13th International Conference on Low-Volume Roads

July 24–26, 2023
Cedar Rapids, Iowa

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13th International Conference on Low-Volume Roads

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Preface

This publication contains papers and abstracts presented at Transportation Research Board's (TRB) 13th International Conference on Low-Volume Roads, held July 24–26, 2023, in Cedar Rapids, Iowa.

The 13th International Conference on Low-Volume Roads provides a global forum to examine new technologies and new techniques in the planning, design, construction, operation, maintenance, and administration of low-volume roads. The Conference focuses on issues that impact all aspects of life throughout the world, including the first and last mile of the roads where trips by people, produce, and resources start and end.

The International Conference on Low-Volume Roads has been held every 4 years since 1975. In 2023, it is held in Cedar Rapids, Iowa, returning to this centrally located state that was the host for the 2nd International Low-Volume Roads Conference in 1979. Approximately 78% of the roadway mileage and 77% of the bridges in this state are owned by its counties. In addition, 75% of this county roadway network is unpaved and helps feed the world, Iowa being the largest producer of corn, soybeans, and pork in the United States.

The conference is a vibrant combination of workshops, technical sessions, exhibits, and a technical field trip. Experts, researchers, and practitioners from across the United States as well as from many countries around the world gathered to explore case studies, practical solutions, and emerging research related to common problems associated to all aspects of low-volume roads.

Attendance at the conference included close to 300 practitioners and researchers from several countries worldwide. Attendees represent federal, state, and local public governmental agencies; private contractors and consultants; universities and colleges; and nongovernmental organizations.

The views expressed in the papers and the abstracts contained in this publication are those of the authors and do not necessarily reflect the views of TRB, the National Academies of Sciences, Engineering and Medicine, the National Research Council, or the cosponsors of the conference. The manuscripts were subject to a formal TRB peer review process, having a minimum of three reviews.

NEW TECHNOLOGIES AND INNOVATIONS

Development, Construction, and Instrumentation of Pilot Freeze–Thaw Resistant Granular Roadways Test Cells

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Granular roads are among the most prone transportation infrastructure to the effects of freeze–thaw cycles causing severe damages such as frost heave, thaw weakening, frost boil, potholes, rutting, and cracking (*1*). These distresses reduce the service life of these roads and incur high maintenance costs. In addition, many departments of transportation (DOTs) and local road divisions enforce seasonal load restrictions during the thawing season, which leads to traffic delays and induces more traffic on diverted roads, reducing their service life.

Frost heave is induced by the freezing of both in situ pore water and migrated water. The frost action in soils requires three conditions: (1) frost-susceptible soils, (2) freezing temperature, and (3) free access to water (*2*). Limiting any of these three factors would mitigate the freeze–thaw damages. The moisture migration toward the ice lens during freezing action is essential for ice growth during the freeze–thaw process (*3*). Engineered water repellency (EWR) is a way to artificially create a water-repellent layer (hydrophobic) to restrict the transport of water toward the frost front (*4*). EWR can mitigate the damages (such as heave) caused by the frost action in the soil. Organosilane, a widely manufactured chemical, can be used as a water-repellent additive (*5*,*6*). Zydex TerraSil is a water-soluble, non-leachable, and reactive soil modifier that makes the soil hydrophobic by modifying the soil surface. SIL-ACT ATS-100 is a clear, premium-grade, penetrating silane. It is used by several DOTs as a waterproofing treatment for the bridge deck, highways, and parking garages.

The construction of a full-scale field test is one of the best ways to investigate the performance of any new technology (*7*). In the present study, the development and construction of pilot test cells of Organosilane-treated subgrade soils of granular roadways to mitigate the freeze–thaw damages are discussed. Four test cells (including control) with two different water-repellent chemicals were constructed in Keokuk County, Iowa. Two test cells using Zydex TerraSil were built, one with the goal of providing the best performance while minimizing freeze–thaw damages and the other with the goal of providing the best performance while taking into account the cost and the potential application in new road construction as well as maintenance of existing roadways. One test cell with the ATS-100 chemical was constructed. The test cells were extensively instrumented to monitor the volumetric water content, electrical conductivity, matric potential, temperature, frost heave–thaw settlement, groundwater fluctuations, and weather parameters throughout the year. Extensive pre- and post-construction field testing, including light weight deflectometer (LWD), dynamic cone penetrometer (DCP), nuclear density gauge (NDG), and light detection and ranging (LiDAR) were conducted. This extended abstract briefly discusses the design of pilot test cells, their construction, and instrumentation in the field.

Methodology

Four pilot test cells were constructed in the freeze–thaw prone region of Keokuk County, Iowa. The test cells were constructed after extensive laboratory testing involving soil index property testing, contact angle testing (CA), water drop penetration testing (WDPT), and freeze–thaw testing. The degree of hydrophobicity of soil was determined using the CA and WDPT test, and the results suggest that Organosilane-treated soil induces hydrophobicity into the soil. Also, the hydrophobicity increases with the increase in the concentration of Organosilane (*6*,*8*). Based on the laboratory performance of the treated soil, the two chemicals, namely Zydex TerraSil and ATS-100, were selected for field testing.

Reconnaissance

A reconnaissance of the proposed pilot test site was conducted. The soil samples were collected from different depths in barrels for laboratory testing; a 3-m excavation was conducted for determining the groundwater table depth (1.5 m) and to investigate the soil variation with depth. Laboratory testing, including index property testing, CA

testing, WDPT, and frost-heave testing were conducted on the soil. Digital photos were taken to document the condition of the site. Field-test measurements, including DCP and LWD tests, were carried out for a 150-m section at 30-min intervals.

Design and Construction of Test Cells

The length of each test cell was 91.5 m. Cell 1 was termed ATS as the chemical used in this cell was ATS-100. The chemical was sprayed at a concentration of 0.67 L/m^2. The concentration was carefully selected after the common application rate of 0.40 L/m^2 in the bridge deck application of ATS. The chemical was sprayed by using a hand-controlled 12-nozzle sprayer bar at a walking rate. The depth of the chemical-treated layer was 0.64 m from the ground surface level. Cell 1 was followed by cell 2 after a 12-m control spacing. Cell 2 was termed Zydex Research because of the large thickness of the treatment of the subgrade soil using the Organosilane chemical Zydex TerraSil. The treatment consisted of a 0.15-m treated remolded layer at a depth of 0.64 to 0.79 m. The concentration of remolded layer was 2.82 L/m^3. The remolded layer was capped with a treated sprayed layer above at a concentration of 0.11 L/m^2. The cell was planned to provide maximum expected performance by mitigating frost damage and increasing the life span of roadways. Cell 3 was the control cell. It was constructed from the bottom up for better quality control and similar design specifications to other cells. Finally, cell 4 was constructed following cell 3. Cell 4 was termed Zydex Practical. The treatment in this cell was optimized to provide the best performance with the minimum cost such that the designed treatment could be applicable for the future construction and maintenance of granular roadways. Two Zydex TerraSil sprayed layer at a depth of 0.25 m and 0.41 m with a chemical concentration of 0.07 L/m^2 and 0.11 L/m^2 was used. The configuration of the four test cells with treatment locations is given in Figure 1. The construction images with the chemical application, testing, and sensor installation are shown in Figure 2.

