln–Hwy. 135	99	99	94	80	54	32	A-7-5	A-7-5
Pennington–Rockerville Rd.	100	93	86	66	24	2	A-4	A-4
Pennington–Bombing Range Rd.	100	96	88	78	63	45	A-7-6	A-7-6
Brown–Hwy. 14	100	100	94	74	64	35	A-7-6	A-7-6
Brown–Hwy. 17	100	98	88	54	41	23	A-7-5	A-7-5

gradation, LL, and PI results for the samples were compared to the specification to determine if the base course satisfied this specification. Base course samples that had any one of the gradations, LL, or PI outside of the range for the specification by more than 2% passing or 2% moisture content were considered to not meet the South Dakota DOT base specification.

SUBGRADE CLASSIFICATION

The results of the laboratory tests, as well as the resulting AASHTO classifications for the subgrade soil samples are shown in Table 9. For the roads that had more than one subgrade sample and the resulting classifications were different, the subgrade was considered to be the lowest classification. This was done to be conservative because the lower classified soils have lower strengths than the higher classified soils.

SUBGRADE DCP

The subgrade support determined with the CBR calculated from the DCP test results are all higher than anticipated because these tests were performed throughout the summer when the conditions were drier than in the wet design conditions, which may lead to an unconservative estimation of CBR. Therefore, the subgrades would have been stronger when they were tested throughout the summer than they are in their weaker condition when it is wet. Flexible road design methods typically use subgrade strength parameters obtained when the subgrade is in its weakest state. Because of this the DCP results may not be accurate representations of the design conditions of the subgrade soils in their weakest state. This may overestimate subgrade strength conditions. Detailed review of the literature did not provide a reliable method to correct DCP results based on changes in moisture content. Had all the roads been tested during the spring thaw, then the DCP results may have been more consistent with the other methods used to estimate subgrade support in the Rural Road Design Guide. Therefore, the subgrade support values estimated from the DCP test results were not used as part of the analysis and the data is not presented in this paper.

ANALYSIS

The Rural Road Manual (Beckemeyer and McPeak, 1995) determines the subgrade support based on its CBR. The correlation between LL and CBR values was presented in Table 4. The ranges of CBR values that correlate to different subgrade supports are shown in Table 10 (Beckemeyer and McPeak, 1995).

TABLE 10 Subgrade Support Based on CBR

Subgrade Support	CBR
High	>10
Medium	3 – 10
Low	<3

The subgrade support may also be determined from its AASHTO soil classification. The AASHTO soil classifications presented in Table 7 show all the subgrade soils were classified as A-6 or A-7 soils, except for the Pennington County Rockerville Road subgrade, which was classified as A-4. According to the AASHTO classification system these soils all are considered fair–poor rating as a subgrade. For this study, the AASHTO subgrade rating of fair–poor was considered to be equivalent to the Rural Road Manual subgrade support of low.

Therefore, there are three different ways the subgrade support was determined in this study. However, not all three methods resulted in the same support. As previously discussed, the CBR estimated from the DCP testing was not considered. The subgrade supports determined from the AASHTO soil classification and LL correlation for each road are summarized in Table 11. The AASHTO classification always resulted in low subgrade support. The subgrade support determined with the CBR from the LL was consistently higher than the CBR estimated from the AASHTO classification.

The actual thicknesses of the layers for the roads were compared to the thicknesses required by the Rural Road Design Guide and the Local Road Plan. This comparison, as well as the ADTT used in the design are shown in Table 12. The actual ADTT was only known for three of the roads: Brown County Highways 14 and 17 and Deuel County Highway 311 Rehab. For the other roads with unknown ADTT, an ADTT range was estimated as 8% to 15% of the road ADT. If this estimated range fell into multiple ADTT levels in the Rural Road Design Guide, the higher ADTT level was used for the design comparison.

Only three roads satisfied the Local Road Plan thickness requirement: Aurora County E. 262nd Avenue, Miner County Railroad Street, and Deuel County Highway 311 Rehab. Also, the Local Road Plan design did not apply to Brown County Highway 14 and Codington County Old

TABLE 11 Subgrade Comparison

Surface Condition	County	Road	Road Age at Testing (years)	Subgrade Support	
				AASHTO	CBR from LL
Category 1	Harding	Hwy. 867		Low	Med
	Bowman	154 Ave.	1	Low	Med
	Beadle	Broadland Rd.	34	Low	Med
	Aurora	262st St E. (Stickney)	23	Low	Med
	Pennington	Bombing Range Rd.	15	Low	Low
	Brown	Hwy. 14	37	Low	Low
	Clay	Saginaw Ave.	10	Low	Med
Category 2	Deuel	Hwy. 311 Original	31	Low	Med
	Aurora	262st St. W. (Stickney)	23	Low	Med
	Pennington	Rockerville Rd.	21	Low	High
	Brown	Hwy. 17	Unknown	Low	Med
	Aurora	386th Ave. (Plankington)	Unknown	Low	Med
Category 3	Lincoln	Hwy. 135	53	Low	Low
	Miner	S. Railroad St.	16	Low	Med
	Deuel	Hwy. 311 Rehab	25	Low	Med
	Codington	Old Hwy. 81	78	Low	Med

TABLE 12 Comparison of Actual Layer Thicknesses to Design Thicknesses

			Actual Thickness (in.)		Rural Road Guide Thickness (in.)		SD Local Road Plan Thickness (in.)	
	County–Road	ADTT (8% – 15%)	Pavement	Base	Pavement	Base	Pavement	Base
Category 1	Harding–Hwy. 867	3-6	0.625	7.25	N/A	6.5	N/A	8–10
	Bowman–154 Ave.	0-5	1	6.25	N/A	6.5	N/A	8–10
	Beadle–Broadland Rd.	8-15	3.5	6.25	6.5	6	N/A	8–10
	Aurora–E. 262nd St.	14-26	1.25	9	N/A	11.5	N/A	8–10
	Pennington–Bombing Range Rd.	53-100	3	8	9	10	3	10
	Brown–Hwy. 14	280*	10.5	1	11.5	1	N/A (ADT >750)	
	Clay–Saginaw Ave.	48-90	1	7.625	N/A	14.5	N/A	8–10
Category 2	Deuel–Hwy. 311 Orig.	3-6	5	5.25	6.5	6	N/A	8–10
	Aurora–W. 262nd St.	17-32	2.75	6.25	N/A	14.5	N/A	8–10
	Pennington–Rockerville Rd.	42-80	6.75	8	9	10	3	10
	Brown–Hwy. 17	0*	1.75	4.375	N/A	6.5	N/A	8–10
	Aurora–386th Ave.	9-17	2.25	6.25	N/A	11.5	N/A	8–10
Category 3	Lincoln–Hwy. 135	40-76	5.625	5.375	9	10	3	10
	Miner–Railroad St.	**	6.5	15	6.5	12	N/A	8–10
	Deuel–Hwy. 311 Rehab	26*	3.25	8.25	7.5	8	N/A	8–10
	Codington–Old Hwy 81	154-290	11.25	8.25	9.5	8.25	N/A (ADT >750)	

*actual ADTT; **estimated ADT < 50

Blotter Mat

Highway 81 because they have ADTs greater than 750. The Local Road Plan design depends only on ADT and not the subgrade support and is considered a more conservative design method than the Rural Road Guide. For these reasons, it is difficult to evaluate the effectiveness of the Local Road Plan design within this study. Therefore, the remainder of the analysis will evaluate only the Rural Road Guide design.

Table 13 summarizes the results of the comparison between the actual and design thicknesses, as well as whether the base course meets the South Dakota DOT specification and whether the road received regular maintenance (regular chip seals). The design thicknesses were based on the subgrade support determined with the AASHTO soil classification only, not with the LL or DCP results. Approximately a third of the Category 1 performing roads had base course that met the state specification or received regular maintenance, and only two roads had adequate layer thicknesses. As presented in the table, adequate layer thickness is defined as the

TABLE 13 Comparison to Design Guides

Surface Condition	County	Road	SD Local Road Plan Adequate Thickness	Rural Road Guide Adequate Thickness	Base Meets State Spec.	Regular Maintenance
Category 1 (poor to failed)	Harding	867	No	Yes	No	< 1 year
	Bowman	154 Ave.	No	Yes	No	< 1 year
	Beadle	Broadland Rd.	No	No	Yes	No
	Aurora	E. Stickney	Yes	No	No	No
	Pennington	Bombing Range Rd.	No	No	No	Yes
	Brown	Hwy. 14	N/A	No	N/A	Yes
	Clay	Saginaw Ave.	No	No	Yes	No
Category 2 (good to fair)	Deuel	Hwy 311 Orig.	No	No	Yes	Yes
	Aurora	W. Stickney	No	No	No	Unknown
	Pennington	Rockerville Rd.	No	No	No	Yes
	Brown	Hwy. 17	No	No	Yes	Yes
	Aurora	386th Ave.	No	No	Yes	Unknown
Category 3 (excellent to very good)	Lincoln	Hwy. 135	No	No	Yes	Yes
	Miner	Railroad St.	Yes	Yes	No	Yes
	Deuel	Hwy. 311 Rehab.	Yes	No	Yes	Yes
	Codington	Old Hwy. 81	N/A	Yes	Yes	Yes

in-place thickness being equal to or greater than the thickness recommended by the Local Road Plan or Rural Road guide. Half (50%) of the Category 2 performing roads had base course that meets the specification. None of these roads had adequate layer thickness. There were also 60% of the Category 2 performing roads that received adequate maintenance. The maintenance records were not known for the two roads from Aurora County in this performance category. However, since these roads were constructed as blotters, their current thicknesses suggest that they have received a few chip seals throughout their approximately 20-year lifetimes. If it is therefore assumed that this satisfied adequate maintenance schedules, then all of the average performing roads received regular maintenance. Three out of the four Category 3 performing roads had base course that met the specification. Half of these roads had layer thicknesses that satisfied the design criteria, and all of these roads received regular maintenance.

Thickness appears to be one of the most important factors in pavement performance. Except for two roads, all those that had adequate thicknesses were categorized as Category 3 performers. Regular maintenance also seems to help a pavement perform as it meant to by preserving the pavement's condition. All of the roads that received regular maintenance, except for two, had either Category 2 or Category 3 performance, and all of the roads in these two performance categories had regular maintenance. When a base course that met state specifications was combined with a pavement that receives regular maintenance, they worked together to keep a pavement at Category 2 performance condition. When a good quality base was combined with adequate layer thicknesses and regular maintenance, the roads had Category 3 performance.

The ages of the roads tested were compared to see if the ages correlated to the pavement performance. It might seem reasonable for the newer roads to perform better than the older roads. However, this is not the case for the roads tested, as shown in Table 11. Some of the newest roads are Category 1 performers, and a couple of the oldest roads were Category 3 performers. There appears to be little correlation between the age of the road since it was built and its performance.

There are three main factors that can impact the performance of an asphalt surfaced road: subgrade support, thickness, and maintenance. Of these three factors, thickness had the greatest impact on the performance of the roads in this study. However, to build a good performing pavement, adequate thickness based on the Rural Road Guide should be combined with good base course quality and regular maintenance. On the other hand, the ages of the tested roads did not seem to have a significant correlation to their pavement performance.

SUMMARY

In this study, 16 different low volume, asphalt surfaced roads in the South Dakota area were tested in order to correlate design with performance. The age, maintenance, and traffic count information was obtained for the roads tested. For each road, the pavement was analyzed for its performance, the in-situ base and subgrade were tested with the DCP and measured for their thicknesses, and base and subgrade samples were obtained. The base and subgrade samples were tested for moisture, gradation, and Atterberg limits. Using the base gradation and Atterberg limit test results, it was determined whether each base course satisfied the South Dakota DOT base course specification. The subgrade gradation and Atterberg limit test results were used to determine the support each subgrade provided. The subgrade support, ADTT, and layer thicknesses were then used to compare each road to the applicable design recommendation in the South Dakota DOT *Rural Road Design, Maintenance, and Rehabilitation Guide* for each road. This comparison was then used with the result of the pavement performance evaluation to determine if the recommendations from this Rural Road Guide are adequate.

CONCLUSIONS

1. The three design requirements of base course and subgrade material quality, layer thicknesses, and maintenance schedule all contribute to the pavement performance of low-volume, asphalt-surfaced roads. When these three aspects satisfy the recommendations of the Rural Road Design Guide, they provide the support needed for the pavement to perform well. Therefore, this design guide provides adequate design recommendations for low volume, asphalt surfaced roads in South Dakota.

- a. If none of the design recommendations are satisfied, the road performance falls into Category 1.
- b. If only one of the design recommendations is satisfied, the pavement may have either Category 1 or Category 2 performance.
- c. If both the base course specification and regular maintenance are satisfied, the road performance will most likely be Category 2 but may also be Category 3.

- d. If both the required layer thicknesses and regular maintenance are satisfied, the pavement will have a performance of Category 3.
- e. If all three of the design recommendations are satisfied, the road have Category 3 pavement performance.
2. If actual CBR test data is not obtained, the AASHTO soil classification is the most suitable method, compared to using the LL or DCP results, for determining the subgrade support used in the Rural Road Design Guide.
3. The age of the road does not strongly correlate to the pavement's performance. Therefore, the way the road is built and the quality with which the road is built and maintained have a greater impact on the performance than the age of the road.
4. The DCP may not be an adequate measure of the base course and subgrade soil design support when tested throughout the summer, at lower moisture contents than the wet, spring design condition.

RECOMMENDATIONS

1. Due to the overestimation of subgrade strength by the summer DCP tests, it is recommended that:
 - a. DCP tests are run on these same roads in the spring during wet conditions and then the analysis repeated.
 - b. A methodology be developed to adjust DCP data based on water content variations and then the analysis repeated.
2. In this study, roads with both blotter and HMA flexible pavements were tested and analyzed together. If the number of roads tested was increased, then the two types of pavements could be analyzed separately to determine the effectiveness of the design methods for each type instead of for flexible pavements as a whole.
3. It is recommended that the Rural Road Design Guide be used as recommendations for the design of low-volume, asphalt-surfaced roads in South Dakota coupled with a regular maintenance schedule in order to provide good performing pavements when resources are unavailable for an engineered pavement design.