LWD, DCP, and NDG testing were carried out pre- and post-construction. LiDAR was also carried out pre- and post-construction. The data will help in analyzing the elevation changes of the roadway post-construction. Further LiDAR testing after the first winter will also be conducted to determine the changes in road profile after the first freeze–thaw cycle season to account for the heave in treated and untreated cells. Soil samples from the untreated and treated layer were collected for conducting laboratory testing to determine the performance of the treated layer. The field testing will help evaluate the performance of the untreated and treated cells by comparing the
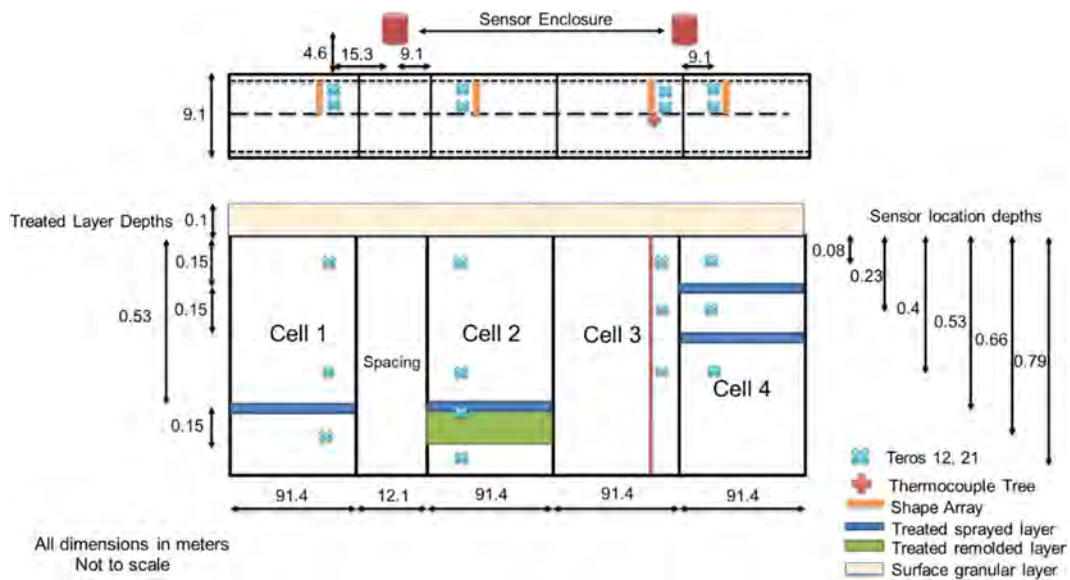
**FIGURE 1 The schematic diagram of the four test
cells with the location of treated layers and sensors.**
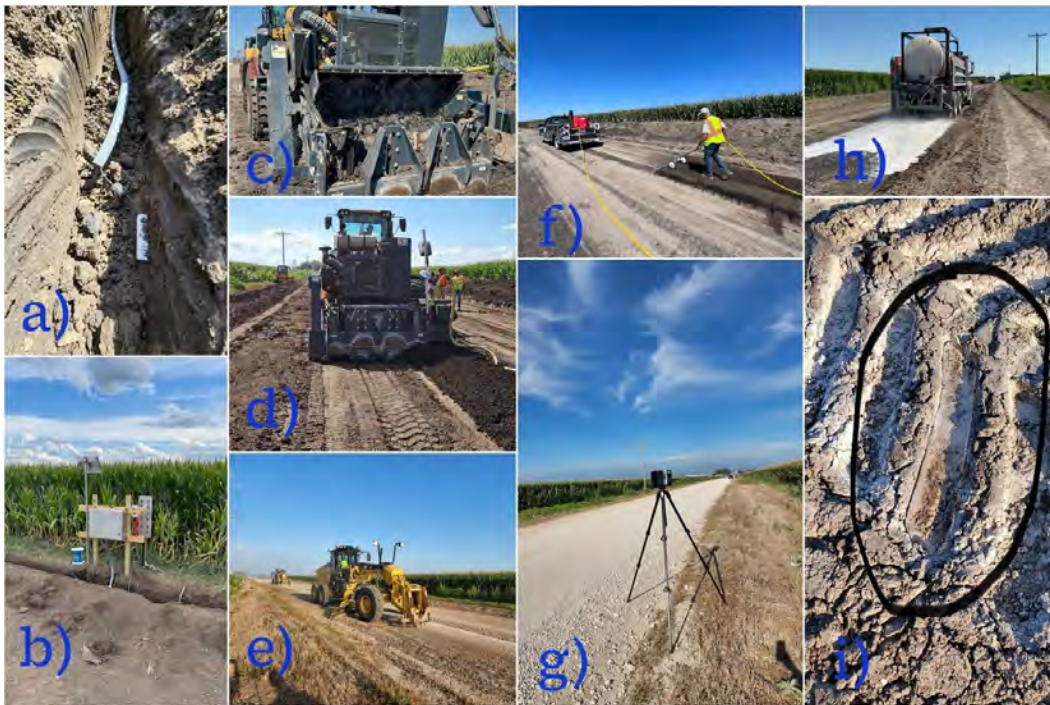
**FIGURE 2 Construction images: (*a*) sensor installation, (*b*) enclosure for data
acquisition system, (*c*) roadhog spray bar, (*d*), roadhog application for cell 2, (*e*)
motor grader in operation, (*f*) ATS-100 spray application, (*g*) LiDAR testing, (*h*) Zydex
TerraSil spray application, and (*i*) water beading up in Zydex TerraSil treated layer.**

engineering properties and frost heave and thaw settlement of the cells before and after the freeze–thaw cycles.

Instrumentation

All the cells were extensively instrumented to measure the volumetric water content, electrical conductivity, matric potential, temperature, and frost heave–thaw settlement. The volumetric water content, electrical conductivity, and temperature of soils at different depths were measured using the Teros 12 sensors from the Meter Group. The Teros 21 (Meter Group) sensors were installed to measure the matric potential and temperature inside the soil. A thermocouple tree was built with 8 T-type thermocouples placed at regular intervals up to 1.5 m to measure the soil temperature for determining the frost penetration depth during winter. Shape array accelerometer sensors were installed at 0.25 m from the surface to measure the frost heave and thaw settlement. Two monitoring wells up to 4.9 m in depth were installed to monitor the changes in the groundwater elevation with time. A weather station, ATMOS 41, was installed to monitor the air temperature, precipitation, wind speed, etc. The configuration of the four test cells with sensor locations is given in Figure 1.

Two enclosures were installed for the data acquisition system. Each enclosure unit consists of the CR1000X data logger, AM16/32B multiplexer, two SAA 232 for the interface between shape arrays and data logger, BP 24 battery, CH 150 charging regulator for the interface between solar panel and battery, RF 407 for radio frequency transmission. A 20-W solar panel was mounted on top of each enclosure for power requirements. A Cell 210 modem from Campbell was installed in one enclosure for remote data collection.

Findings

The pilot test cells were constructed in August 2022 and are continuously monitored. The performance of the treated layer was noticeable immediately after construction as the water poured over the treated layer was beading up and not infiltrating into the soil as shown in Figure 2*i*. The results were fascinating as the water-repellent layer was performing great, even at the locations with macro cracks. Because the soil moisture in the top layers, which is most vulnerable to frost action, will be limited as a result of capillary rise being controlled by the water-repellent layer, it is anticipated that the treated cells will decrease the effects of frost-heave damage. The authors anticipate a

considerable difference in the treated cells' moisture content compared to the control cells, with the treated cells having less moisture above the treated layer. The overall heave monitored using shape arrays, LiDAR, and visual observation is expected to be less in treated cells compared to untreated cell. The team is looking forward to presenting promising results after the freeze–thaw cycle season and expects the treated cells to perform better due to less frost action.

Conclusions

Any examination of new technology must be carefully planned and built, starting with the pilot test cells. Investigating the success or failure of new technology can be done with the use of careful instrumentation and monitoring. In this study, the optimum design of freeze–thaw resistant granular roadways was developed after extensive laboratory testing, including index property testing, CA testing, WDPT, freeze–thaw testing, and site reconnaissance. Zydex TerraSil and ATS-100 chemicals with optimum concentrations and treatment locations were selected for field testing in Keokuk County, Iowa. Different cells were designed to investigate the maximum performance or optimal performance considering the economical constraints. The cells were extensively instrumented to monitor various critical soil parameters (volumetric water content, matric suction, temperature) and environmental parameters (air temperature, precipitation, groundwater level, etc.). A large number of pre- and post-construction field tests were conducted. The Zydex-treated cells 2 and 4 showed promising results during construction as water started beading on the treated layer after a few hours of the treatment even on places with macro cracks. It implies that researchers can anticipate success in the future in reducing frost impact on roads due to limited capillary rise and thus extending their lifespan. The data collection began in the late fall of 2022.

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A Demonstration of Automated Driving System Performance on Rural Roadways

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While the technology behind automated driving systems (ADS) is rapidly advancing, nearly all testing has occurred in fair weather and dry road conditions. Also, data on automation performance are not publicly available from private industry. This project, ADS for Rural America, is meant to provide the first publicly available data set of ADS performance on rural roads—to show where automated vehicle technology works and, more importantly, where and why it doesn't work. Literature on this topic is not currently available.

This project also aims to show the benefits of ADS technologies for those not able to drive, such as aging populations and those with disabilities. Researchers are recruiting riders from local communities to ride along and provide their perceptions and trust levels of the technology.

Rural roads often have sharp grades, limited lines of sight, high differentials in road-user speed, variability in surface type, and poor or non-existent lane markings. Rural roads often have uncontrolled intersections with cross-traffic. In fact, farm equipment, animals, and other slow-moving road users can and do get on and off rural roadways at numerous unmarked locations. Extreme weather conditions, including rain and snow, exacerbate the challenges that an ADS would need to be able to recognize and respond to. For these reasons, this project's 47-mile project route includes a variety of road types, from interstates to gravel roads, that expose the ADS to unique challenges of rural roadways.

The automation software that is being utilized requires the use of a digital high-definition (HD) map. This map enables the centimeter-level accuracy that is needed by the automation to position the vehicle in a specific lane. The use of an HD map has the added benefit of enabling an ADS vehicle to drive under automation on unmarked or poorly marked roads.

This project aims to boost rural equity and represent the transportation needs of rural populations and low-volume roads. While 19% of Americans live in rural areas, nearly 50% of traffic fatalities occur on rural roads; there is a clear safety need in this area. If ADS technologies are going to realize their promise of greater safety on roadways, then they need to do so on all roadways, including rural roadways.

Methodology

This project comprises the six phases shown in Table 1. After each phase, the automation capability is enhanced to add functionality to the system and increase the percentage of the route that can be driven under automation. During each new phase, the project team is also assessing the automation's performance and using the data collected to inform improvements in successive phases.

For example, in Phase 1 the vehicle was driven under automation primarily on controlled access roadways (interstates) and stretches of rural highways (which was beyond the goals of Phase 1), and automation on on/off ramps was added in Phase 2. By Phase 3, the vehicle was able to drive under automation through the urban areas, read and respond to the color of traffic lights (via cameras), and navigate four-way stops. The starting location varies among the four planned stops along the route, each with distinct types of parking areas. As of the writing of this abstract, Phases 1 through 3 have been completed with 40 drives. By the end of Phase 6, 80 total drives will have been completed.

TABLE 1 Project Phases

Project Phase	Description	Drives Planned	Drives Completed	Date	Status
1	Controlled access roadways	10	10	11/2021	Complete
2	Highways and ramps	20	17	03/2022	Complete
3	Urban areas	10	13	07/2022	Complete
4	Unmarked roads	10	–	10/2022	Planning
5	V2X	10	–	01/2023	Planning
6	Parking areas/full route	20	–	05/2023	Planning
Total		**80**	**40**		

The types of data being collected include the following:

- Automation performance, documenting challenges;
- Mileage and percentage of each drive under automation, with maps of automation activation;
- Environment data, such as weather conditions, surface state, surface and air temperature, and wind speed and direction;
- Video data (exterior, front, and rear);
- Voluntary takeovers by the safety driver, including reason for takeover;
- Forced takeovers (when the automation disengages on its own or becomes unavailable, requiring the driver to intervene);
- Encounters with vulnerable road users (VRUs): number and types of VRUs, in automation versus manual mode; and
- Safety critical events.

Data regarding the riders include the following:

- Video data;
- Demographic information;
- Survey data regarding trust and acceptance of the technology (pre- and post-drive);
- Biometric data (heart rate variability and galvanic skin response); and
- Anxiety ratings at nine specific locations along the route.

Data on the safety driver include the following:

- Survey data regarding perceptions of the automation's performance;
- Biometric data; and
- Video data.

Findings

The purpose of this project is to provide a publicly available data set for others to analyze. However, trends and summary data are provided after each phase. Following are some overview numbers from Phases 1 through 3:

- 40 complete drives (6 partial);
- 2,152 total miles recorded;
- 1,518 miles recorded under automation (70%);
- 3,850 GB of data collected; and
- 92 participant riders.

The percentage of the route that has been driven in automation per phase on average has increased from 58% in Phase 1 to 93% in Phase 3. As the project continues, additional functionality will be introduced to the vehicle that will allow it to drive a larger portion in automation, but it will increase at a much slower rate as researchers will attempt to automate smaller sections of the route, including the low-volume roads (i.e., gravel roads) as well as the parking lots. Figure 1 shows how the percentage of miles driven in automation for the various federal function classification (FFC) of road types has increased for each phase.

Low-volume roads often mix unconventional traffic (e.g., farm machinery, bicycles, and pedestrians with highway passenger cars, buses, and trucks). As part of this project, all of the encounters with these types of VRUs have been flagged (Table 2). This offers researchers the opportunity to examine the vehicle data to understand how the automation handles interaction with these VRUs and compare that to interactions when the vehicle is being driven manually.
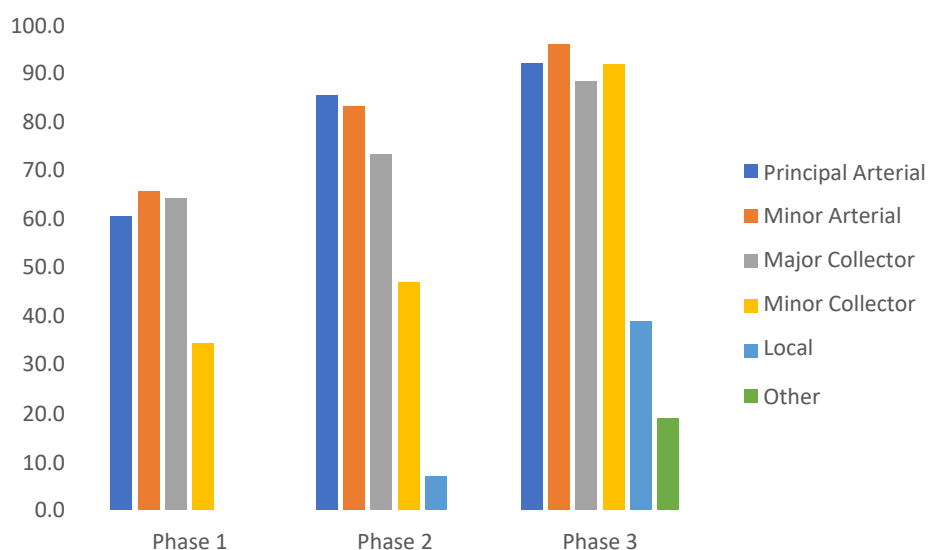
FIGURE 1 Percentage of miles completed in automation for the FFC road classification across phases.

TABLE 2 Vulnerable Road-User Encounters Under Different Modes of Operation Across Phases

VRUs	Automation	Manual
Horse and buggy	56	16
Pedestrian	52	70
Farm equipment	18	8
Police or emergency vehicles	2	2
Bicycle	24	7
Other	30	15
ATV	6	1
Animal	3	3
Object in roadway	6	0

There are many design challenges associated with low-volume roads, such as adequate width, sharp curves, blind hills, single-lane roads, and poor or inadequate pavement markings. Some potential solutions include inserting artificial speed zones into the HD map to reduce speed when travelling at the posted speed limit feels unsafe (i.e., blind hills and curves) or moving the virtual lanes on the HD map to deal with narrow or single-lane roads. As part of this project, researchers are examining all of these challenges and investigating the best way to deal with them to ensure that automated vehicles have the potential to work in all environments and on all types of roadways.

Conclusions

In general, our experience has shown that ADS technology is not ready for wide implementation. Testing in rural areas has revealed many shortcomings that need to be addressed before ADS vehicles can be driven or monitored by drivers with little or no training with these systems. Many challenges can be overcome by making changes to the HD map to help the automation contend with location-specific challenges. This includes altering path of travel on narrow roads or inserting virtual speed limit signs on roadways with sharp curves. However, this approach is not feasible for dealing with dynamic situations such as poor weather and construction. This is because the automation travels at the posted speed limit and on the defined path of travel (programmed into the HD map) irrespective of weather and roadway surface conditions.

With respect to gravel roads, the automation currently (as of Phase 3) follows the digital "lanes" of the HD map, and therefore would drive along the edge of the road, where it is not safe to drive due to sharp drop-offs and loose gravel. It also drives

where it is not safe to drive due to sharp drop-offs and loose gravel. It also drives without respect to actual roadway conditions. Phase 4 will focus on improving the automation behavior on gravel roads, which will require updating the lanes so that the vehicle travels closer to the gravel-road center. It will need to be programmed to respond accordingly when approaching another vehicle or blind corner/hill by moving the vehicle to the edge of the road to make room for an oncoming vehicle and then move back closer to the center of the road when it is safe to do so.

For practical purposes, the data are meant to show where improvements need to be made to the technology before ADS is ready to be safely and widely implemented. This publicly available data can be analyzed by anyone, including those within the industry, government, and academia. The results of the analysis can be used to inform policy and highlight shortcomings that need to be overcome by technology providers, including original equipment manufacturers. Additional rural testing and technology enhancements are needed for ADS to handle the unique characteristics of rural roadways.

A Preliminary Engineering Design of a Very Remote Project During a Global Pandemic

Use of Advanced Satellite Technology to Plan and Design Low-Volume Roads

ALEX CAMPBELL

AnyWay Solid Environmental Solutions

A primary Sustainable Development Goal (Goal 9) for Papua New Guinea is to develop quality, reliable, sustainable, and resilient infrastructure, including regional and trans-border infrastructure, to support economic development and human well-being, with a focus on affordable and equitable access for all (*1*, *2*). The current classified road network consists of 30,000 km, 8,740 km are classified as national roads and about 22,000 km as sub-national roads, with the majority of these being low-volume roads. Lack of funding and commitment to maintenance has contributed to the generally poor state of road conditions throughout the country. The effects of poor road conditions are high vehicle operating, traveling costs, and road accidents (*3*). Furthermore, many bridges have deteriorated, posing other traffic hazards for the traveling public and businesses (*4*). Further impacting mobility is the lack of a road route between the capital, Port Moresby, and the resource-rich Highlands.

The Trans Island Highway, connecting the National Capital District with the Highlands, has been a vision of the government for decades (*5*). At present, after 34 years of independence, there is still no road linking the agriculturally and minerally rich Highlands; the second largest urban center, the city of Lae in the Morobe Province; and Port Moresby in the National Capital District. In August 1979 the Department of Transport and Civil Aviation commissioned a report to look at possible routes connecting Lae and Port Moresby. This report looked at the socioeconomic and construction costs of nine possible routes from Port Moresby to Lae. The most significant study since then was conducted in 2007 to further review and identify appropriate alignments for the link between Port Moresby and Lae, of which five refined routes were proposed.

At present, there is considerable momentum in constructing the missing link. In 2019, prior to the global pandemic, the Department of Works commissioned a

preliminary engineering study and route alignment of two routes (Route 1 and Route 4) to complete the Trans National Highway. However, given the remoteness of Papua New Guinea and the necessity to travel through a major Australasian hub (e.g., Singapore, Hong Kong, Brisbane) to fly to Port Moresby, and the significant travel restrictions due to COVID-19, a methodology to conduct the preliminary route alignment study and associated engineering designs was sought. This extended abstract outlines the use of advanced satellite technology that was used to collect high-definition data as an example of how the technology can be used for other projects throughout the world to assess route alignments and how the technology will become more predominant in other facets of pavement projects (e.g., pavement distress surveys).

Methodology

AnyWay Solutions (AnyWay) was selected to identify the best route alignment over two routings (Route 1 = 179 km and Route 4 = 204 km) that included both upgrading an existing highly distressed gravel road to a sealed road, and optimization of the route through a mountain pass where no route currently existed. This project included a topographic survey, optimization of the alignments, preliminary design of drainage and slope stability structures, and a pavement design. However, given the international pandemic and restriction to travel, the ability to undertake the project using traditional techniques, including surveying and traditional in-country review, was considerably complicated. It simply wasn't possible to have a team in the field, so AnyWay had to look at incorporating state-of-the-art technology to help them facilitate the project. As such, advanced satellite technology to acquire high-definition data to execute the project alignment was employed. This technology is at the forefront in allowing for faster data acquisition and at higher definitions than traditional surveying techniques.