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Correlating Visual–Windshield Inspection Pavement Condition to Distresses from Automated Surveys Using Classification Trees

GEORGES BOU-SAAB

INYA NLENANYA

AHMAD ALHASAN

Iowa State University

INTRODUCTION

Pavement performance assessment is a crucial element in pavement management systems (PMS), where it directly impacts maintenance and rehabilitation requirements, as well as the needs for future funding (1). Pavement performance indices can be defined using one of three methods: direct panel rating, utility functions, and deduct values and weighting factors. Increased accuracy of the assessment method will lead to a more objective and accurate pavement performance index, and thus will lead to a more reliable and optimal maintenance decision (2).

A simple, convenient, and cost-effective way to assess the surface condition of pavements in a road system is through the assignment of a numerical rating known as the pavement condition index (PCI). Not only does the PCI play a significant role in monitoring the overall serviceability and physical condition of pavements, it also assists in identifying maintenance and rehabilitation needs and assures to wisely assign available funds. Each pavement section is evaluated systematically through inspection and observation. Roadway public officials employ their knowledge and experience to complete pavement condition surveys.

The development of PCI is based on estimated and measured condition parameters. Commonly, trained observers estimate conditions by visually inspecting and observing pavement surfaces then provide an overall rating index. In recent years, the process shifted towards automation such as laser crack measurement systems (LCMS) for accurate and reliable physical measurement of distresses including cracking, rutting, and ride, which can be used in conjunction with expert judgement. Automated surveys have the capability to provide less-biased and more-accurate results by collecting physical measurements and snapping high-resolution pictures or video logs using technologically advanced equipment. Consequently, it is possible to scan and rate a large network coverage in an efficient manner.

Pavement condition assessment requires extensive amount of data collection along with subjective expert judgement to detect cracks and irregularities in pavement segments. Hence, data quality is a key factor in any PMS as it impacts both the current and predicted condition ratings as well as the reliability of the decision-making processes to select appropriate maintenance and rehabilitation activities. To determine the severity and extent of deterioration, the pavement evaluation program focuses on the collection of data via manual, semi-automated, or fully automated surveys. In previous years, raters detected pavement cracks manually using the conventional windshield visual survey which is time-consuming and resource-intensive. To accelerate the data collection process, technologically advanced automated systems are employed to allow high-resolution crack detection at varying lighting conditions and vehicle

speeds (3, 4). State agencies combine raters' knowledge and experience with accuracy of distress measurements to influence the PCI.

Some studies compared PCI from automated data collection to visual surveys while others compared PCI among different vendors (5–7). However, the focus of this research study is to correlate subjective pavement condition ratings from windshield inspections to distress measurements from semi-automated processes by examining local Iowa network-level pavement condition data from Cedar Rapids in Linn County, Iowa. This effort will help transportation officials to understand and statistically justify which type of distresses are the driving factors behind the engineers' decisions while assigning an overall condition index to roadway segments. As a result, findings from the study can assist in the development of consistent ways to measure distresses using automated processes which are not subjective and can be easily replicated.

DATA COLLECTION

This section describes the data acquisition process for manual and automated pavement condition.

Manual Rating

The manual data used in this research was collected by the city engineer in Cedar Rapids over a period of several years. The data received from the city was rated in 2004, 2010, and 2013 by the same engineer. The entire network was not fully covered in any year but the covered segments provide a dataset that was representative of the entire network by traffic volume and pavement type.

The rating was based on the pavement surface evaluation and rating (PASER) methodology. The methodology involved a visual inspection of the streets to identify the types of surface distresses that exist and subjectively determine the principal severity levels and extents (8) based on the PASER Manual (9, 10). Following the visual inspection, the engineer estimated an overall condition index (OCI) value based a scale of 1 to 10, where 1 is failed and 10 is excellent, for the street. In some cases, the engineer also made instant recommendation of what street repairs needed to be done or how much longer before another treatment is required. This information was used to update the street segment (polyline) data in geographic information system (GIS) format.

Automated Data Collection

Prior to 2013, the Iowa Department of Transportation (DOT) funded data collection for local federal aid-eligible (FAE) paved roads for participating metropolitan planning organizations (MPOs) and regional planning affiliations (RPAs) in the state. Starting in 2013, as a result of the addition of some local roads to the National Highway System (NHS) because of Moving Ahead for Progress in the 21st Century (MAP-21) (11), distresses are being collected on every paved public road in the state on a 2-year cycle.

Pavement distress data is collected by a vendor in one direction on undivided two-lane highways and in both directions if there is a median. Data is collected every 16 m for the entire network. Each 16-m section is identified by street–route name, county, city, and latitude and longitude using a differential global positioning system (DGPS). Distresses collected include

transverse cracking, longitudinal cracking (wheelpath and non-wheelpath), alligator cracking, patching, rutting, and ride. The data collection is fully automated using an Automated Road Analyzer (ARAN) van. The ARAN collects and stores video images of the pavement surface that are digitized and processed using pattern recognition software to identify, quantify, and classify each distress (12).

Linking the Database

A critical step was necessary to be able to analyze the data. The street segment data from the manual collection and the raw data from the automated data collection were linked together by their spatial attributes to create one database.

Cleaning the Database

The resulting database was checked for completeness. Street segments with less than 100% coverage of the distress data were removed from the database. Since the data collection covered several years, there was a need to ensure that the raw data was assigned to values collected in the same year as the visual inspection. The resulting database was then summarized to the segment level. For rutting, the average over the entire segment was calculated to represent the street segment. For the other distresses, the sum of the individual points was calculated per tenth of a mile (528 ft). This was done to normalize the segments since the street segments were varying in length. The resulting summary data consisted of 2,389 total street segments.

METHODOLOGY AND RESULTS

Classification trees are generated to model the probabilistic path relating the OCI from visual inspection to the different variables from the semi-automated data collection method. The pavement condition subjectively assigned to each road segment by the city engineer through the visual–windshield inspection technique is categorized into five different levels using an ordinal scale for evaluation purposes. The new categorical scale ranged between one and five to describe the overall condition of the pavement is shown in Table 1.

Models for each pavement type, concrete–rigid (portland cement concrete) and composite–flexible (COM) pavements, are developed with categorical OCI as the response variable. The models excluded ride from the decision trees due to the fact that ride is not reliable on low-volume and low-speed roads. A standard quarter vehicle moving at a speed of approximately 50 mph is used to simulate roughness response which is correlated to IRI (13).

TABLE 1 Categorizing OCI into Different Levels

Windshield Inspection OCI	Ordinal Scale Assignment	Condition Description
$0 \leq \text{OCI} < 2$	1	Very Poor
$2 \leq \text{OCI} < 4$	2	Poor
$4 \leq \text{OCI} < 6$	3	Fair
$6 \leq \text{OCI} < 8$	4	Good
$8 \leq \text{OCI} \leq 10$	5	Excellent

Decision trees are generated for the rigid and flexible pavement types with 80% of the data used for training and the remaining 20% was used for validation. Interpreting results from decision trees can be complicated with larger splits. Researchers and statisticians overcome this limitation with the use of leaf reports (14). These are tabular format of the various probabilistic paths from the decision tree module which makes it practical to conduct an analysis from the split nodes. The optimal split for each classification tree is achieved when the difference between the training and validation R^2 is minimal and the nodal split was sufficient to explain the dataset.

For PCC pavement sections, the distress variables in the tree are explained by approximately 13% of the variation in the assignment of condition index based on visual inspection. Transverse cracking, annual average daily traffic (AADT), and joint spalling are the principal factors potentially impacting the decision of agencies while visually rating concrete pavements. An engineer will most likely classify the condition of rigid pavements as good or excellent if the average number of transverse cracks per tenth of a mile is less than 31. Whereas, higher number of transverse cracks and joint spalling along with higher traffic volume are correlated with lower visual OCI ratings.

Classification tree model for composite showed that assessments are primarily influenced by rutting, AADT, and transverse cracks. Only 12% of the visual inspection pavement condition rating are described by the variables in the tree. According to results, transportation officials most likely rated composite sections either good or excellent when average rutting is less than 0.18 in. Condition index of pavements deteriorated as rutting increased and the traffic volume is at least 930 vehicles per day.

CONCLUSION

The study examined local Iowa network-level pavement condition data from Cedar Rapids city in Linn County. Probabilistic path model using decision tree algorithm was used to relate OCI values from visual inspection to the distresses from the automated data collection for each segment, which will facilitate in finding consistent ways to measure distresses. In addition, results can help raters at local agencies to develop better guidance in order to precisely use the visual-windshield process to evaluate the condition of pavements. This is specifically beneficial in the case when local agencies lack the ability to have access to automated technology.

For rigid pavements transverse cracking is the major discriminator when performing visual inspection followed by traffic volume and joint spalling. The model devised for COM pavements, showed that rutting is a dominant distresses that impacts the visual OCI, followed by the AADT, and transverse cracking. AADT had an impact on the models for PCC and COM pavements, which can implicate that raters are willing to take more risk while assigning condition index to low-volume sections. Also, keeping in mind that raters are well-informed about the exposure level of the roadway system in their city, the various roadway functional classes should be rated differently to capture the full effect of the distresses and AADT. Moreover, the decision tree models showed weak correlations between the visual OCI and surface distresses. This reflects the subjectivity of the visual inspection methods, and possible inconsistency in the evaluation procedure.

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PAVEMENT EVALUATION

Knowledge Is Power *Managing Road Maintenance on a Remote Island*

NEIL BENNETT
Fulton Hogan

Fulton Hogan—an infrastructure consulting company with offices in New Zealand, Australia, and Fiji—won a contract to deliver maintenance services on the Chatham Islands, a remote location 500 mi (800 km) east of the South Island of New Zealand. Despite the remoteness, the project introduced simple, cost-effective performance-monitoring techniques.

This paper explains the use of a cell phone application, Roadroid, originally developed in Sweden to survey roughness on sealed roads and to monitor the level-of-service on all the island roads, the majority of which are unsealed.

The contract commenced in January 2016 and this paper presents the findings from 3 years research and condition monitoring and how the information can be used to make better management decisions.

CHATHAM ISLANDS

The Chatham Islands form an archipelago in the Pacific Ocean about 500 mi (800 km) east of the South Island of New Zealand (Figure 1). It consists of about 10 islands within a 25 mi (40 km) radius, the largest of which are Chatham Island and Pitt Island (Figure 2). Some of these islands, once cleared for farming, are now preserved as nature reserves to conserve some of the unique flora and fauna. The Chatham Islands Council is the smallest local authority in New Zealand, serving a population of about 640 people and a small tourism industry.

Almost all of the contract maintenance is on the main island of the Chatham Islands.

MAINTENANCE CONTRACT

In January 2016 Fulton Hogan commenced operations on the Chatham Islands to manage the road, water and wastewater operation and maintenance contract. The contract manager relocated from New Zealand to the Chathams to manage the contract with the support from the Wellington team who oversee the contract, and the wider Fulton Hogan team.

The contract work is to maintain and renew approximately 13 km of sealed and 166 km of unsealed local authority rural and urban roads and related infrastructure within the Chatham Islands Council area. Total expenditure averages \$NZ 4.2 million per annum and the financial assistance rate from the New Zealand government is currently 88%.

With the January 2016 start it was not possible to get the first construction season work completed before winter so 2 years' work was combined. A construction crew and sealing crew from both the North and South Islands were sent to the island to undertake the work on



FIGURE 1 Locality plan of Chatham Island.

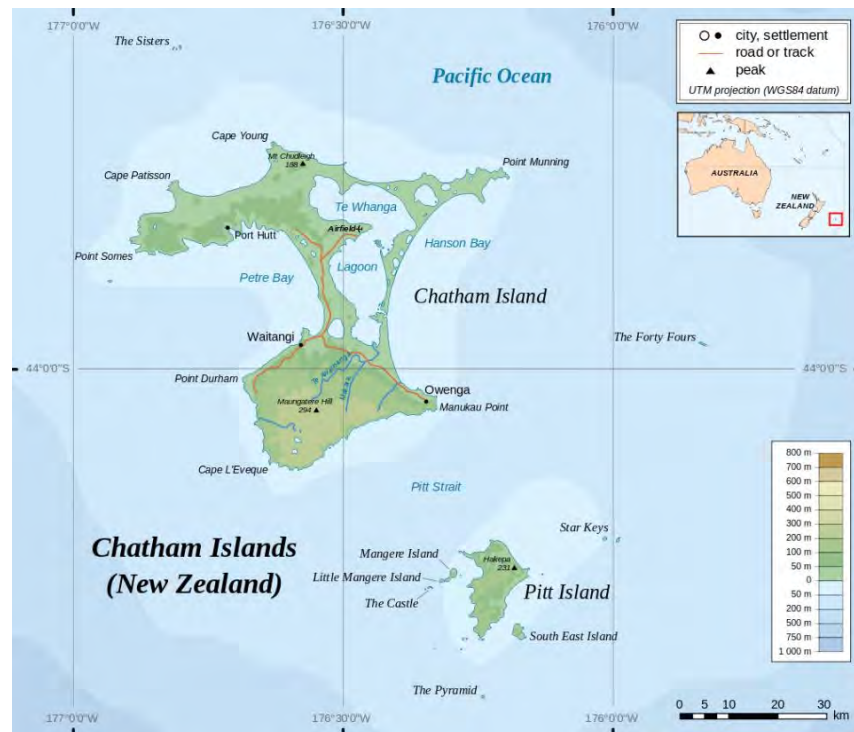


FIGURE 2 Map of Chatham Islands.

some badly neglected sections of road. The local team were involved through the whole construction project with operating plant, traffic control, and quality assurance.

The work is depicted in these before and after photos (Figure 3). In addition, an annual customer satisfaction survey was administered to gather feedback on the work.

The survey found a high increase in customer satisfaction which can likely be attributed to carrying out the work with specialist crews. A bitumen sprayer and a large roller were shipped



FIGURE 3 Photos of road reconstruction project on Chatham Island.

to the Chathams to undertake this work and returned to the mainland after the construction work was complete. The new methodology will be used to undertake rehabilitation projects every 3 years to get better economies of scale and therefore more work at a lower overall cost.