Satellite-Based Survey

This phase involved selecting and acquiring high-resolution (>1 m per pixel) geo-referenced stereo imagery with forward, nadir, and backward satellite images of the designated area. Each point on earth is covered by at least two, and in some cases three, satellite images taken at the same time from the same spot in space. The stereo acquisition mode enables the generation of digital elevation models (DEMs) using a stereo-matching technique, with the stereo satellite data. Using the stereo acquisition mode, multiple images from different angles (forward, close to nadir, backward) are

recorded for the same location at the same time with the same sensor, as illustrated in Figure 1.

Considerable efforts have been invested in the acquisition date and time of the satellite images so that they would be taken as close to noon as possible where the sun is at its highest. This is important to minimize shadowed areas, which could cause errors in the DEM generation, especially on a mountainous terrain. Collecting data when the sun is at or near its highest point also decreases the work required during data processing. Tropical areas are characterized by cloud cover all year round, which creates challenges. Even though some of the designated path was covered by clouds, each location on earth was visited many times by the imaging satellites. A significant effort was made to find and allocate imagery from different dates of acquisition to eliminate cloud cover completely.

Processing of Satellite Collected Imagery

This phase of the project involved stereo pairs matching, which includes making sure that all the area needed for mapping is covered, each of the satellite image pairs have a known epipolar geometry attribute, and accurate RPC files are provided with the images. Epipolar geometry is the geometry of stereo vision. When two cameras view a 3-D scene from two distinct positions, there are several geometric relations between the 3-D points and their projections onto the 2-D images that lead to constraints between
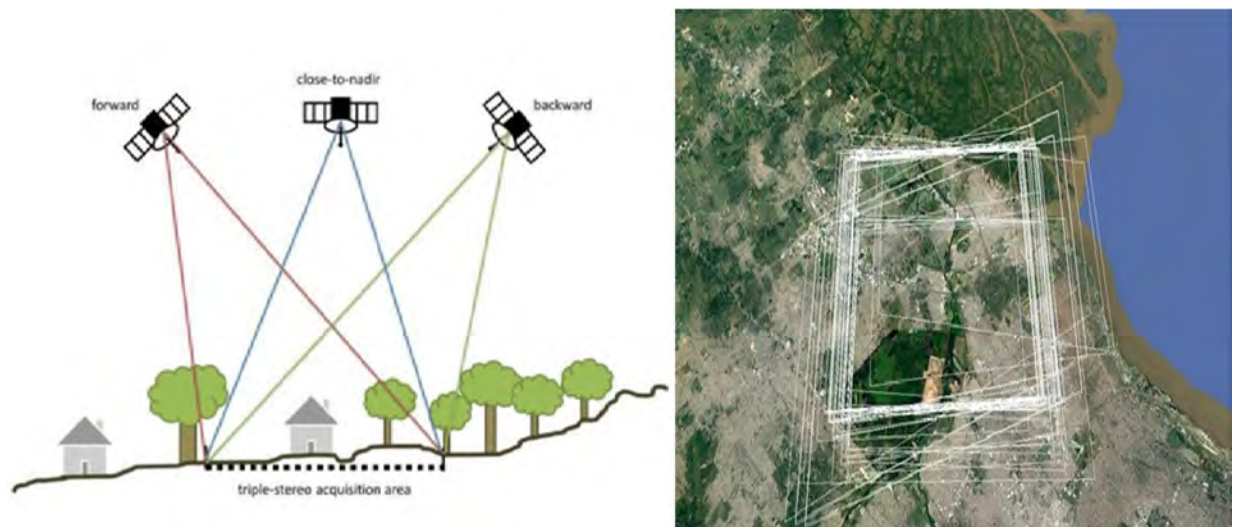
FIGURE 1 Tri-stereo acquisition mode for the DEM generation that was applied in the study. Each location on earth is visited many times by the imaging satellites.

the image points. RPC files contain the metadata that each satellite image has (attributes such as latitude, longitude, height, angle, sun azimuth, cloud detection, etc., at the time of image acquisition, together with field of view and off-nadir angle). The result of this process is a fully covered corridor, which allows conducting the next phase of automatic stereo mapping and point-cloud creation.

Stereo Mapping of a Bare Earth Terrain Elevation Model

The procedure for DEM generation from stereoscopic views can be summarized as follows. The image-matching algorithm identifies corresponding points in at least two images. For a given point in one image, it searches a 2-D grid of points in the second image. By having orientation data, the search is reduced to one dimension: along an epipolar line in the second image. The coarse DEM imposes a constraint on the range of heights in the matching area, which constrains the length of the epipolar line. This reduces both the risk of false matches and the time in matching by reducing the search space. The success of the algorithm depends on the intersection angle and similarity between the images.

The resulting point cloud is then extracted and converted to a DEM, which is a 3-D computer graphic representation of elevation data to represent terrain. In other words, a DEM is a regular grid of points, where each point's x, y, and z geographic location is defined in floating-point numbers.

Creation of a Comprehensive, Ultra-High-Resolution Digital Elevation Model and Digital Terrain Models

For this project, the DEM and digital terrain model (DTM) were calculated by using a stereo-matching technique in a proprietary image analysis software. A pixel matching approach was implemented using known photogrammetry techniques, which combines local and global matching methods to reduce computational effort. This is a significant advantage for large data sets because calculation can be done efficiently. For the stereo matching, it is necessary to have a known epipolar geometry, which is ensured by the RPC files provided with the images. The panchromatic images of the different acquisition angles were used because of their higher spatial resolution in comparison to the multispectral images. A block file containing all the information needed for triangulation (interior orientation, exterior orientation, sensor model, and coordinate system) was calculated. After triangulation, the stereo matching was calculated pairwise for image pairs of the stereoscopic acquisition (forward-close-to-nadir, forward-

backward, close-to-nadir backward), resulting in a point cloud for each image pair containing one point per pixel with the elevation information.

In the last step of the DEM/DTM production, the produced orthophoto of the path was added and applied to the DEM and DTM (Figure 2). Height contour lines were then added, and the result was rendered into a topographic contour map.

Findings

The benefit of using advanced satellite technology to gather high-definition data is that it can be used immediately to assess project contours. The production of contour lines is helpful to plot the route alignment, which is governed by project input parameters such as vehicle speed, maximum gradient, lines of slight, etc. Further, the digitally optimized route can then be used to calculate the cut-and-fill plan, which can then be used to create a bill of quantities for the project. The rapid processing of the data, and the ease of making changes based on different pavement design, culvert thicknesses, or other key project parameters, allows for more accurate bills of quantities and project cost estimates.

Low-volume roads are well suited for rapid assessment of route alignment and geometric designs with advanced satellite technology as long as the length of the project is sufficient that costs can be justified. Several studies have indicated that the cost to acquire good satellite imagery is one constraint to it becoming a more widely
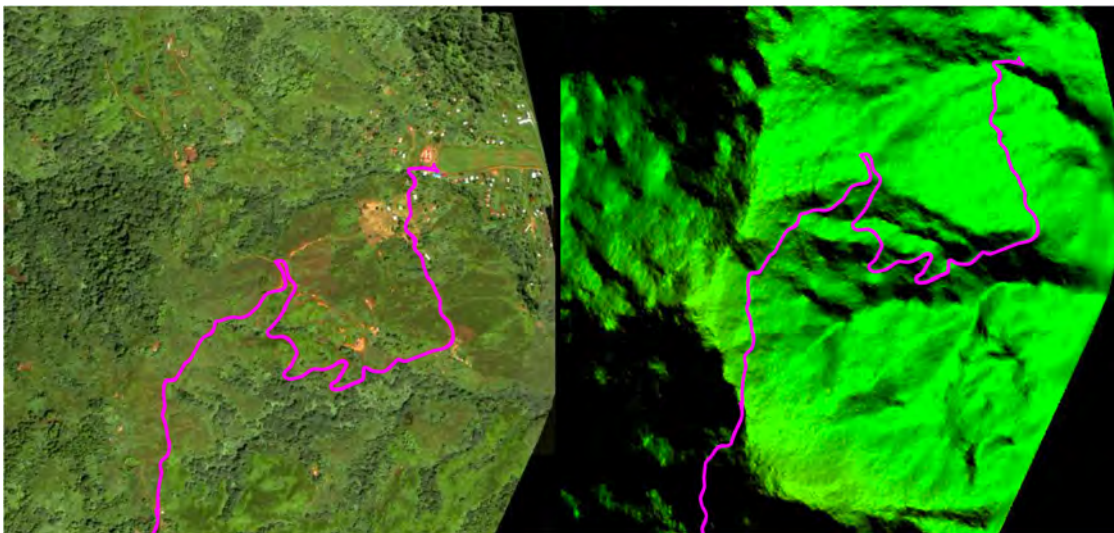
FIGURE 2 The resultant orthoimage (*left*) and DTM from the satellite data for the 174-km route.

spread resource (*4, 5*). However, as with most technology, costs decrease with more users and as the technology is refined.

Conclusions

Technological advancement can be brought about by necessity. In having to conduct two major projects that included route alignment optimization and preliminary engineering design in a remote part of the world during a global pandemic, we were able to create high-definition satellite data that have allowed for creation of contour maps along the proposed pavement alignment. Through application of project specifications, we optimized the project routing to create the most efficient alignment considering the project constraints. The costs associated with the satellite-based methods were considerably lower than traditional remotely piloted aircraft (i.e., drones), planes, or conventional approaches. The turnaround time to acquire the data was almost immediate, which required resources to be allocated to the conversion of the data into digital maps.