ROADROID

Roadroid, a cell phone application developed in Sweden, was introduced on many of our road maintenance contracts in New Zealand. One of the outputs from analysis of the phones motion sensors is an estimated International Roughness Index (IRI) score. Results are assessed at the World Banks Information Quality Level 3 (IQL3). While originally designed for sealed road networks, Fulton Hogan was keen to apply it to unsealed road networks where there were few cost-effective options for roughness measurement. Roadroid produced results with the following information available:

- Date and time stamp,
- GPS position,
- Speed,
- Altitude, and
- Roughness.

Implementation of Roadroid on the Chathams

It was decided to test the roads very soon after starting the contract to establish the baseline scores from which performance could be monitored. There is no cell phone service on the

Chathams and with the Internet being “slow” Fulton Hogan sends the Roadroid phone to the Chathams for each survey and it is returned to New Zealand to upload the data to the Roadroid cloud server for analysis. There is no ISQ1 survey completed on the Chathams so the cell phone is calibrated on a section of state highway in New Zealand in an identical vehicle and is retested when it is returned to New Zealand.

The new “Pro” version of Roadroid now being used enables either photos to be taken at set intervals or video. Due to the phones storage capacity and inability to upload work at the end of each day we have chosen to use photos taken every 3 s.

The survey photos are automatically uploaded to cloud-based service Mapillary, which is discussed further later in this paper.

Benefits of Roadroid

The information available from Roadroid surveys provides a number of benefits to assist with the management of an unsealed network.

1. Developing strategies (treatment changes and traction seals) around repeatedly poor sections to improve customer experiences and journeys;
2. Assessing and reporting on the level-of-service across the network;
3. Engaging with customers armed with robust data to demonstrate the network’s performance in comparison to their road, clearly portraying the complexities and scale of unsealed road management;
4. Repeatedly and reliably measuring the effectiveness of our grading strategy and unsealed treatments and material choices;
5. Targeting resources to poor performing areas and optimizing grading frequency, increasing value for money; and
6. Using the photo capture to identify failure modes and treat the causes of problems (shape correction and drainage improvements).

Survey Methodology

The Roadroid phone is mounted directly on to the windscreen of the survey vehicle; on the inside with the camera facing forward (Figure 4). This enables any vibrations in the vehicle to be felt directly by the cell phone sensors.

Surveys are completed in one direction, the same route is used each time, and the vehicle travels at approximately the same speed to remove some of the possible variations in results.

DATA ANALYSIS

Roughness surveys are loaded directly to the cloud-based server. Analysis can either be done directly on the server where a survey summary is created in four score bands from good to poor (Figure 5) or detailed information can be extracted to a comma-separated value (csv) file for analysis in other software. Examples of this analysis are shown below:



FIGURE 4 Roadroid phone as mounted for surveys.



FIGURE 5 Condition analysis calculated directly from Roadroid server.

Microsoft Excel can be used to plot the results from the data extracted from the server, and the chart below shows the results from the last five surveys on Airbase Road (Figure 6).

Aggregated readings can be viewed on the server (Figure 7) and each photo viewed on the screen (Figure 8). Figure 7 shows the high readings (circled) are the result of the transition off the bridge deck on Airbase Road where the cross fall changes and the area is potholed.

The most useful information has come from using all the road condition results (shown earlier in Figure 5) to produce trend line information over all the quarterly surveys since January 2016 for all roads. The trend line is plotted through the worst sections of road for each year and it shows a significant improvement in road condition. In February 2016, 91.5% of the network was in poor condition and in October 2018 there was only 29% poor (Figure 9). Airbase Road is 3.75 mi

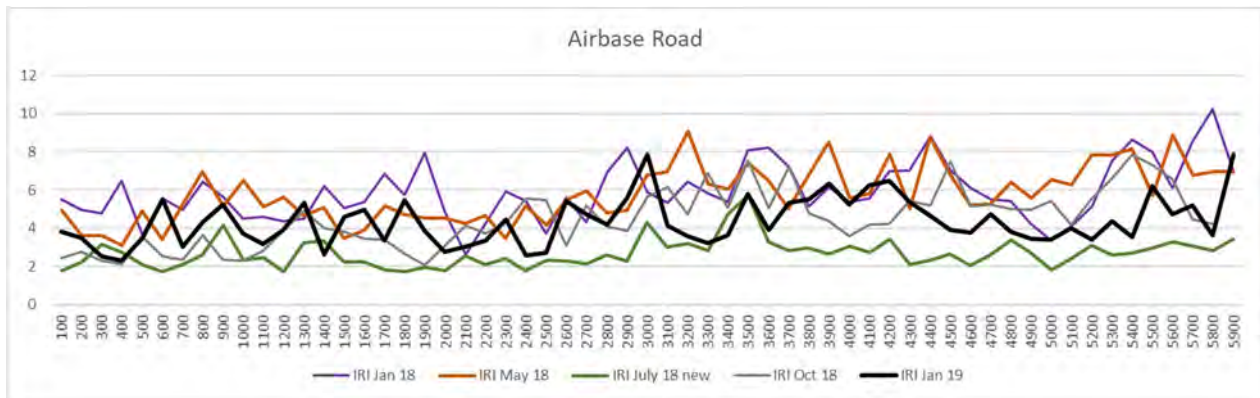


FIGURE 6 Linear plot of estimated IRI calculated in Excel.

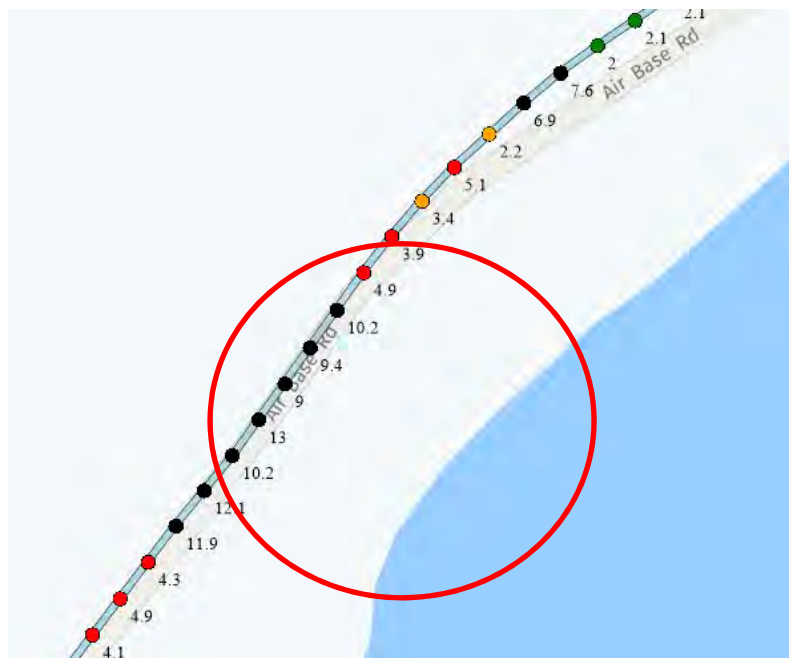


FIGURE 7 IRI scores viewed on Roadroid server near the bridge on Airbase Road.



FIGURE 8 Photo downloaded from the Roadroid server.

(6 km) long and is the road to the airport that has one flight per day in the summer. This road carries the highest volume of traffic on any of the unsealed roads on the island. Improvements made on the road include better-quality gravel and enhanced drainage.

The Chatham Islands are of course in the Southern Hemisphere so December and January are summer months. The roads are in their best condition (most green color) during each winter and worse in summer (Figure 10).

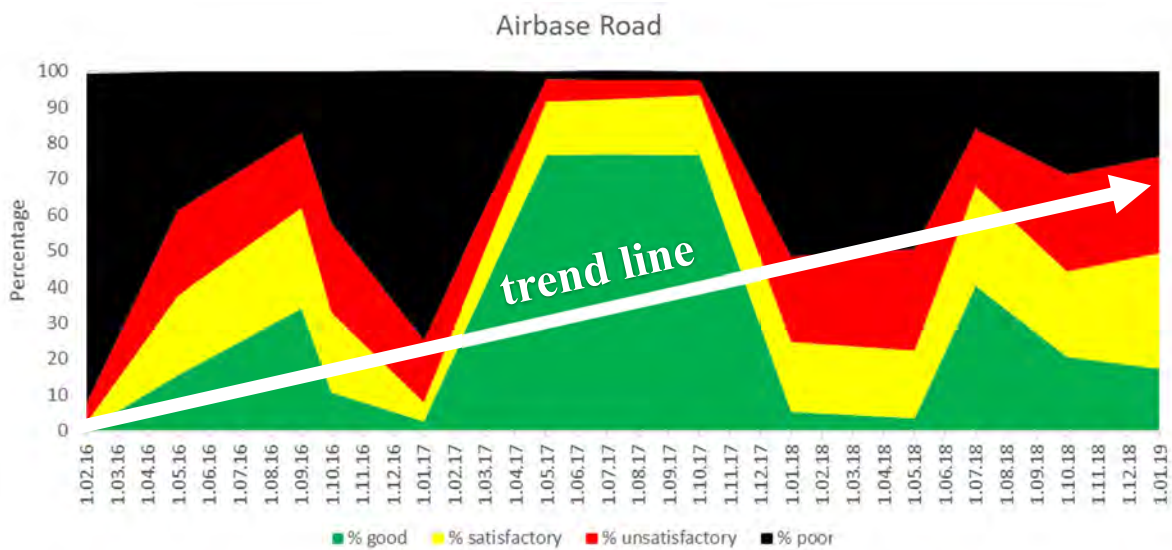


FIGURE 9 Condition trend for Airbase Road.

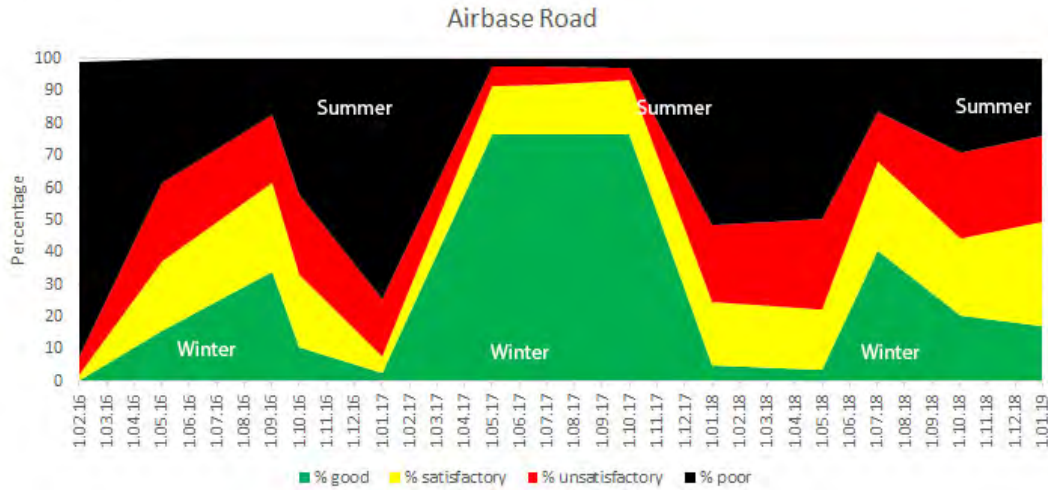


FIGURE 10 Seasonal variation in pavement condition.

This information is well known to practitioners. Roads should not be graded as frequently in summer when they are very dry as there is not enough moisture to bind the surface together again after the grading. A loose surface is more exposed to the environment allowing wind to remove the fine particles from the road as dust, creating a hazard for road users and removing the material that is needed to bind the surface.

Further investigation of the reason for the lower level of good results in the winter and spring of 2018 is shown in the graph of monthly rainfall in the main settlement on the island (Figure 11). The rainfall in August and September 2018 was 140 mm, half of that recorded in 2017 for the same 2 months.

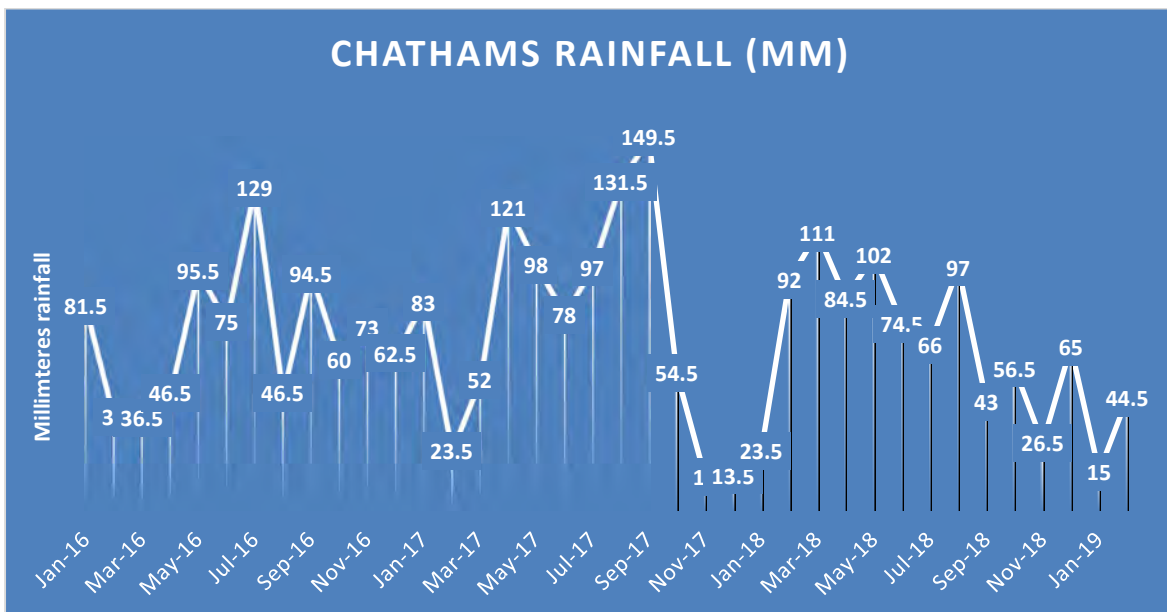


FIGURE 11 Rainfall on Chatham Islands.

MAPILLARY

As indicated earlier in this paper, during each survey photos are taken on the cell phone every 3 s. As an additional service Roadroid uploads each photo to a free public website called Mapillary (www.mapillary.com) which is a global network of contributors who want to make the world accessible to everyone by visualizing the world and building better maps. Anyone can join and collect street-level images, using simple tools like smart phones or action cameras.

The technology behind this joins the photos together, removes any images that could identify individuals and makes them freely available to the public. A large proportion of Europe and the United States has been surveyed but not so much in developing countries (Figure 12).

When the map is expanded to enable individual roads to be seen the road can be clicked on the open the video view (Figure 13).

A high-quality video is produced from the individual photos (Figure 14).

Mapillary is of huge benefit to a remote island such as the Chathams. Fulton Hogan's senior contract and asset management support is based in New Zealand, as is the Chatham Islands Council engineering consultant. Any issue on the road network such as the drainage problem shown in Figure 14 can now be viewed without an expensive flight to make decisions or delaying decisions until the next programmed visit. The consultant and contract staff can use the video to determine the extent of the repair. The consultant requested that the Roadroid surveys and photos be done at time where sunstrike could be avoided, as the photos and video were not as useful when the camera was facing the sun during the surveys.

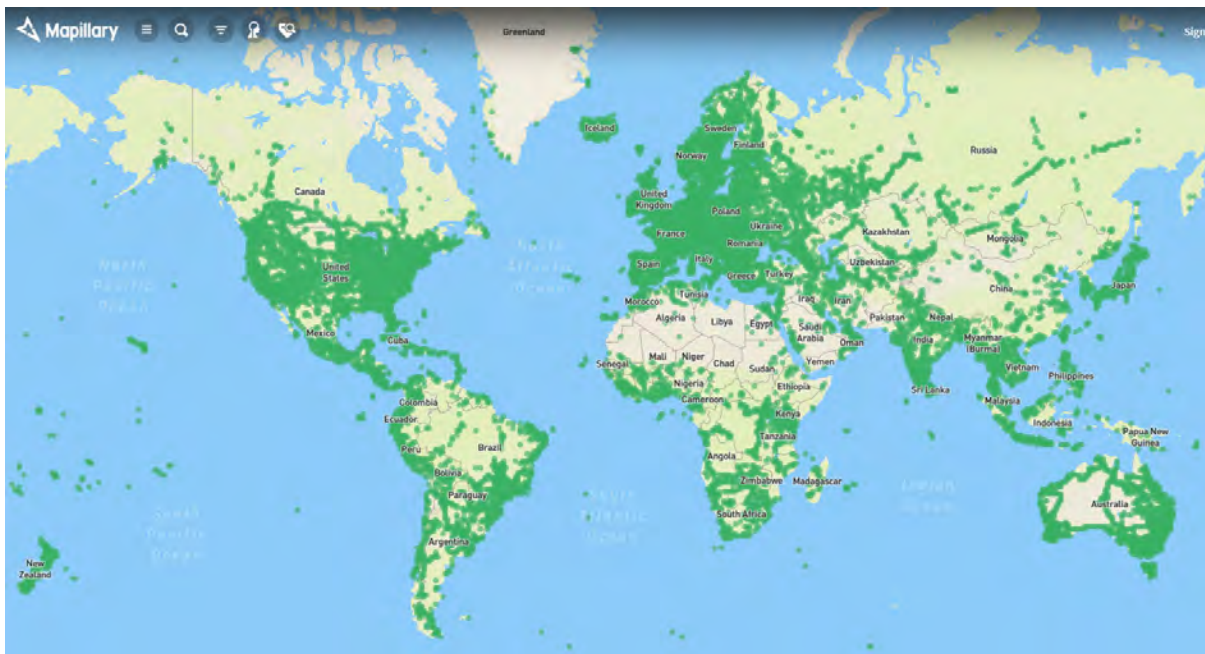


FIGURE 12 Mapillary video locations worldwide.

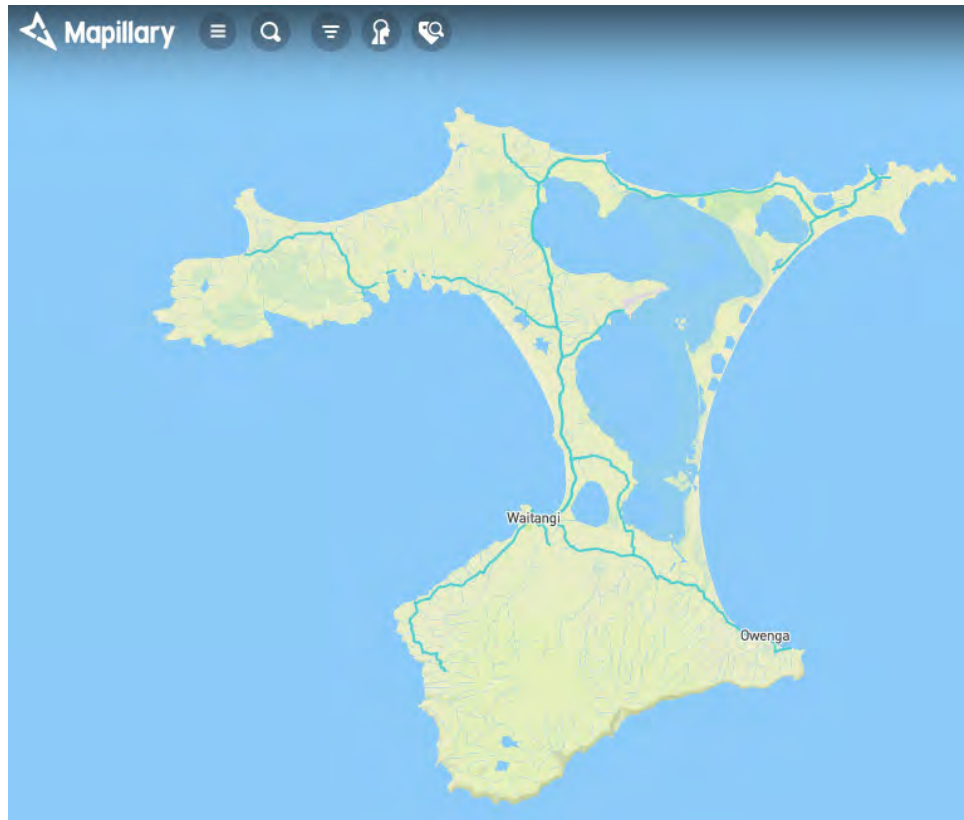


FIGURE 13 Mapillary video locations on the Chatham Islands.

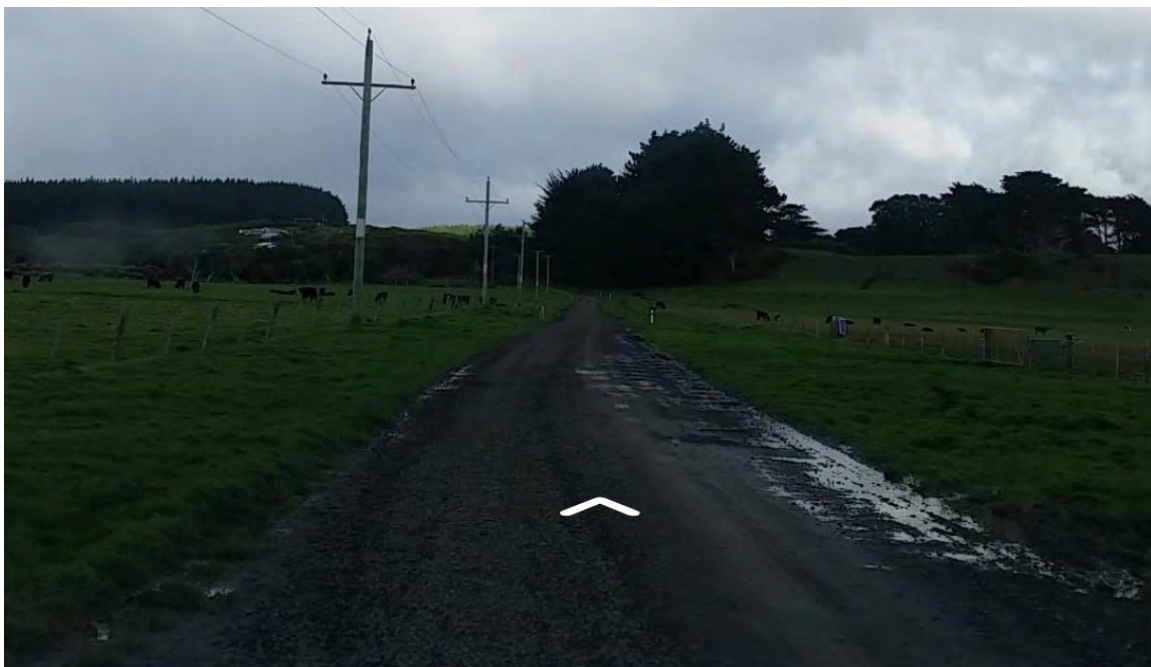


FIGURE 14 Mapillary video screenshot of Roadroid survey.

CONCLUSION

It was a contract requirement to monitor the condition of the network but the introduction of state-of-the-art technology and the ability to integrate the results with other technology platforms has made a significant difference to the road maintenance contract on the remote islands. The cost of the data capture and analysis quarterly is far outweighed by the savings from the smart engineering decisions that now can be made on the network.

Work to-date includes improved drainage and new sources of gravel for maintaining the unsealed roads. The improved level-of-service now being delivered on the network is at a level where the consultant has indicated that it is now about maintaining this level-of-service for the road users rather than trying to improve it further. The focus for the limited available funding will be to continue the positive drainage improvements on the network, slowly working our way around the islands main routes.

**ReCAP:
Research for Community
Access Partnership**

Transport Sector Leadership Development Programme
Establishment of Centre for Sub-Saharan Transport Leadership

L. R. SAMPSON

Independent Consultant (formerly ReCAP)

N. V. LETA

Research for Community Access Partnership

K. KELLY

MOTT MACDONALD

J. MALOTA

Association of Southern Africa National Road Agencies

INTRODUCTION

This paper outlines the requirements to establish a Centre for Sub-Saharan Transport Leadership (CSSTL), whose role would be to develop and implement the sustainable delivery options for future transport leadership capacity building. The paper will further provide updates on the status of the setting up of the CSSTL and implementation of the Transport Sector Leadership Development Programme (TSLDP).

It has long been accepted that transport is a critical supporter and driver of a country's economy. However, as highlighted by the World Bank (Ali et al., 2015), Africa, and specifically sub-Saharan Africa, remains the least-connected and under-developed region in the world in terms of transport provision. This remains a major challenge that current and future leaders of the transport sector continue to face.

In line with an approach by the Association of Southern Africa National Road Agencies (ASANRA) and confirmation from member countries of the Research for Community Access Partnership (ReCAP), an urgent need to build the future transport leadership core—not just in ASANRA member countries but throughout the whole of sub-Saharan Africa—was identified.

It is clear that one important gap exists in current development efforts: the availability of a holistic human capacity-building program to facilitate the training and mentorship of future leaders to meet the enormous responsibility and challenges they will face.

METHODOLOGY

A scoping study for a TSLDP was commissioned as part of the ReCAP program, funded by the U.K. Department for International Development (DFID) through UKAid. The study was undertaken by a consortium of Mott MacDonald and the University of Cape Town. Based on the findings of the scoping study and further deliberations by the ReCAP member countries, ASANRA and the ReCAP Program Management Unit, the program has now moved into an implementation phase.

A needs assessment was undertaken to identify training gaps in technical, managerial, and leadership skills for transport professionals in SSA. The assessment was carried out in the 12 ReCAP¹ countries and included the following:

- A literature review of existing policy documents on capacity building in the SSA transport sector;
- Online surveys of experts drawn from the civil engineering and transportation sectors;
- The identification of curricular requirements for the ideal leader;
- A gap analysis; and
- A survey of relevant academic curricula in Africa.

Three levels of delivery options have been identified to form part of the TSLDP:

- Mentorship programs, where candidates receive on-the-job training and mentoring focusing on leadership and management in the transport sector;
- Continuing Professional Development (CPD) courses (existing and new) that encapsulate the leadership, managerial, and technical content identified for leading transport professionals in Africa; and
- A recognized post-graduate qualification in transport leadership. This option offers best theoretical learning outcomes, combined with mentoring for practical on-the-job learning.

All training established under this program will be a blend of theoretical and practical application. Technical development in transport services and engineering content will mostly be taught theoretically. Leadership development is a more blended topic. The principles of people management shall be taught theoretically, but it is not until the candidate applies and practices these principles that the skills develop. For this reason, the leadership modules will be oriented around class-based learning followed by apply tasks which the candidates should undertake within their work environment.

A module plan for the post-graduate degree is provided in Figure 1. A detailed curriculum for these modules has also been developed and can be presented at the conference. The CPD program follows a similar module plan, but does not require the completion of a dissertation.

FINDINGS

The broad needs of the transport sector were identified from the literature review and problems to be addressed may be summarized as follows:

- Poor transport network connectivity and state of infrastructure;
- Insufficient human and institutional capacity;
- Insufficient funding;
- Lack of mentorship and training for mid-level engineers;
- Disconnect between the industry and academia;
- Limited opportunities for CPD;
- Insufficient regional interaction;

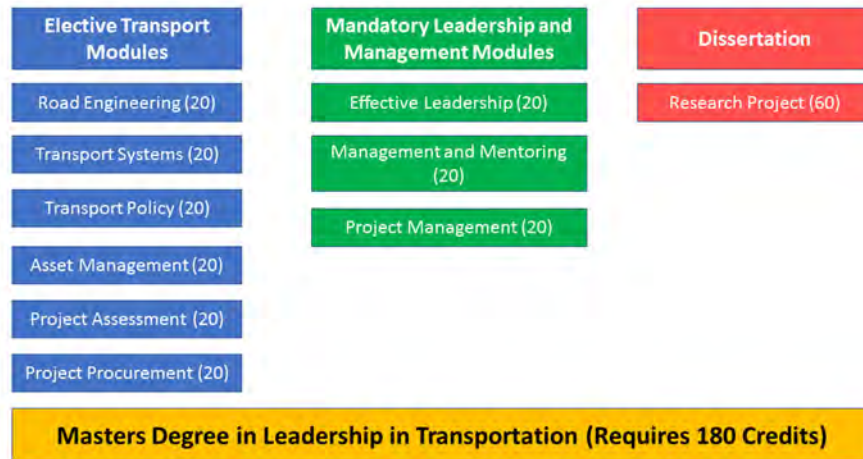


FIGURE 1 Post-graduate degree curriculum overview.