In conclusion, the satellite technology was a very valuable approach to data collection, and its applicability to low-volume roads, remote roads, and the assessment of pavement infrastructure will only become more prevalent as both technology and access to data become available.

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Use of a Linear Crusher to Reduce Stoniness Fixed in Existing Gravel Wearing Course Layers

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The most frequently occurring defect on the Western Cape Government's Department of Transport and Public Works (the Department) proclaimed provincial gravel road network is stoniness fixed. Stoniness fixed consists of oversized aggregate (material retained on the 37.5-mm sieve) embedded in the existing wearing course layer (*1*), which reduces riding quality while also creating maintenance problems.

The Department has traditionally utilized grid rollers as the method for breaking down in situ wearing course material, but it was found that harder materials (requiring more than eight grid roller passes) could be broken down more efficiently and effectively using a mobile crusher (*2*). The use of a linear crusher that would be able to break down hard wearing course material on the road was seen as a possible cost effective and efficient solution that would supplement current work methods.

The Department investigated, selected, and acquired an FAE STC200 linear crusher and a John Deere 8245R tractor as shown in Figure 1 for further testing in the field. This study investigates the use of reworking existing gravel wearing course layers by way of a tractor-driven linear crusher to reduce the percentage of oversize material and consequently improve riding quality and maintainability. The testing of the selected tractor and crusher combination was done on three trial sections with stoniness fixed defects, and for which samples were taken and analyzed to determine the impact of the linear crusher on reducing stoniness fixed in existing gravel wearing course layers. The parameters analyzed focused on the percentage oversize index, shrinkage product, and grading coefficient before and after crushing.

FIGURE 1 FAE STC200 linear crusher and John Deere 8245R tractor.

Findings from this study indicate that the FAE STC200 linear crusher and John Deere 8245R tractor combination performs the function it was acquired for and is effective in reducing the percentage of oversize material present in the in situ wearing course layers, without significantly changing the grading that would negatively affect bearing capacity and compaction of the material, thereby increasing the maintainability and performance of the wearing course layer.

Methodology

The Directorate Mechanical Services of the Department did a detailed study on 24 types of crusher and tractor combinations that could be suitable for the application the Department envisaged, from which the FAE STC200 linear crusher and a John Deere 8245R tractor combination, as shown in Figure 1, was selected.

Three trial section sites with stoniness fixed defects were selected near Oudtshoorn to test the tractor and crusher combination. The trial sections were set at 250 m in length and between 6 m to 7.5 m in width. The materials present at the three selected sites consisted of quartzite, greywacke, and shale wearing course as these materials are generally used in the area as wearing course. An optimal work method was developed after several trials were done to determine the most effective work method with the available equipment and personnel. A description of the work method and the trial sections follows:

1. Survey poles are set out every 50 m for layer width and depth control.

2. A dozer (Cat D7H) starts ripping on the left-hand side (LHS) and does three passes to cover the total width of road. A steel plate was fitted to the teeth of the rippers to limit the ripping depth to 150 mm, which may include subgrade in cases where the wearing course thickness is less than 150 mm.

3. The grader (Bell 670G) blades loosen wearing course material that was previously bladed off the road, containing a high percentage of oversize material, from the side drain and shoulder onto the road surface. This material then forms part of the crushing operation.

4. Behind the grader the linear crusher starts with crushing the material, working across the width of the road in four pulls with a ±250-mm overlap on each pull. The linear crusher works at an optimal rate of ±700 m/h.

5. The crushed material is windrowed to the LHS, mixed, and placed using the grader, water cart, smooth steel drum (±12 ton), and pneumatic rollers (±27 ton). Traffic accommodation is done per standard re-gravel operation.

6. Grading and Atterberg limit tests are done on the samples taken in each pull at every 50 m before and after crushing.

The material properties before and after crushing were evaluated in terms of the Technical Recommendations for Highways 20 (TRH20) (*3*) materials performance classification graph. The following specifications pertain to the materials performance classification graph:

1. Grading coefficient: (%passing 28 mm - %passing 2 mm) x (%passing 5 mm/100)
2. Shrinkage product: Linear shrinkage x %passing 0.425 mm
3. Oversize Index: % retained on 37.5 mm ≤ 5%
4. California bearing ratio: ≥ 15% at 95% maximum dry density.

The severity of the stoniness fixed defect is a function of the Oversize Index. Compliance with the Oversize Index specification will result in gravel roads with minimal stoniness fixed defects. It follows that reducing the percentage retained on the 37.5-mm sieve to below 5% (Oversize Index) will decrease the degree and extent of the stoniness fixed defect on gravel roads.

Findings

The before and after crushing TRH20 materials performance classification graph for the greywacke trial section is shown in Figure 2. The red dot shows the average result of the samples tested. The trial sections for the quartzite and shale materials show similar trends.

To evaluate the effect of crushing on the performance of the gravel wearing courses tested, the average and coefficient of variance (COV) for the percentage passing the 28-mm, 5-mm, 2-mm, and 0.425-mm sieves and the difference between the percentage passing the 28-mm and the 2-mm sieves were calculated for all the gradings performed as part of the trial sections. The results of the calculations are shown in Table 1.
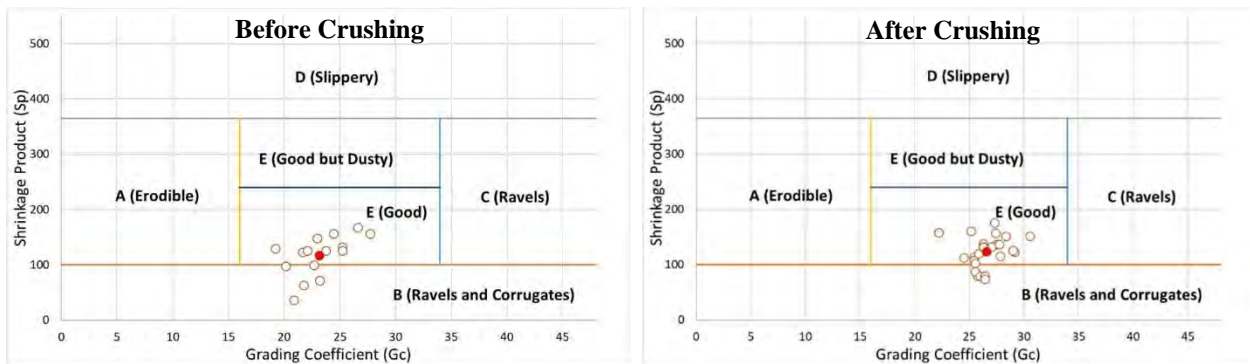
FIGURE 2 Material properties before (*left*) and after (*right*) crushing plotted on the TRH20 materials performance classification graph for the greywacke trial section.

TABLE 1 Grading Analyses for Before and After Crushing

Road No	Material	Before/After Crushing	No. of Samples	%P 28 mm Sieve		%P 5 mm Sieve		%P 2 mm Sieve		%P 0.425 mm Sieve		Difference between %P 28 and 2 mm sieves
				Avg	COV	Avg	COV	Avg	COV	Avg	COV	
DR1693	Quartzite	BC	21	90	5	69	12	59	14	43	19	31
		AC	23	97	2	75	8	65	11	48	15	32
		% increase		8%		9%		10%		12%		
DR1689	Greywacke	BC	28	87	10	61	13	52	13	38	14	36
		AC	50	93	7	65	12	55	12	40	12	38
		% increase		7%		7%		6%		5%		
DR1655	Shale	BC	9	85	10	47	15	37	16	26	20	47
		AC	11	93	3	52	11	41	11	26	14	52
		% increase		9%		11%		11%		0%		

The following conclusions are drawn from the information in Table 1 and Figure 2:

1.   There is a slight increase in the shrinkage product caused by the increase in the percentage passing the 0.425-mm sieve before and after crushing. The linear shrinkage is not expected to change with crushing of the material.

2.   There is an increase in the grading coefficient. This is caused by an increase in both the difference between the percentage passing the 28-mm and 2-mm sieves and the percentage passing the 5-mm sieve with crushing of the material.

3.   There is an increase in percentage passing on all the sieve sizes that are variables in the shrinkage product and grading coefficient equations. The material therefore becomes finer on all the sieve sizes and not only the larger sieve sizes.

4.   The percentage increase in the percentage passing after crushing does not show any significant trends for the different sieve sizes and are of the same order for a specific material type (except for the shale on the 0.425-mm sieve, which cannot be explained other than being an outlier). It follows that the stone sizes for the range of sieves that determines the performance of a gravel wearing course are broken down equally in the crushing process. The complete grading becomes finer during crushing and not only one stone size.

5.   The percentage increase in the percentage passing after crushing does not show a significantly higher percentage for the soft material (shale) than for the harder materials (quartzite and greywacke).

6.   The COVs are lower after crushing than before crushing, indicating that the material becomes more uniform during crushing. This is truer for the larger stone sizes, as can be expected. This is also evident from the before-and-after plots on the TRH20 materials performance classification graph (see Figure 2).

It follows that the grading becomes finer during crushing for the sieve sizes that control the performance of the gravel wearing course, but that one stone size is not broken down unproportionally to other stone sizes. This is significant for the materials tested in that a continuous grading will stay continuous and that crushing will not have a detrimental effect on the bearing capacity and compaction of the material, by generating a large proportion of fines, for example.