- Lack of support of transport-related policies and programs by key stakeholders outside the transport sector;
- Lack of a full appreciation for the connection between transport infrastructure design and service planning; and
- Lack of awareness and consideration of gender balance and diversity challenges in the industry.

In addition to the needs highlighted from the literature review, the online survey of 71 transport professionals revealed that there is a specific need for a capacity-building program focused on leadership and management. The program should also provide a solid ground in areas where there are obvious deficiencies and which are critical to the effective management of the sector in SSA, such as transport policy and planning, transport systems analysis, transport operations planning, land use planning, and road safety.

Building on the findings of the needs assessment and gap analysis, the project defined format options, a delivery method and program structure to achieve the defined learning objectives. The format the TSLDP takes is key to ensuring candidate and wider stakeholder participation, and ultimately, the long-term, sustainable success of the program. While TSLDP content will cater for both the technical and managerial/leadership needs of the candidates, it needs to

- Be accessible and achievable for all candidates across SSA;
- Provide clear guidelines for addressing gender balance and diversity as part of the key selection criteria for candidates; and
- Strike a balance between achieving the required learning goals and creating something that is too complex, in terms of content, or onerous, in terms of time inputs, as this will have an adverse effect on uptake.

Figure 2 presents an outlook of the curriculum content of programs in transport in the 15 SSA countries surveyed. This has been broken down to a planning related component and the engineering–technical design components, with a score for each component based on a

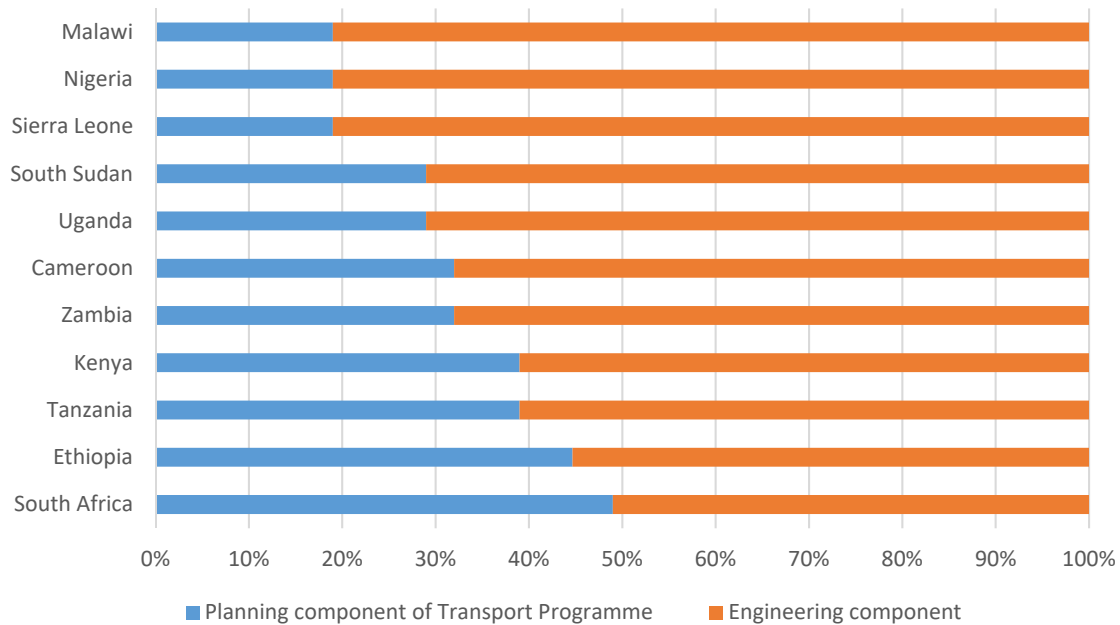


FIGURE 2 Country comparison of transport curriculum content.

summation of programs offered at both the undergraduate and post-graduate levels of study. The planning component relates to areas such as transport demand–supply analyses and forecasting; travel demand management; multimodal systems planning; land use planning; and service and operations planning. The engineering component, relates to aspects focusing on design of physical infrastructure, such as highway engineering and railway engineering.

From Figure 2, for the majority of countries, more attention is given to the engineering design aspect of transport infrastructure over planning-focused components. The transport curriculum in institutions in the East African countries, such as Ethiopia, Tanzania, and Kenya, as well as South Africa, tend to be more balanced, in terms of these two components, when compared with the other countries.

CONCLUSION

As part of the initial scoping study, key areas, from initial needs assessment and gap analysis, through to defining options for the program, their structure and a delivery mechanism were undertaken. From this, conclusions have been drawn and a way forward developed as part of the establishment of a CSSTL. This delivery mechanism is the way forward that has been adopted by ReCAP in association with a DFID sister program for high-volume transport to the next phase of establishment. This would involve development of a CSSTL establishment/business plan and offers a sustainable means of developing the TSLDP into the future.

The CSSTL is seen as key to the long-term success of TSLDP. This option involves funding one or two staff in a centre which should be setup within an existing institution in sub-Saharan Africa. The centre would be responsible for further developing and implementing the identified training options and be associated with reputable international and African tertiary

institutions. The CSSTL will have a business model for delivering the TSLDP that is cost-effective, implementable and manageable on a self-sustaining basis.

The TSLDP aims to inspire the next generation of young engineers, to take up leadership roles to shape future policy and development of the transport sector in sub-Saharan Africa.

NEXT STEPS

The contract for the establishment of the CSSTL has been awarded and the inception phase commenced on December 3, 2018. It is expected that significant progress will be achieved by the time of the 12th Low-Volume Roads (LVR) Conference in Montana in September 2019. Updates to that effect will be presented at the ReCAP LVR Special Session as part of the conference program.

Monitoring of progress and success will commence once the host institution is selected. It will be measured by two sets of key performance indicators (KPI) the team has established. These measure short- and long-term goals that will be used to monitor progress and evaluate whether the program is having its desired impact.

The first set of KPIs was developed to monitor the performance of the CSSTL and will be updated as the project develops. These KPIs cover areas such as

1. The links the CSSTL has developed to other institutions that can assist it with delivery and accreditation of its training program;
2. The links the CSSTL has developed to funding bodies who can support its ongoing operations and provide funding or scholarships for students to participate on the program;
3. The extent to which it has secured funding for its ongoing operations over the coming years; and
4. Marketing aspects, such as number of journal papers and articles published and attendance or presentations at conferences.

For the training program, the KPIs monitor the following areas:

1. Student numbers and successful completion rates;
2. Whether candidates are progressing into senior leadership roles (long-term monitoring required);
3. Numbers of female candidates on the program; and
4. Assessment of candidate feedback on the quality and content of the program, which will then be used to revise and update the program where required.

ACKNOWLEDGMENT

The authors acknowledge the support of the U.K. Government (UKAid), which is funding the project through the ReCAP program.

NOTE

1. ReCAP comprises 12 countries in Africa under the Africa Community Access Partnership (AfCAP): DRC, Ethiopia, Ghana, Kenya, Liberia, Malawi, Mozambique, Sierra Leone, South Sudan, Tanzania, Uganda, and Zambia; and five countries in Asia under the Asia Community Access Partnership (AsCAP): Afghanistan, Bangladesh, Myanmar, Nepal, and Pakistan.

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Establishment and Growth of a Road and Transport Research Centre

The Case of Uganda

MARK HENRY RUBARENZYA
Uganda National Roads Authority

INTRODUCTION

Transport infrastructure is vital for the economic development of Uganda. As part of institutional strengthening of the road sector, the Uganda Government created the Uganda National Road Authority (UNRA) which is responsible for the management, operation development, and maintenance of the country's national road network. The Uganda National Roads Authority Act was passed by Parliament in 2006. By statutory instrument, in December 2006, UNRA became a legal entity. UNRA is responsible for the development and management of national roads of which approximately 3,500 km are paved and 17,500 km are gravel or earth roads. Positive and tangible improvements in road development and management were recorded since UNRA's creation including the following:

- Development of good internal policies in corporate and technical management;
- Technical and management systems established that support various functions;
- Improvements in network planning and implementation activities; and
- Improvement in the road asset management function and tools.

However, by 2015 specific processes and functions remained weak, including research, development, and innovation. UNRA embarked upon a transformation journey in 2015 to improve its performance and become more responsive to stakeholders' needs. As part of this transformation, UNRA's Road and Transport Research Centre (RTRC) or Department of Research and Department (the Department) was created to strengthen and direct the organisation's research, development, and innovation (RDI) activities. It became operational in January 2016, with the appointment of a head of the Department to lead and champion research in UNRA.

This paper discusses the process of establishing and strengthening the Department as a RTRC in UNRA, the rationale behind it, and its key initiatives from creation to date. The successes and challenges provide valuable insights for similar initiatives, particularly in Africa and parts of Asia.

ESTABLISHING AND STRENGTHENING OF UNRA'S RTRC

The steps in establishing and strengthening of UNRA's RTRC are outlined below. Their implementation has not been undertaken sequentially, but rather in an agile manner.

Obtain an Institutional Mandate

The Department's mandate arose from UNRA's Strategic Plan (2013–2014/2017–2018) which highlighted the need for research, innovation; and UNRA's current Corporate Strategic Plan (2017–2018/2022–2023) which clearly identifies RDI as a key strategic initiative towards the effective execution of the organization strategy as well as supporting the continuous development of the organization and the roads and transport sector.

The Department's mandate was strengthened by embedding the broad research strategic initiatives within UNRA's Corporate Strategic Plan (2017–2018/2022–2023). This was done to mainstream the initiatives, and to ensure that the organization provides resources to implement these initiatives.

Clarify the Department's Role and Strategic Focus

The Department's role was broadly defined to include providing evidence-based advice on ways to construct and maintain roads that last their lifetime; developing knowledge for evidence-based decision-making on efficient and effective management of the national roads network and other services provided by UNRA; and promoting the continuous improvement of the services provided by UNRA.

Recruit and Equip a Core Team of Researchers

The Department was established without any preassigned resources including staff, financing, a laboratory, processes and procedures, and knowledge or training facilities. Recruitment of key staff was undertaken in consultation with the Department's leadership, who provided input on desired researcher competence. This has enabled the Department to selectively recruit staff who are aligned to short-term and medium-term research needs.

The Department pays researchers up to three times more than equivalent professionals in the local private sector. Staff regularly participate in regional meetings and workshops where they get to develop their competence. They also receive mentoring from experienced international researchers with whom we are collaborating on ongoing research projects.

Baseline Study to Support Planning and Strategy Development

To support research planning and strategy development, a baseline survey was undertaken of past and current road and transport research in Uganda (1). The survey also aimed to establish an online databank that enables access to such research. The first phase was completed in 2017. The second phase is yet to commence pending identification of financing.

The baseline survey provided information on international best practices, and how the Department could apply these practices. It also made proposals on research quality and researcher competence evaluation. The survey also highlighted inadequate funding for research in the country, and weak research linkages between government, industry, and academia. It noted that similar weaknesses are observed in other parts of the world (2).

Identify Core Research Focus Areas

The process of managing road and transport research needed to be focused on ensuring that appropriate and cost-effective research was conducted to support the strategic focus of the organization. Appropriate areas of research, where the envisaged research outcomes would address UNRA's current and expected challenges, were determined through a participatory process. Ten areas were agreed upon: Traffic, Material, Environment, Pavement Structure, Design, Construction, Management, Maintenance and Rehabilitation, Operations, and Road Safety.

To date research projects have been undertaken in the areas of Materials, Design, Management, Maintenance and Rehabilitation, Road Safety, and application of emerging technologies (including satellite imagery and mobile apps).

Establishment of Strategic Partnerships

Establishment of strategic partnerships is undertaken to bridge resource gaps and leverage missing competencies and best practice. It is a continuous process and follows a systematic process, and the Department dedicates financial resources to support collaborative research initiatives with its partners.

Domestically, the Department championed the formation of a National Research Steering Committee composed of representatives from academia, the National Council of Science and Technology, and the government Ministries and agencies responsible for roads and transport.

Internationally, the Department has established external partnerships with leading research institutions in Africa, Europe, and Asia. It has signed memoranda of understanding with the Centre for Scientific and Industrial Research (CSIR), and with Research for Community Access Partnership (ReCAP). It has ongoing research projects with the University of Birmingham, is a contributor to the Sustainable Mobility for All Initiative (SUM4All), and is active on committees of the Transportation Research Board (TRB).

Develop Documentation to Support Research Management

The Department has developed guidelines to support effective research management. The RDI policy is soon to be approved. Internal guidelines prepared to date include

- Guidelines for the assessment and adoption of new soil improvement products for road engineering purposes; and
- Guidelines for assessing the suitability and viability of surface sealants products

A document management system was developed and has been implemented.

STRENGTHENING OPERATIONS OF UNRA’S RTRC

Research Management and Process

The Department sought to identify an appropriate approach for managing research. Following a benchmarking exercise, the ‘Technology Tree’ (TT) model (3) developed for road research in South Africa, was preferred approach to managing the Department’s research.

Implementation of Research Process

The Department’s research needs are identified using a participatory process. This process recognizes that research is a continuous process, based on both the strategic focus areas and the evaluation of previous research and its effectiveness to drive the process for new research (FIGURE 1). The role of dissemination is provided for because research outputs only become transformational when they are disseminated to potential users who can incorporate the new knowledge into their daily activities.

Challenges and Opportunities

Sustainable Financing

Currently around 80% of ongoing research initiatives being undertaken by the Department rely on partial financing by external partners. This is a risk for sustainability in the event that these external partners should end their support. This risk is being mitigated by gradually increasing internal finance for research. The Baseline Study (1) recommended that the percentage of the national budget dedicated to research should be benchmarked and approved along international guidelines in terms of the percentage of its gross domestic product or the budget that a country needs to spend on research to be competitive. This would ensure sufficient financial resources are available for research.

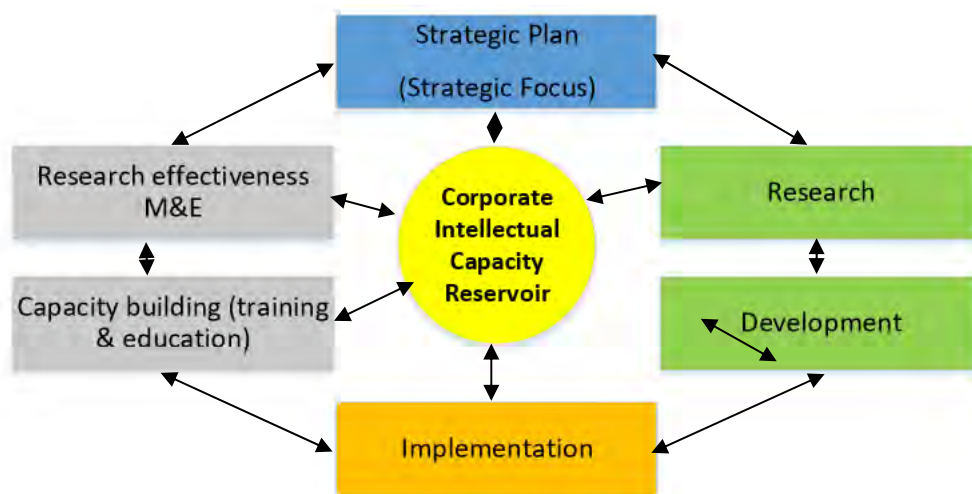


FIGURE 1 Conceptual outline of the Department’s research process.

Local Capacity and Resources

Like many other RTRCs in the region, the Department remains underresourced in terms of personnel and equipment. The Department would like to recruit more researchers but has struggled to do so in the local market.

Stakeholder Buy-In

While the need of research is generally accepted by all stakeholders, there is a reluctance across government and various partners to commit adequate resources to this activity. This may reflect low understanding and appreciation of the role and importance of research in achieving an organizations' strategic objectives. This is compounded by the fact that the outcomes and impact of research are seldom instantaneous unlike is the case with development or maintenance projects.

WAY FORWARD

Sustainability and relevance are considered key pillars to the success of the Department. They will be supported if identified research is conducted in such a way that the benefits of the research can be quantified and expressed in terms of the costs and larger investment and asset value of the network to demonstrate that a positive return on investment is obtained.

The next steps in establishing and strengthening of the Department include:

- To prepare 5- and 10-year business plans for the Department and
- To develop a skills enhancement plan in line with the 5- and 10-year business plans.

Going forward the Department's activities will be informed by a Research Framework and Business Plan, which is under development. This Framework and Business Plan will include short-, medium-, and long-term research initiatives, together with estimates of the resources required for their accomplishment.

CONCLUSION

In 2015 UNRA embarked on a transformation journey which included the creation of a Department of Research and Development which became operational in January 2016. A head of department was appointed to lead and champion research in UNRA. A stepwise process was followed during the process of establishing and strengthening of the Department. Noteworthy steps included clarification of the Departments role and strategic focus, systematic research planning and strategy development, resource acquisition and enhancement, identification of research focus areas, and establishment of strategic partners. To date research projects have been undertaken in the areas of Materials, Design, Management, Maintenance and rehabilitation, Road safety, and application of emerging technologies (including satellite imagery and mobile apps). These projects were identified through a participatory process. Some of the key challenges and opportunities facing the Department include the need for sustainable financing, recruitment of researchers in the local market and commitment of resources to fund research.

ACKNOWLEDGMENT

The work described in this article was supported by Uganda National Roads Authority and by the Research for Community Access Partnership (ReCAP) programme. ReCAP is funded by the UK Government's Department for International Development.

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Establishment of a Road and Transport Research Centre in Mozambique

MARCO VAZ DOS ANJOS

IRENE LANGA SIMÕES

National Road Administration, Mozambique

The National Roads Administration (ANE)—safeguarded by the Minister of Public Works, Housing, and Water Resources—is a public institution endowed with legal personality and administrative autonomy and whose main remit is to plan, build, and maintain roads. ANE is responsible for the management of classified road network while districts and municipalities are responsible for the management of the nonclassified road network.

With the support of Africa Community Access Program, Mozambique embarked on the challenge of establishing a Road Research Centre (RRC) aimed at providing the basis for improving the long-term capacity to undertake relevant, high-quality research supporting the roads of Mozambique.

The Council for Scientific and Industrial Research, in association with National Laboratory of Civil Engineering from Portugal, was engaged by Crown Agents to provide technical assistance to the project to address institutional issues relating to the RRC, to draft a strategic research plan, and to develop a 5-year business plan for the RRC.

This document addresses the rationale for the creation of the RRC; methods, approach, results, roles and responsibilities of the RRC department in ANE structure; organization structure of the RRC; research projects; completed and ongoing projects; training program; developing and sustaining networks and relationships; and identifying sustainability and risk to sustainability issues.

INTRODUCTION

The National Roads Administration (ANE) which is safeguarded by the Minister of Public Works, Housing and Water Resources, is a public institution endowed with legal personality and administrative autonomy, whose main remit is to plan, build, and maintain roads. ANE is responsible for the management of classified road network while districts and municipalities are responsible for the management of the nonclassified road network.

Due to its geographic location, Mozambique is prone to disasters as a result of climate changes and the country spends a huge amount of money annually repairing damages caused by flooding on the road network.

Thus, a need to improve knowledge and understanding of the performance of road construction material through research is identified as an important strategy to finding better solutions. Although the road sector started research activities in 2008 through construction of trial sections in a program called Rural Road Investment Program (RRIP) followed actually by monitoring of the performance these trial section, the country did not have capacity to undertake

the much-needed research, and it was decided to seek technical assistance to build the required research capacity in the country.

METHODS

With the support of Africa Community Access Program, Mozambique embarked on the challenge of establishing a RRC with the aim of providing the basis for improving the long-term capacity to undertake relevant, high-quality research supporting the road sector of Mozambique. The RRC will provide practical, innovative, and cost-effective engineering solutions for the design, construction, maintenance, and management of road infrastructure assets based on basic and applied research supporting the provision of a sustainable and cost-effective road network.

The Council for Scientific and Industrial Research, in association with National Laboratory of Civil Engineering from Portugal, was engaged by Crown Agents to provide technical assistance to the project to address institutional issues relating to the RRC, to draft a strategic research plan, and to develop a 5-year business plan for the RRC. Therefore, a 5-year business plan was developed in which are outlined strategic issues, including capacity building, establishment of laboratory capacity and procurement of equipment requirements, and building an information centre and other related infrastructure.

The following describes the approach undertaken to establish the RRC, building on the existing structures of ANE, established by the Decree No 23/2003 and was later revised under the Decree 12/2007. Figure 1 shows the location of the RRC within ANE structure.

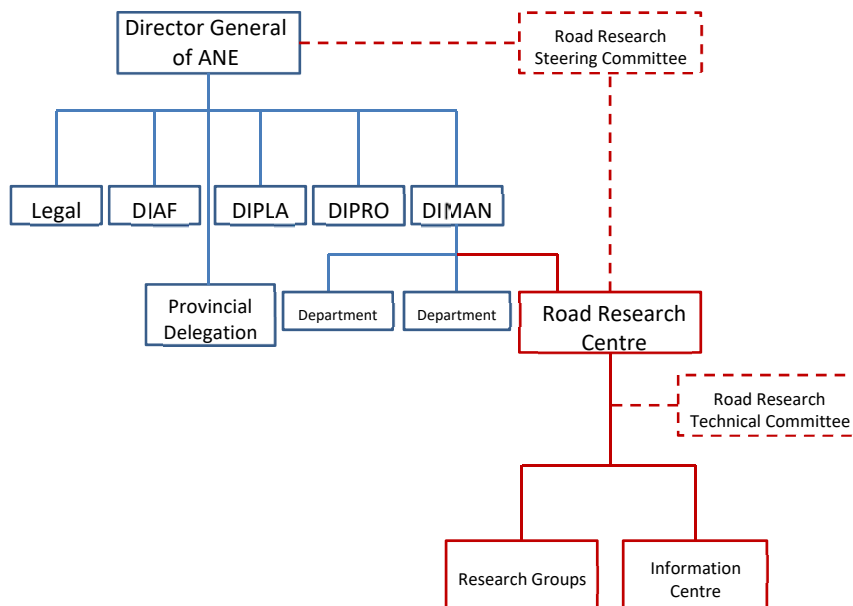


FIGURE 1 Institutional location of the RRC within ANE structure.

The organizational structure of the RRC will evolve over time, but currently has been established as an interim structure as shown Figure 2.

A Road Research Steering Committee (RRSC), which is expected to meet at least twice a year, has been constituted to provide overarching strategic oversight of the RRC. It is composed of senior representatives of the following organizations: ANE, Road Fund, Development Partners, Mozambican Engineers Council, Road Traffic and Safety Board, and Mozambican Engineering Laboratory.

The RRC is supported by three subgroups: a research group, an information centre, and laboratories. Initially, grouping all research engineers and scientists together in the research group would have the advantages of supporting shared learning, more research groups, and effective and efficient capacity building while also creating commonality of purpose among all researchers. As the RRC evolves over time, it may become necessary for the RRC to be further divided into two or more new research groups from the existing one, each addressing specific fields of study (e.g., materials and construction, pavement engineering, and structures).

The Road Research Technical Committee (RRTC) has been constituted to provide technical guidance and direction to the RRC and to advise the RRSC on the nature and scope of research, development, and implementation activities to be or being undertaken in the road infrastructure engineering domain.

ANE has a well-administered centralized document management section with five staff members responsible for scanning and indexing of official documents. A document management system is used to manage scanned copies of contracts, project reports, and correspondences. However, an electronic document management system is not yet available.

The information centre is yet to be established and will be expected to serve the information needs of RRC researchers, ANE, and road sector in Mozambique. It will also play a key role in knowledge and expertise transfer to road sector stakeholders and thereby increase the visibility and reputation of the RRC. And an information technician to manage and supervise the information centre will be hired.

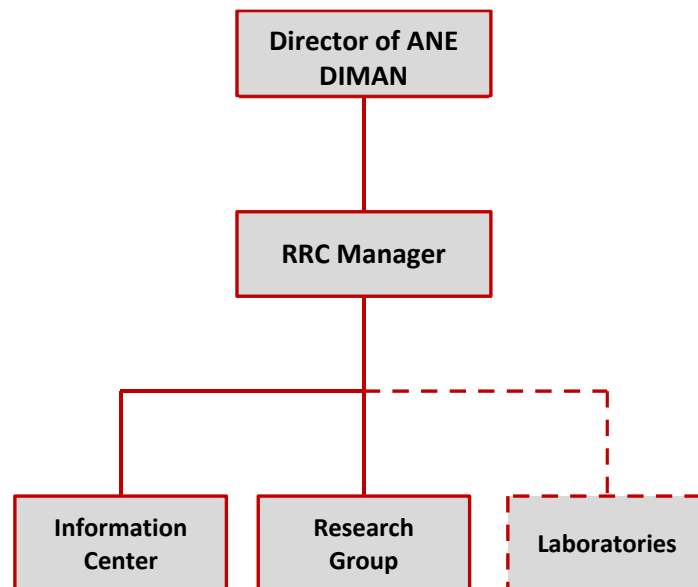


FIGURE 2 Organizational structure of RRC.

ANE's central laboratory is equipped with basic equipment for testing soils, aggregate, and asphalt. However, much of it needs updating, repair, or replacement. ANE has recently signed a contract with a company to supply laboratory equipment funded by the World Bank.

Mozambique Engineering Laboratory's (LEM's) facilities include testing equipment for bitumen, asphalt, cement, concrete, aggregates, and soils. The asphalt laboratory has recently been improved and it is equipped to undertake standard asphalt testing. The aggregates laboratory is still under improvement, but also can be used for standard testing. The soils laboratory is also under improvement and will be equipped for performing shear tests and triaxial testing as well as basic tests for soil characterization. LEM's cement and concrete laboratories are also operational.

RESULTS

While it is expected that the RRTC will meet at least three times a year, it has not yet been possible to fulfill this requirement. Usually the RRTC meets once a year. The last meeting was held in September this year and is expected to have the next meeting in April 2019.

In association with the RRTC, the RRC needs to prepare an annual Road Research Strategic Plan to direct its research and development operations and plan its deliverables. This plan needs to be endorsed by the RRSC before it is implemented.

Progress against the plan is expected to be reported at the meetings of the RRTC, and the plan needs to be updated after each meeting to incorporate recommendations made by the RRTC. Any significant changes made to the plan will be presented to the RRSC for final approval.

Research Projects

The RRTC identified 12 priority research projects. These 12 research priorities were categorized as follows:

- Six high-priority, “quick win” and/or “immediate need” projects that have the potential to be viewed as “breakthrough projects” on their completion, addressing the most pressing needs, yielding high impact, and demonstrating the value of the RRC to its stakeholders early in its existence.
- Six high-priority, longer-term projects with similar if not greater impact, but which will take longer to complete, although subcomponents of these projects will yield benefits within timeframes similar to those of the “quick win” projects.

Table 1 indicates the ongoing projects and their actual status.

Design of new and rehabilitated roads will be able to make use of identified materials which can provide the optimum design based on the best local materials available.

TABLE 1 Actual Status of Ongoing Projects

Project	Budget	Partner Contribution	Current Status	Final Output
Development of the Manual for Provision of Low Volume Roads	164,946GBP	6,770 GBP	Completed. Dissemination to be held in February 2019	Manual
LTPP Monitoring of Trial Sections	152,666GBP	146,227 GBP	Ongoing. 3 papers to presented at IRF conference in Durban last October.	Refined LTPP Monitoring Guidelines
Protocols for Improving Materials Testing Laboratories	85,335GBP	3,650 GBP	Ongoing	Standardised guidelines for Materials Testing
Climate Adaptation Study		5,032 GPB	Ongoing	Demonstration sections funded by World Bank
Research Capacity Building / DCP-DN Training	To be confirmed	To be confirmed	Concept Note to be developed	DCP-DN Trainers of Trainers
Materials Mapping and Development of a Road Construction Materials Database for Mozambique		To be confirmed	Conceptual Note under finalization	Materials Database

Completed and Ongoing Projects

Several experimental sections with different solutions for the base and sealing were constructed as part of the targeted interventions for low-volume roads in Mozambique under the RRIP. Construction took place at different places and at different periods.