The changes in Oversize Index before and after crushing are shown in Table 2 for the materials present in the different trial sections. The values shown are the averages for all the grading tests conducted. Although oversize material is still present after crushing, the percentage is reduced to below the TRH 20 specified value of 5% for

**TABLE 2 Change in Oversize Index Before and
After Crushing the Materials in the Trial Sections**

Material	Oversize Index Before Crushing	Oversize Index After Crushing	Percent Decrease
Quartzite	5.4%	1.3%	77
Greywacke	7.5%	3.2%	58
Shale	9.6%	2.5%	74

all trial sections. As can be expected, there is a significant decrease in the Oversize Index after crushing.

It follows from the preceding discussion that the linear crusher performs the function that it was acquired for and is effective in reducing the percentage of oversize material present in the gravel wearing course, but without changes to the grading that would negatively affect bearing capacity and compaction of the material.

Further evaluation of the performance of full-scale production sections confirmed increased performance after linear crushing of these sections with slower deterioration in riding quality and increased ease of maintenance. An initial cost comparison of the linear crusher method versus the traditional grid roller method showed that, with optimal productivity of the linear crusher, up to a 50% reduction in comparative cost can be achieved. More detailed studies and cost comparisons are required to confirm the findings regarding the cost of the linear crusher operation.

Conclusion

Considering the data collected to date, it is evident that the FAE STC200 linear crusher and John Deere 8245R tractor combination performs the function it was acquired for and is effective in reducing the percentage of oversize material present in the in situ wearing course layers without significantly changing the grading that would negatively affect bearing capacity and compaction of the material. The linear crusher provides an alternate method for breaking down in situ gravel wearing course material while requiring little to no change in current work methods.

From the positive findings of the initial trial sections, in terms of quality and production, the Department purchased an additional two linear crushers and tractors to complement current regravel and rework production teams. Further testing is being done to investigate the use of the linear crusher in regravel operations when breaking down virgin regravel material with varying Treton hardness test values, with the

objective of developing guidelines for the selection of the most appropriate breakdown method for varying material types.

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DRAINAGE

Managing Roadside Ditches and Landowner Right-of-Ways
Connecting the Lines

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Networks of roadside ditches crisscross every watershed. Recently, they are receiving growing recognition as significant contributors to flooding and water pollution in nearby streams. These impacts are being exacerbated by increases in torrential downpours associated with climate change. A 2014 survey of town highway staff from 999 New York towns indicated that the primary barriers to improving ditch management practices were insufficient resources, either as funding, labor, or equipment (*1*).

However, a second key barrier was concern by private landowners who typically own the right-of-ways (ROWs) on the roads that contain the ditches along the 97,800 miles of locally maintained roads in New York State (*2*). Given that over 50% of that local mileage is maintained by towns, there is a need to obtain local support. Ninety percent of the town highway managers (each known as a Superintendent of Highways) are elected officials, so that need for local support becomes even more critical (*3*). There has been no previous research documenting residents' perceptions or understanding of the ROW or roadside ditch management, or their impacts on water quality.

Methodology

Since 2019, we have conducted two mail-out surveys of landowners to assess their understanding of and attitudes toward ROWs and the roadside ditches on their properties. Addresses for both surveys were obtained by initial exploration using Google Earth, then driving and visually recording mailbox addresses for properties having a roadside ditch. The standard Dillman survey method was followed using three mail-outs over a one-month period. The first, broader survey was sent to 400 landowners

throughout rural Tompkins County, New York, in 2019, and it had a final response rate of 38%. The second survey in 2022 was a more concentrated survey of 161 landowners located around Trumansburg Creek in Seneca County, a tributary to Cayuga Lake, and had a higher response rate of 49%.

Findings

Overall, there were several consistent findings between the two surveyed groups. First, the majority of respondents were uncertain about ROW dimensions and who is responsible for management. Despite this uncertainty, they were not comfortable when the ditch was overgrown with weeds, considering it unkempt, with some reporting it felt unsafe. Most respondents were engaged in cleaning trash, mowing, and grass trimming. Only a few respondents reported gardening in their ditches, but only when they were shallow. Three quarters of respondents knew where water in their ditch went after leaving their property, however 50% or more never thought about water quality in the ditch and the potential impacts on streams. Fifty-five percent of respondents in Tompkins County, versus 35% in the Town of Covert, Seneca County, indicated that they had never interacted with their town highway staff and did not know who they were. However, almost every respondent was glad to see the highway staff maintaining the roadside ditches.

There were striking similarities between the two surveys in the responses to questions about supporting practices to improve ditch management that would reduce pollution and flooding. In both surveys, residents indicated a surprising willingness to support better ditch management through an increase in taxes. In Tompkins County, although 43% of respondents indicated that they would not be interested in increasing taxes, 51% said they would, with 28% willing to increase taxes by 0.5%, 15% accepting a 1% increase, and 8% supportive of a 2% increase. In the Town of Covert, 45% reported no allowable increase, but 19% were willing to support a 0.5% increase, 14% to accept a 1% increase, and 2% to support a 2% increase (21% did not answer this question).

The willingness to allow ditch widening so that ditches could be mowed instead of scraped was less similar between the two surveys, with 60% of residents in Tompkins County and only 43% in Covert supporting ditch widening (26% did not answer this question). In Tompkins, 18% indicated that they would support widening the ditch by less than 2 ft, 13% from 2 to 4 ft, 2% from 4 to 6 ft, and 27% responded "as wide as needed." In Covert, the responses for the proposed widths respectively were 19%, 10%, 2%, and 12%.

Conclusions

The results of this survey provide valuable evidence concerning the potential new role of private landowners for improving roadside ditch management to reduce floods and water pollution. There is a real need for an education campaign targeting homeowners about their ROWs and how their ditch functions in flooding and pollution. Homeowners already care about and are engaged in ditch maintenance, and at least half of those surveyed are willing to provide support. In particular, efforts are needed to build stronger communication between highway departments and the people they serve.

Resources

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Performance of Low-Volume Roads Built Over Expansive Soils Reinforced with Wicking Geotextiles

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Farm-to-market (FM) roads built on problematic expansive soils suffer from long-term serviceability, and durability issues due to cracking, rutting, and differential settlements (*1, 2*). The seasonal moisture fluctuations induce cyclic swelling and shrinkage strains on the expansive subgrades that result in the deterioration in pavement quality (*3–9*). Among several alternatives, the application of geosynthetic products has proven to be a cost-effective and sustainable solution to mitigate the problems associated with expansive subgrades (*10–13*).

Traditional geotextile reinforcement improves layer strength and stiffness and helps in partial drainage through gravity (*14–17*). However, due to the development of capillary barrier effects between the macropores of geotextile fabric and the fine pores of soil, the traditional geotextiles often fail to desaturate the subgrade moisture effectively (*8, 9*). Accumulation of the water leads to the lowering of the subgrade stiffness and strength and consequently leads to a rutting failure. A newly available wicking geotextile capable of multiple functions, including separation and reinforcement as well as both gravity and capillary-suction-induced drainage, has been investigated over the last decade (*18–20*). The geotextile is manufactured with special hydrophilic fibers needle-punched in alternate channels with traditional high-strength polymer fibers. The hydrophilic fibers with a multichannel cross-section and high shape factor enable moisture transport in the unsaturated environment (*18, 21*). These microfibers prevent the capillary barrier's development and provide capillary-suction-induced drainage in addition to gravity flow. The combined effects from the needle-punched high-strength polymer fibers and the wicking fibers improve the performance of the pavement layers (*22*).

Several research studies have illustrated the benefits of the wicking geotextiles in improving the drainage capability using both laboratory studies and numerical validations (*19, 23–28*). However, limited studies were undertaken on actual pavement

sections and particularly with subgrade soil having characteristics of swell–shrink potential. Furthermore, the Texas Department of Transportation (TxDOT) has an accumulating stockpile of recyclable asphalt pavement materials, which needs to be disposed of considering the geoenvironmental aspects and landfill costs. Considering the above requirement, a research study was planned to construct and monitor the performance of pavement sections near North Texas, reinforced using wicking geotextiles and with different aggregates in the base layer. The implementation of the new geotextiles is expected to reduce the pavement distress and increase the serviceability of pavement built on expansive subgrades with marginal base aggregates.

Objective and Scope

Two test sections (TS), each 40 m (130 ft), were constructed on a low-volume road at FM-1807, Venus, Texas, during the fall of 2018 (November 2018) (Figure 1). Both sections were reinforced with wicking geotextiles between the base and subgrade layer. The base layer in TS-1 was constructed with 38 cm (15 in.) of reclaimed asphalt pavement (RAP) aggregates and in TS-2 with 38 cm (15 in.) of conventional crushed stone aggregate [or Flex-Base (FB)] conforming to Grade-1 of the recommended material guide by TxDOT (*29*). The base layers were overlaid by 5 cm (2 in.) asphalt course and opened to traffic. The properties of the subgrade soil, base materials, and geotextile are presented in Table 1.