Successfully implemented projects include the RRIP, the Back Analysis Project, and the development of the *Manual for Provision of Low-Volume Roads*. Projects being implemented include the long-term pavement performance monitoring of trial sections and the protocols for improving materials testing laboratories in Mozambique, at national level. The Climate Adaptation Project is underway and currently on the phase 2 and the demonstration section is being funded by the World Bank.

A number of projects committed for the future include knowledge management and training on DCP-DN.

Training Projects

For this purpose was developed a capacity building and skills development plan, which consists of strategic development of individual or collective knowledge, skills and productivity aimed at maximizing the potential for achieving the objectives of a given organization. Capacity building and establishment of such skills are vital in a context where intellectual capacity and expertise are the key element of knowledge-based institutions such as RRC, where appropriate skills are an essential element leveraging the impact of research and development.

Development of human capital and establishment of skills are essential for the creation of a solid and sustained basis of science, engineering, and technology (SET), which allows the achievement of the objectives by the RRC. The professional development of the research career, production of knowledge through learning and sharing culture, and creation of opportunities for young researchers are essential to sustaining a solid basis of SET. In order to achieve the goals, particularly with regard to human capital development, the RRC should focus essentially on the following aspects: (1) attracting, fostering, and establishing local skills for the execution of research work; (2) creating a specific research career for the RRC staff; (3) developing and implementing capacity building programs for skills development in research and developing a research culture; and (4) cooperation with universities, opening doors for the participation of students in the RRC research program.

The RRC as a knowledge-centred entity will have the responsibility to provide its employees with opportunities for continued and sustained learning and to provide them with all necessary support, in the form of time and incentives, in order to enable them to take advantage of these opportunities.

Traditional and formal learning opportunities (workshops, conferences, seminars)—although their importance continues to be significant—will not be enough to sustain a skilled workforce, which is centered on the knowledge. This way, the CPR should structure its work environment in order to provide enriching, continuous, and informal learning. In short, learning should be deeply rooted in work practices.

Development and Sustaining Networks and Relationships

ANE is carrying all operational costs of the RRC through its operational budget. ANE has other sources of income. By implication, the RRC would be able to attract funding from development partners such as the World Bank, U.K.'s Department for International Development, Swedish International Development Cooperation Agency, and Japan International Cooperation Agency to supplement its own funding. It is expected that in future the RRC will be able to source funding from other institutions, both public and private, for which research or training is undertaken.

Identified Sustainability and Risk to Sustainability Issues

Sustainability not only refers to allocation of budget but also to and more important human resources, infrastructure, and all necessary conditions so that the RRC can easily operate. Although there are difficulties at times, we are proud that RCC in Mozambique is receiving support.

All the projects implemented with different solutions, either in the base or as sealing option, are included in our programs, and these different solutions are implemented on Provincial

Road Programs and we hope to implement two more projects based on research capacity building and deepened training.

On the other hand, in order to influence the long-term research in Mozambique, universities, private companies (contractors and consultants), and industry are part of the technical committee of the RRC.

To ensure good research through demonstration and trials, we are creating small units of research in all 10 provinces, as well as promoting younger engineers.

CONCLUSION

Increase of human capital and establishment of skills are crucial for the support of a solid SET basis, allowing the RRC to fulfill all tasks for which it was mandated. Professional development of career research staff, production of knowledge through learning and sharing culture, and creation of opportunities for young researchers are essential for the development of the RRC human capital.

Knowledge of materials and the undertaking of research as a result of capacity building have improved through Research for Community Access Partnership. However, as there is no direct budget to be allocated to the RRC, it has been difficult to schedule activities. The RRC activities are actually financed under the annual budget in general activities. Going forward we will focus more on knowledge management and development of links with development partners working in Mozambique, such as World Bank and the European Union.

It is also expected to develop a guideline to provide guidance in conducting research in order to maintain the expected standards that the RRC is expected to meet as a research institution and to ensure that the RRC achieves highest quality and ethical standards in research, driven by integrity, honesty and professionalism of its staff. The guidelines are targeted to researchers and research support staff of RRC and any third parties that may be contracted by the RRC in cooperation with other research entities, internally and internationally through formal linkages.

Research Capacity Building Intervention in Rural Road Sector in Myanmar

SOE HTUN NAING

AKARE ZAW

NYEIN WAI PWINT

Ministry of Construction, Myanmar

NANDAR TUN

Yangon Technological University, Myanmar

INTRODUCTION

Myanmar has had to cope with major technical and financial challenges associated with the rural road sector. As a developing country, Myanmar is being challenged by the lack of adequate rural road infrastructures, requiring the rural population to travel long distances to access markets and essential services. It is estimated that 9.2 million people living in rural areas have no access to any form of road. Another 11.3 million people are connected by non-all-weather roads, significantly impacting on access and mobility during the wet season [Asian Development Bank (ADB), 2016]. As there is a strong relationship between the provision of rural access, and socioeconomic development and the improvement of quality of live in rural communities, a reliable rural road network is important for the country's socioeconomic development.

The establishment of the Research and Development Unit (RDU) was identified as a high priority for the Department of Rural Road Development (DRRD), which falls under the Ministry of Construction (MOC) and whose mandate is to systematically develop Myanmar's construction sector. The goal of the RDU is to serve the rural road engineering and transport needs of the public and private sector of Myanmar through the development, application, and dissemination of best practices and new knowledge, and the development of human capital.

As a newly established entity, there is currently no completed research program to report on, but there is already one ongoing project that was recently initiated. It is envisaged that several research projects will be activated once the RDU is fully established. The completion of these projects will benefit and significantly contribute to the sustainability of the rural road system in Myanmar.

METHODOLOGY

The strategy to establish capacity for road research has been supported by technical assistance provided by the Research for Community Access Partnership (ReCAP) program. ReCAP is a research program funded by UK Aid with the objective to promote safe and sustainable transport for rural communities in Asia and Africa. The capacity building program initiated in Myanmar, and more specifically within the RDU of DRRD, is aimed at providing scientific, engineering,

and technological leadership on rural road infrastructure and transport services through research, development and the implementation of research outcomes.

Once fully established and operational, the RDU will add value to Myanmar through the provision of

- A multidisciplinary skills and expertise base in rural road and transport engineering, which could include specific competences in geometric design; road design; performance evaluation; materials design; construction and maintenance methodologies; quality control; asset management; road safety; traffic management; and rural transport services;
- Core competences to support the development or updating of guidelines, norms and standards for rural roads;
- When the RDU material testing laboratory is established, access to research infrastructure;
- Access to several technology and software platforms that will provide technical support to the roads sector and support advanced research;
- An information resource center that will be accessible to all stakeholders; and
- Capabilities for developing solutions that will address rural road and transport-related problems in support of national priorities and contributing to socioeconomic development and public service delivery.

At present, the RDU has 11 staff members, with the potential to grow to at least 20 staff members in 5 years' time. The internal structure of the RDU is shown in Figure 1.

A Rural Road Research Technical Committee (RRRTC) was established in 2017 with the responsibility to identify and prioritize potential research areas. The current 10 prioritized research areas and projects recommended by the RRRTC include the following:

1. Rural road and bridge standards and development of a low-volume rural road design manual;



FIGURE 1 Internal structure of the RDU.

2. Asset management;
3. Road protection measures (drainage and slope protection);
4. Geometric design guidelines for rural roads;
5. Road surfacing trials;
6. Climate adaptation of rural road networks;
7. Best practice guidelines for the maintenance of rural roads;
8. Guidelines for the optimum utilization of local materials in rural roads;
9. Complimentary access infrastructure (i.e., footpaths, footbridges); and
10. Integration of road safety considerations into land use planning decisions and investment decisions on rural road infrastructure.

CURRENT ONGOING EXAMPLE PROJECT

Research Area 4—namely the monitoring of road surfacing trials on road TGI 1A, which is situated in Taung Gyi (Shan State)—is currently one of the ongoing projects. Two teams, with an appointed leader for each team (flexible pavements and rigid and semi-rigid pavements), have been formed for the assessment of structural and functional performance and data capturing, processing, and reporting. It is expected that new trial sections will be established and incorporated into World Bank and ADB implementation projects.

Civil Design Solutions (CDS) has been appointed as advisor for the TGI 1A trial section. CDS has provided site training to RDU staff on the establishment of monitoring trial sections and on how to conduct most of the standard measurements along project road TGI 1A, which was done during their initial visit in July 2018 and their second visit in October 2018 (CDS, 2018). Monitoring is carried out with the help of Myanmar's *Draft Guidelines for the Visual Assessment of Road Pavements*. These guidelines provide procedures for the visual assessment of road conditions and describe the various methods of assessment used. They provide detailed descriptions of the various distress types, the various degrees of distress and color photographs of typical examples of each distress type.

Project road TGI 1A was upgraded to a paved road standard in 2016 using four different surfacing types for research purposes: (a) penetration macadam; (b) double-chip seal (DBST); (c) nonreinforced concrete slabs; and (d) concrete block paving. All experimental sections were designed using Overseas Road Note 31. Activities undertaken on the four experimental sections include (1) measurement of surface roughness; (2) rutting measurements; (3) drainage factor measurements; and (4) visual condition assessments.

Following the first monitoring exercise, it was found that

1. The trial sections are performing reasonably well after 2 years in service.
2. The paved road width (4.5 m) is too narrow (leads to edge breaks and edge drops).
3. The concrete slabs were displaying cracks.
4. Some blocks of the concrete block paving had cracked.
5. Bleeding occurred in the DBST section.
6. The drainage system (side drains and culverts) was performing adequately despite the lack of maintenance of the drains.
7. The project road is probably not a low-volume road since the traffic using the road includes a wide range of vehicles other than motorcycles, up to heavily laden 6-axle trucks. Also,

trucks were seen to carry rock from the numerous quarries along the road as well as agricultural produce. Consequently, the volume of traffic is high for a rural road. A traffic count and axle load survey are required to verify whether the road falls into the definition of a low-volume road (i.e., carrying less than 1 million equivalent single-axle loads over its design life). In addition, some baseline data were collected by the RDU team with the help of the CDS team, and the CDS team discussed the requirements for future monitoring and data collection.

ACHIEVEMENTS

Successfully completed key activities included a review of the existing RDU business plan (Verhaeghe, 2018), assessment of RDU facilities, evaluation of job descriptions for all staff positions, and the implementation of skills development plan. The key performance indicators (KPIs) outlined in the strategic plan are taken into consideration to fulfill RDU's goals and operational objectives in the short to medium term. One of the KPIs was the recruitment of RDU staff. This was completed in November 2018.

A management plan for the trial sections has been established. It is essential that local road maintenance teams are aware of the locations of the trial sections and are responsible for ensuring that any activities to be undertaken are brought to the attention of the monitoring team immediately, to react to any actions affecting the trial sections, including maintenance activities.

Signboards have been installed, indicating the location and purpose of the trial sections, to provide useful information for local communities, and to discourage disruptions and vandalism.

CHALLENGES TO SUBSTAINING THE RDU

To ensure the sustainability of the capacity building efforts, the RDU will require funding with long-term guarantees to support its operations. There are two strategic options in order to ensure long-term sustainability of road research capacity: (1) a line item in the annual DRRD budget and (2) introducing a levy on construction–implementation projects that are directly funded by the MOC as well as by development partners.

The immediate constraints identified as barriers to an effective implementation of the capacity building and development of the RDU include the lack of (1) adequate research infrastructure, such as a comprehensive laboratory facility, (2) an operational information center for knowledge management and dissemination, (3) information and communication technology, and (4) vehicles for field studies. However, DRRD has road materials testing laboratories in Naypyitaw and Yangon that are temporarily being used by the RDU. A fully funded RDU laboratory development plan for Naypyitaw has been approved, and construction is expected to be completed in September 2019 with the facility becoming operational by December 2019.

Capacity building is an ongoing process for the institutional development of RDU. The Council of Scientific and Industrial Research and other research entities will continue to provide technical support as well as mentorship of staff to undertake road research. In addition, collaborative research will be undertaken with other local and international research entities through the establishment of formal linkages.

CONCLUSION

The objective of the article was to highlight the activities that have been undertaken by RDU since its inception. It is evident that RDU is going to contribute to the improvement of the Rural Road System in Myanmar. Capacity building is currently being prioritized but it has been possible to establish a long-term pavement plan monitoring program that will be undertaken systematically, and will also benefit the road sector in Myanmar through the review of guidelines and design manuals. The assessment of the baseline conditions, and evaluation of features most critical for the success of the different designs in the trial sections has and will continue to benefit the development of low-volume rural road standards and specifications for Myanmar. The draft final report of Rural Road Standard and Specification Manual in Myanmar was submitted in March 2018 (Dingen, 2018).

The above has been made possible through the support provided by the ReCAP program for rural road sector improvement. Constraints that may create barriers to the successful implementation of the capacity building initiative have been identified and measures have been put in place to mitigate their impact.

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Outcome of Research Capacity Building in Tanzania

FIKIRI FREDRICK MAGAFU

President's Office, Regional Administration and Local Government

VINCENT LWANDA

Tanzania Rural and Urban Roads Agency

INTRODUCTION

The constitution of Tanzania mandates that the President of the United Republic of Tanzania form ministries and government agencies (1). In 2016, the President issued a Ministerial Instrument that placed the coordination and monitoring of rural and urban roads under the President's Office, Regional Administration and Local Government (PO-RALG) (2).

In July 2017 PO-RALG established the Tanzania Rural and Urban Roads Agency (TARURA) under the Executive Agencies order with GN 211 of 2017 with a mandate to manage development and maintenance of rural and urban roads (3). The Construction Policy of Tanzania, among others, directs TARURA to achieve high-quality construction, cost reductions, use of local available materials, and use of special vulnerable groups in construction works (4). The Road Act mandates TARURA to do research to accommodate the requirements of the Construction Policy. TARURA is responsible for construction, rehabilitation, and maintenance of 127,489.23 km of roads of which 25,332.65 km are urban roads and 102,156.58 km are rural roads. Urban roads are located within cities, municipalities and town centres while rural roads are located within rural areas of villages.