Note: 1 ft. = 30.48 cm
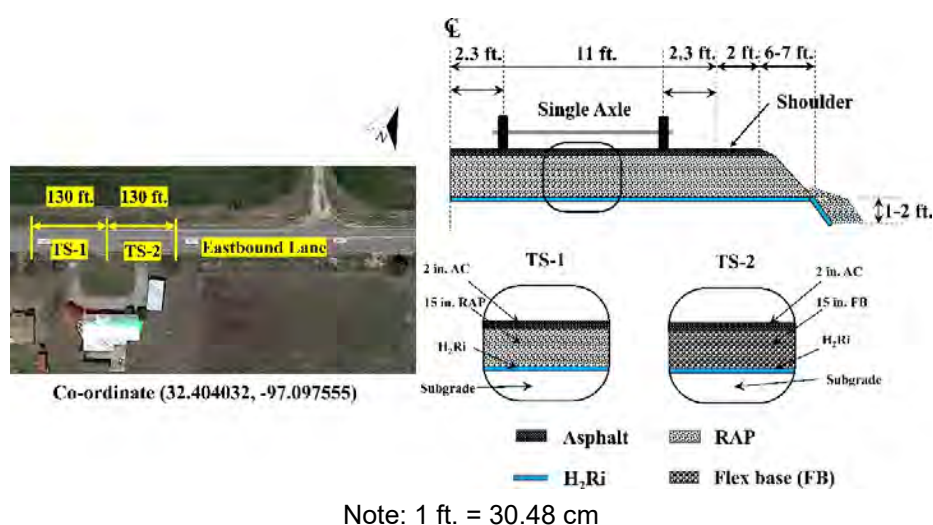
FIGURE 1 Location of sections near FM1807, Venus, Texas, and test section details.

TABLE 1 Basic Material Properties (21)

Material	Reference/ Standard	Property		Value
Soil	ASTM D4318	Liquid limit (LL)		58
	ASTM D4318	Plasticity Index (PI)		31
	ASTM D698	Optimum moisture content (OMC) (%)		23.5
	ASTM D698	Maximum dry unit weight (MDUW) (pcf)		89.9
	ASTM D4546	Vertical free swell strain (%)		8.5
	AASHTO T 307	Resilient modulus (MR) (ksi)		9
RAP[d]	Tex–113E	Optimum moisture content (OMC) (%)		4.5
	Tex–113E	Maximum dry unit weight (MDUW) (pcf)		122
FB[d]	—	Material conforming to Grade 1, Type A of standard Texas DOT construction materials for the flexible base layer (Item 247)		—
Wicking Geosynthetic[c]	—	Roll dimensions (width x length) (ft.)		15 × 300
	—	Roll area (yd²)		500
	ASTM C1559	Wet front movement at STP[a]	24 min in vertical direction (in.)	6.0
			983 min in horizontal direction (in.)	73
	ASTM D4595	MARV[b] of wide width tensile strength (lbs./ft)		5,280
	ASTM D4751	Apparent opening size (AOS) (US Sieve No.)		40
	ASTM D4491	Permittivity (s⁻¹)		0.4
	ASTM D4491	Flow rate (gal/min/ft²)		30

[a] STP = standard temperature and pressure.
[b] MARV= minimum average roll value.
[c] Manufacturer specifications.
[d] FB = flexible base (crushed stone aggregates); RAP = reclaimed asphalt pavement aggregates.

The primary scope of this study was to understand the performance of the reinforced pavement sections monitored using falling weight deflectometer (FWD) studies performed over the next couple of years after the construction of the section. The performance of the TS(s) was compared with existing control pavement sections (CS), having 10 cm (4 in.) asphalt layer and 33 cm (13 in.) of the base with FB aggregates.

Methodology

The structural performance of the pavement layers was evaluated using non-destructive FWD tests. FWD has been used extensively to evaluate the in situ moduli of the pavement layers for material model calibration according to the *Mechanistic–Empirical Design Guide*. The FWD device is capable of applying impulse load by dropping weights from various drop heights on the pavement surface. An impulse load of 9,000

lbs. was applied vertically on a 30-cm (12-in.) diameter loading plate to record the field deflections at 0, 30 (12), 60 (24), 90 (36), 120 (48), 150 (60) and 180 (72) cm (in.) with seven deflection sensors: D_0, D_{12}, D_{24}, D_{36}, D_{48}, D_{60}, and D_{72}. The first deflection sensor was placed just under the loading plate to record the maximum deflection (D_0). The center-to-center spacing among the remaining deflection sensors was 30 cm (12 in.), resulting in a deflection basin with a radius of 180 cm (72 in.). The stations were selected at an interval of approximately 20 ft from each other in each TS and CS, and two load drops were performed at each station. A total of three FWD tests were performed over the last 3 years (2020–2022) after construction in December 2018. The results from the FWD studies and performance interpretations are presented in the next section.

Analyses of Results and Discussions

The performance of the TS(s) and the CS was interpreted using the deformation values recorded with the geophones. The performance indicators selected in this study were base layer index (BLI), lower layer index (LLI), and AREA72 (Equations 1, 2, and 3).

$$BLI = D_0 - D_{12} \qquad (1)$$

$$LLI = D_{24} - D_{36} \qquad (2)$$

$$AREA_{72} = 6 \left(1 + 2\frac{D_{12}}{D_0} + 2\frac{D_{24}}{D_0} + 2\frac{D_{36}}{D_0} + 2\frac{D_{48}}{D_0} + 2\frac{D_{60}}{D_0} + \frac{D_{72}}{D_0}\right) \qquad (3)$$

The BLI and LLI provided useful information on the structural condition of the base and the subgrade layers, respectively. The AREA72 parameter was calculated based on the trapezoidal rule and used to characterize the deflection basin due to the applied loads. The method has been extensively adopted with some added refinements in the 1993 version of AASHTO's *Guide for Design Pavement Structures*. The results from the FWD analyses for every station are presented in Table 2.

The sections with BLI values below 200 μm and LLI values below 50 μm were indicative of the sound base and subgrade layer. Similarly, BLI between 200–400 μm and LLI between 50 to 100 μm indicated a moderate structural performance and needed to be monitored closely. The section with RAP and wicking geotextile was observed to perform partially better than traditional crushed stone aggregate FB material and wicking fibers. Furthermore, it was observed that the performance of the 2-in. asphalt-

TABLE 2 Performance Analyses of Test and Control Sections

Section	Station	BLI (µm)			LLI (µm)			AREA72 (in.)		
		2020	2021	2022	2020	2021	2022	2020	2021	2022
TS-1	1	135.6	171.7	194.1	53.0	50.0	53.8	26.2	23.9	23.6
	2	165.9	195.1	194.3	48.3	46.7	51.6	24.7	23.1	23.7
	3	160.1	181.1	189.4	48.5	48.5	52.2	24.6	23.3	23.9
	4	162.2	190.2	172.0	46.2	45.5	54.7	24.5	23.0	24.5
	5	150.7	185.7	228.6	45.7	47.2	71.0	25.0	23.2	22.8
	6	139.4	182.4	246.8	47.1	47.2	67.9	25.6	23.0	22.1
	7	180.7	230.1	248.5	58.9	56.9	78.4	23.9	21.8	22.2
TS-2	1	197.9	258.1	247.9	66.5	56.4	73.3	23.5	21.1	22.5
	2	206.2	256.3	249.7	70.5	62.2	63.1	23.1	21.2	22.0
	3	202.3	248.7	275.0	73.4	58.4	61.2	23.7	21.6	20.9
	4	191.3	230.9	220.9	63.4	53.1	69.2	23.7	21.9	23.0
	5	231.0	264.4	245.1	62.0	51.3	59.3	22.0	20.5	22.1
	6	208.4	215.4	207.3	90.7	57.4	56.3	23.1	22.4	22.6
CS	1	138.7	222.8	222.9	49.9	37.6	50.9	24.7	20.4	21.7
	2	133.9	207.5	216.7	41.0	36.1	48.9	24.4	20.6	21.5
	3	125.1	201.2	176.0	39.6	38.9	49.4	24.6	21.3	23.6
	4	105.5	185.9	161.4	42.3	51.3	57.7	27.2	22.5	24.7
	5	105.0	197.4	176.9	43.2	45.5	57.7	27.3	21.9	23.9
	6	126.7	187.2	163.6	45.7	41.7	52.5	25.7	22.4	24.6
	7	117.7	223.3	202.1	43.6	45.2	56.0	26.3	21.0	22.9

Note: 1 µm = 3.9 × 10^{-5} in.

layered TS(s) were comparable with the 4-in. asphalt-layered CS. This indicated that the application of the wicking geotextiles was effective in potentially improving the layer stiffness or moduli values and consequently resisted distresses from both traffic and environmental loads.