METHODS

To this end, the African Community Access Programme (AfCAP) responded to the request from PO-RALG for support to establish a Road Research Centre. The Local Government Infrastructure and Transportation Research Centre (LoGITReC) was established as a research and quality-control unit and placed within TARURA to fulfill the Road Act of 2007 (Act No: 13 of 2007). The mandate of LoGITReC is given under the Act in Sub-Section 6.2, which states "the road authority shall undertake research or collaborate with any research organization with a view to facilitate the road authority's planning, development and maintenance activities" (5).

As a newly established road research entity, LoGITReC currently has three researchers supported by eight laboratory technicians and one laboratory manager. It is expected to evolve over time according to the Strategic Plan that was developed with the support of AfCAP. Technical assistance and support from AfCAP has assisted in the implementation of capacity building through skills development in human resources and in terms of establishment of laboratory management system to ensure the delivery of credible laboratory test results, establishing quality-assurance procedures and basic laboratory work inventory system, and development of general operational protocols. The organizational components of LoGITReC

consist of the Research Group, the Laboratory, and the Information Centre. Once fully functional LoGITReC will manage data and disseminate knowledge. The Information Centre is a bridge to research uptakes of LoGITReC and continues to receive technical and financial support from AfCAP, the Road Fund Board, TARURA, and PO-RALG.

Policy decision-making processes for LoGITReC are already in place. Formation of the LoGITReC Technical and Steering Committees is a mechanism for decision-making. Key stakeholders are provided with the opportunity to assist in its functions. These committees are required to meet twice every year so as to provide inputs for research-related issues to be forwarded for policy decision-making. However, for almost two consecutive years, the committees could not meet due to the transition which was unavoidable.

RESULTS

LoGITReC is implementing the 5-year Strategic Plan and several key performance indicators have been achieved as summarized below.

Staff Performance Measures

LoGITReC measures the performance of staff using an Open Performance Review and Appraisal System (OPRAS) which is a tool used in Tanzanian civil service to measure staff performance. In addition to OPRAS, LoGITReC measures the performance of researchers only on the basis of output as established in the Strategic Plan.

The position of a researcher on the proposed career ladder is based on a number of important input and output factors. A researcher's position on the career ladder will be established by determining the weighted sum of their level of competence in regard to both input and output factors (6).

Laboratory Facility

Currently the most active component of LoGITReC is the Laboratory. Several interventions have been conducted so far to ensure the Laboratory is working efficiently. A list of key performance indicators that have been achieved by LoGITReC with the support of AfCAP include acquisition and renovation of a building for the Laboratory, procurement of laboratory equipment, recruitment of laboratory staff, and capacity building and skills development for staff as well as implementation of actual testing works.

Research Capacity-Building Initiative

Establishment of LoGITReC is a result of research capacity building and various initiatives have been undertaken to build the capacity within LoGITReC through

- Purchase of laboratory equipment;
- Training on the proper use of laboratory equipment;
- Training on techniques for data collection, data entry, analysis, and management;
- Building “worklist software” workshop sample-handling procedures; and

- Training on procedures to monitor trial sections and special instruction on using and handling research equipment.

Management of LoGITReC

LoGITReC, with the support from AfCAP, held the laboratory manager to an ISO 17025 accredited research laboratory standard. The Council of Scientific and Industrial Research (CSIR) in South Africa equipped the manager with the learning experience required for a research laboratory and, thereafter, mentorship programs are required for researchers to equip them with research management skills.

Deficient of Physical Resources

There are two major physical challenges that face LoGITReC: insufficient laboratory equipment and limited number of staff. Lack of some equipment in the Laboratory limits the capacity to conduct important tests, hence creating a service delivery gap in the laboratory.

Road Research Capacity Building and National Funding

Road research capacity building in both PO-RALG and TARURA is conceptualized as a function of national research expenditure. This is approved by a special fund which is allocated for road research activities although not sufficient for the budget of a particular year. In addition, there has been a decrease in allocation of funds for this financial year 2018–2019. The budget for establishment of TARURA for the past 2 years explains the increase in demand of funds from the same constant source (Roads Fund Board). In 2016–2017, the funds allocated by PO-RALG for road research was TZS 500 million (about US\$250,000), that to TZS 750 million (about US\$375,000) in 2017–2018 and in 2018–2019 the fund was allocated by TARURA decreased to TZS 200 million (about USD 100,000). LoGITReC is confident that as TARURA gains stability, financial resources allocated for road research will also increase.

Strengthening Institutional Structure and Processes

To date, LoGITReC under TARURA has made little advancement as compared to how it was doing soon after its establishment. Most of what is going on today is what was inherited from PO-RALG with exception of the recruitment of one engineer who is on-contract basis. This is due to the ongoing transition from PO-RALG to TARURA.

Developing Sustainable Networks and Relationships

LoGITReC is a public entity which operates in a network of partnership with other road stakeholders. Key stakeholders from four sectors are the public, community, private, and academic sectors. LoGITReC successfully has made strong partnership with three stakeholders—the public, private, and academic sectors. The community sector is somehow a difficult but important partner to reach. The use of website, documentaries, posters, radio and television are being utilized to reach and help the community understand the results of research work.

There has been a strong partnership with the academic sector in the sense that LoGITReC has been involving the academic sector in various research-related projects it undertakes. The academia has been involved in the review of the Tanzania *Low-Volume Roads Manual*, training for Long-Term Pavement Performance monitoring of existing trial sections and implementation of regional guidelines for establishing and monitoring trial sections in Tanzania.

Internal Partnerships

LoGITReC should have strong links with other institutes that carry out research and laboratory testing within Tanzania. Cooperation with Tanzanian universities (e.g., University of Dar es Salaam, Mbeya University of Science and Technology, and Dar es Salaam Institute of Technology) is beneficial for LoGITReC. Some of the research activities may be supported by graduate students, which will contribute to the development of the students' skills in a work environment and in some cases, become a starting point for a research career. Reciprocal arrangements for LoGITReC staff to further their qualifications should also be negotiated.

External Partnership

Although not fully established, LoGITReC aims at building strong linkages with similar international organizations such as research institutes [e.g., ANE (National Road Administration), Mozambique; Australian Road Research Board; CSIR; Building and Road Research Institute, CSIR, Ghana; Ethiopia Roads Authority; Ministry of Health, Republic of Kenya; and Transport Research Laboratory, United Kingdom] and regional universities such as the Universities of Pretoria and Stellenbosch in South Africa, as well as international associations such as the World Road Association (PIARC).

The African Road and Transport Research Forum initiated in February 2015, a regional facility whose objectives are to promote research and innovation activities through networking, coordination, collaboration, knowledge transfer, and to provide advice on policies for sustainable development in Africa.

Strategies Towards Sustainability and Risks to Sustainability

Capacity-building programs that are sponsored by AfCAP through mentorship and coaching are fundamental to sustainability of LoGITReC. The training offered built internal research capacities to both LoGITReC and three higher learning institutions (University of Dar es Salaam, Dar Es Salaam Institute of Technology, and Mbeya University of Science and Technology). LoGITReC will continue to collaborate with these higher learning institutions to ensure sustainability.

The Government of Tanzania sets aside fund to support those programs. PIARC has also expressed an intention to support.

Benefits of Building Road Research Capacity for the Country

Various supports from AfCAP, Roads Fund Board, PO-RALG, and PIARC at different stages have assisted LoGITReC to purchase major laboratory equipment. In 2015, Tanzania had no laboratories for rural roads soils and aggregates testing services and only few Councils (about

0.5%) were able to access soil and aggregate testing facilities through Tanzania National Roads Agency (TANROADS).

As a result of the support, access to soils and aggregates testing facilities for rural roads projects increased to currently 52 projects that have consistently accessed LoGITReC laboratory as compared to no project before establishment of LoGITReC in 2015.

Furthermore, road surface conditions improved for roads having good and fair category from 56% in 2016 to 65% in 2018. This improvement is due to the services provided by LoGITReC that includes improvement in the quality-assurance procedures and development of human resources (7).

CONCLUSIONS

A number of initiatives have been undertaken in Tanzania as a result of the capacity building through AfCAP support. The road research capacity-building initiative has delivered benefits for the road sector in the country and continues to build the required capacity within LoGITReC.

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Comparative Analysis of the DCP-DN Design Approach Against Other Methods Used for Upgrading of Low-Volume Roads

Case of Uganda and Zambia

LEAN MUSENERO

Uganda National Roads Authority

JOSEPH CHIBWE

Road Development Agency, Zambia

Low-volume roads form a significant part of the road network in Sub-Saharan Africa (SSA). During the dry season, these often unsurfaced (predominantly gravel) roads generate a lot of dust which is a health hazard and has adverse effects on the environment. In the wet season, some sections of these roads become impassable, limiting accessibility and disrupting economic activities. Rapid depletion of gravel sources for road construction has rendered the re-graveling of these roads unsustainable. On the other hand, upgrading these roads to bituminous standard using conventional design approaches would be costly given the vastness of the network in question. For sustainability, it is imperative to explore alternative approaches for design of low-volume sealed roads (LVSR). Research in the region has highlighted the dynamic cone penetrometer–cone penetration rate (DCP-DN) method as one such plausible approach. In the DCP-DN design approach, the DN value is used directly, without correlation to the California bearing ratio (CBR). This paper provides a comparison of the DCP-DN pavement design method with other common methods for design of LVSR in SSA—particularly Uganda and Zambia. In both countries, the DCP-DN method was found to be a promising alternative for pavement design of LVSR in relation to potential reduction in cost of implementation of the pavement layers that resulted from the design.

To view this paper in its entirety, please visit:

https://journals.sagepub.com/topic/collections-trr/trr-1-2019_low_volume_road_conference/trr.

Appendix:
12th International Conference
on Low-Volume Roads
Preliminary Program



Preliminary Program

12th TRB

International Conference on Low Volume Roads



September 15 – 18, 2019
Hilton Garden Inn Kalispell, Montana

Sponsored by
Transportation Research Board

Co-Sponsored by
Forest Service, U.S. Department of Agriculture

Organized by

- Standing Committee on Low-Volume Roads (AFB30)
- Conference Subcommittee for the 12th International Conference on Low-Volume Roads (AFB30(1))

Local Host
Western Transportation Institute (WTI) at Montana State University (MSU)

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12th TRB International Conference on Low Volume Roads

Preliminary Program			
<i>as of May, 14, 2019</i>			
Sunday September 15, 2019			
Time	Session	Type	Session Title
8:00am-6:00pm	<i>Registration</i>		
9:00am-12:00pm	1	Morning Workshops	<ol style="list-style-type: none"> 1. Converting Distressed Paved Roads to Engineered Unpaved Roads 2. Pavement Preservation & Recycling Alliance Interactive Website Demo 3. Steps for Making Low Volume Roads More Climate Resilient
12:00pm-1:00pm	<i>Lunch Break (lunch on your own)</i>		
1:00pm-4:30pm	2	Afternoon Workshops	<ol style="list-style-type: none"> 4. Unstable Slope Management Program (USMP) for Low Volume Roads 5. Unmanned Aerial Vehicles (UAVs) in Low Volume Road Management 6. Low-Cost Safety Improvements for Low Volume Roads
5:30pm-7:30pm	<i>Exhibits Open and Welcome Reception</i>		

Monday September 16, 2019		
Time	Session	Session Title
7:30am-5:00pm	<i>Registration and Exhibits Open</i>	
8:30am-10:00am	3	Opening Plenary Session <ul style="list-style-type: none"> • Welcome • Road Management Challenges in the Urban/Rural Interface • LTAP at the National and Local Level • Low volume roads role in disaster recovery and network resilience, a case study
10:00am-10:30am	<i>Break</i>	
10:30am-12:00pm	4	Road Surfacing 1
	5	Safety on Low Volume Roads
	6	Geotechnology
12:00pm -1:30pm	<i>Lunch</i>	
1:30pm-3:00pm	7	Cold Regions & Climate Change
	8	Pavement Management 1
	9	Road Surfacing 2
3:00pm-3:30pm	<i>Break</i>	
3:30pm-5:00pm	10	Planning and Economics of Low Volume Roads
	11	Unpaved Roads
	12	Geosynthetics in Low Volume Roads
6:00pm-7:30pm	<i>Dinner on your own</i>	

Tuesday September 17, 2019		
Time	Session	Session Title
7:30am-12:00pm	<i>Registration and Exhibits Open</i>	
8:30am-10:00am	13	Drainage & Stream Crossings
	14	Climate Change Resiliency
	15	Low Volume Roads – States Pooled Fund 1
10:00am-10:30am	<i>Break</i>	
10:30pm-12:00pm	16	Stabilization
	17	Pavement Management 2
	18	Low Volume Roads – States Pooled Fund 2
12:00pm-1:00pm	<i>Lunch</i>	
1:00-5:30pm	Conference Field Trip	
6:00pm- 8:30pm	<i>Conference Banquet</i>	

Wednesday September 18, 2019		
Time	Session	Session Title
7:30am-9:00am	<i>Registration</i>	
8:30am-10:00am	19	Safety
	20	Pavement Design and Materials
	21	Education for LVR Engineers
10:00am-10:30am	<i>Break</i>	
10:30pm-12:00pm	22	Pavement Evaluation
	23	Invited – (TBD)
	24	ReCAP: Research for Community Access Partnership
12:00pm-1:30pm	<i>Lunch</i>	
1:30pm-3:00pm	25	Mini Workshops on Unpaved Road Apps
	26	Mini Workshops on Pavement Management System Apps
	27	Mini Workshop on Road Safety Apps
3:00pm-3:30pm	<i>Break</i>	
3:30pm-5:00pm	28	Closing Plenary Session (Tentative) <ul style="list-style-type: none"> • A Look to the Future (wrapping up a career) • A Look to the Future (early career stage) • Resolutions and Closing Remarks

Thursday September 19, 2019	
Time	Session
8:30am-10:00am	LVR2019 Conference Committee Meeting (Members only)

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