In addition to the performance evaluation, the results from the FWD analyses were used to back-calculate the in situ modulus and remaining rut-life for the pavement sections at different stations (Figure 2). The analyses were performed for both test and control sections using FWD data collected in 2022. The back-calculated moduli values of base and subgrade layers in all the sections ranged between 150–410 MPa (22–60 ksi) and 100–150 MPa (15–22 ksi), respectively. The non-destructive test results indicated that the marginal quality aggregates reinforced with wicking fibers (TS-1) have comparable performance to CS with a 4 in. asphalt layer. The geotextile layer improved the layer modulus values of the RAP layer more than the crushed stone layer. The improvement in the subgrade modulus in TS-1 and TS-2 was comparable to each other. Additionally, the remaining rut-life of the pavement sections was calculated for 20 years of traffic loads with an approximate 5 million equivalent single-axle loads (average daily traffic = 1,500) and rutting failure criteria of 0.25 in. Based on this criterion, the
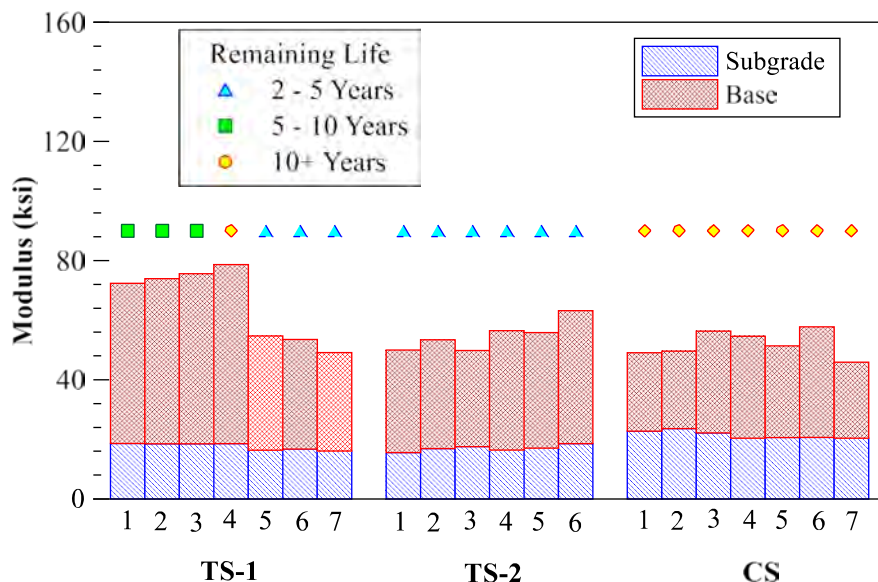
FIGURE 2 Layer moduli and rut-life for test and control sections.

remaining rut-life of CS was observed to be 10 plus years, and in the TS-1 and TS-2, the values ranged between 5 to 10 and 2-5 years, respectively.

Conclusions

The study presents the results from the monitoring of pavement sections reinforced with wicking geotextiles constructed over expansive subgrade. FWD studies were performed on the test and control sections to evaluate the performance of the wicking geotextiles. Some major conclusions from the analyses of the results are presented below.

- The application of wicking geotextiles helped to improve the pavement sections, and no major distress was noted after 3 years of serviceability.
- The application of wicking geotextiles with marginal quality base aggregates (RAP) was observed to have comparable performance to unreinforced sections with thick asphalt layers, indicating the potential efficacy of the geotextiles.
- The reinforced layer helped to improve the base and subgrade moduli values.
- The novel geotextiles have the potential to be used for reinforcing the pavements constructed over expansive subgrade.

A long-term monitoring of the test and control sections in the future would provide

additional performance data and help us to develop a sustainable construction guideline for pavements built on problematic expansive soils.

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Detection and Localization of Culverts on a Road Stretch Using Ground-Penetrating Radar

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Climate change is causing more intense loads on the drainage systems, and increased damages partly caused by malfunctioning drainage systems have become more common. It is important that all parts of the drainage system are working well. If one part fails, it can lead to damages on the road and its subbase as well as a reduction of the driving safety. Culverts lead the water from one side of the road to the other side (*1*). There are a huge number of culverts on the road. More than 550,000 culverts are registered in the Norwegian national data base (NVDB). Culverts are, by definition, located under the surface and are spread out over the whole country, even in remote locations or surrounded by terrain, making access difficult. Traditionally, culverts are monitored via visual inspection, which is not only a time-consuming and costly method, but also an inefficient one. This means that a large part of the culvert is never inspected before problems arise. In the last several decades non-destructive testing (NDT) methods have gained more and more interest in the transportation field as they allow data collection with minimal impact or damage. Ground-penetrating radar (GPR) counts as an NDT technique and has been successfully tested and implemented in different fields, including for measurement of pavement thickness, bridge inspection, and subsurface void detection (*2*). There are also several studies dealing with subsurface utilities detection and mapping as well as with condition assessment (*3–6*). These studies are usually carried out on test sites or in a laboratory, and they focus on urban conditions and surroundings.

In this study GPR is used to test its suitability and efficiency to locate culverts on a road stretch. The data are compared to the records in the database to see how complete and correct they are as well as to explore how GPR can contribute as a data collection method. With a culvert detection rate of 68.8%, the study shows that GPR is successful in gathering more information about the road drainage. Moreover, undocumented culverts have been found in the GPR data set, and a shift between the location documented in the NVDB and in the GPR data set has been discovered. This shows what a valuable contribution GPR can make in updating the information.

Methodology

The study includes a field study for data collection on a road section and a comparison between the collected GPR data and data registered in the NVDB. For the field study, GPR data were collected on a 19-km-long road stretch on a Norwegian two-lane road. The GPR system used is a ground-coupled GPR antenna array with a total scan width of 1.5 m. A total of four runs were carried out, two in each driving direction, to cover as much of the cross-section of the road as possible. The radar is a step-frequency radar with a frequency range of 200–3,000 MHz. The depth range was set to 62.5 ns, which resulted in an approximate penetration depth of 4 m. The selected system and survey settings led to a maximum driving speed of 16 km/h. The data were then processed and analyzed with the Examiner software, comprised in the radar system. As the system set-up is equipped with an internal and external GPS system, all data measured with the radar are georeferenced. The following four processing steps were applied: interference suppression, inverse fast Fourier transformation, gradual low-pass filter, and background removal filter. The focus of the analysis was on finding hyperbolas corresponding to culverts in the three-dimensional data set and georeferencing the exact location. As a following step, the culverts found in the GPR data are compared to the culverts registered in the national database. In addition, the data set was analyzed regarding the correlation between culvert properties, such as inner diameter and material, and the success rate of the proposed methodology. There is no data registered in NVDB regarding the depth of the culvert, therefore the correlation between the depth and the success rate of finding the culvert could not be examined.

Findings

On the 19-km-long road stretch, 173 culverts are registered in the national road database. Of those 173 culverts, 119 culverts were identified in the GPR data set, which is a detection rate of 68.8% in the GPR data. Culverts were found in depths ranging from 0.4 to 3.9 m in relation to the road surface, which covers almost the full penetration depth. The inner diameter of the culverts varies from 100 to 2,000 mm. The distribution of inner diameter sizes is very uneven, and some diameters are occurring only one time on the road stretch. Therefore, the comparison is difficult, and it cannot be said with certainty if the success rate of locating a culvert is correlated with its inner diameter size. Figure 1 shows a comparison of the success rate of finding the culvert for inner diameters, which occur in total more than 10 times on the road stretch
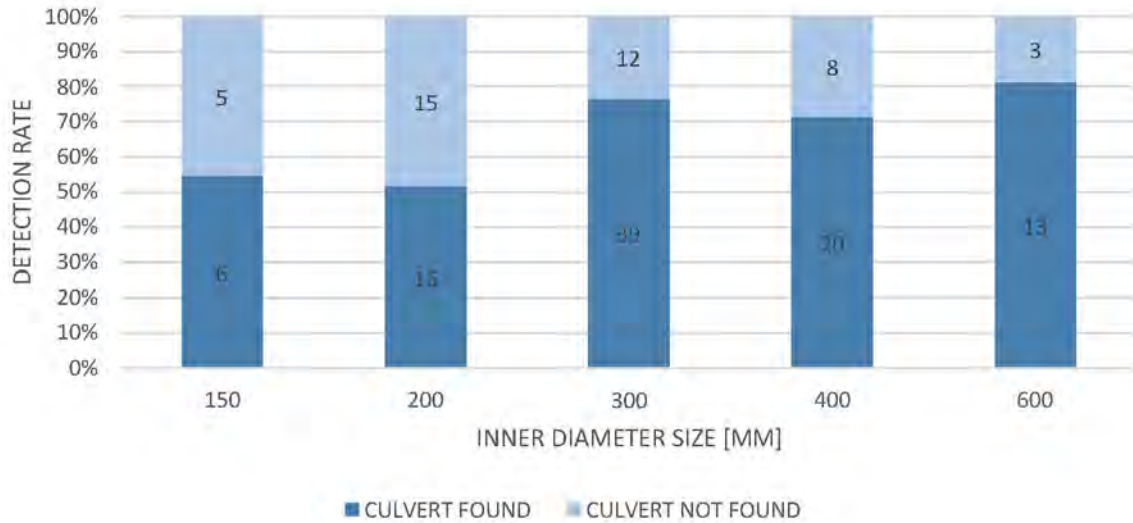
**FIGURE 1 Success rate of finding a culvert in correlation
with the inner diameter size.**

(150 mm, 200 mm, 300 mm, 400 mm, and 600 mm). It appears that with an increasing inner diameter it is more likely to detect the culvert with a GPR. A comparison between the detection rate for all culverts and the detection rate for culverts with an inner diameter size greater than 300 mm shows the same trend, as the detection rate increases from 68.8% to 76.5%.

The same issue with an uneven distribution applies to the material, where concrete culverts are more common than plastic culverts and only three culverts are stone culverts. However, if only concrete and plastic culverts are compared, the detection rate for concrete culverts (71.8%) is higher than the detection rate for plastic culverts (64.1%) (see Figure 2). One reason could be that the percentage of culverts with an inner diameter size smaller than 300 mm is higher for plastic culverts. The total number of concrete culverts is in addition much higher, which again makes it difficult to compare the detection rate.

The analysis of the GPR data set also revealed that the data on NVDB are not complete and imprecise. First, 39 more locations were identified where there is potentially another culvert, and where no corresponding culvert could be found in the national database. To confirm that there is an undocumented culvert, further investigations, such as visual inspections are necessary. Furthermore, the recorded location in the database is in most cases inaccurate. An offset of between –16.0 and 43.8 m between the location documented in NVDB and the location determined through the GPR measurements exists.
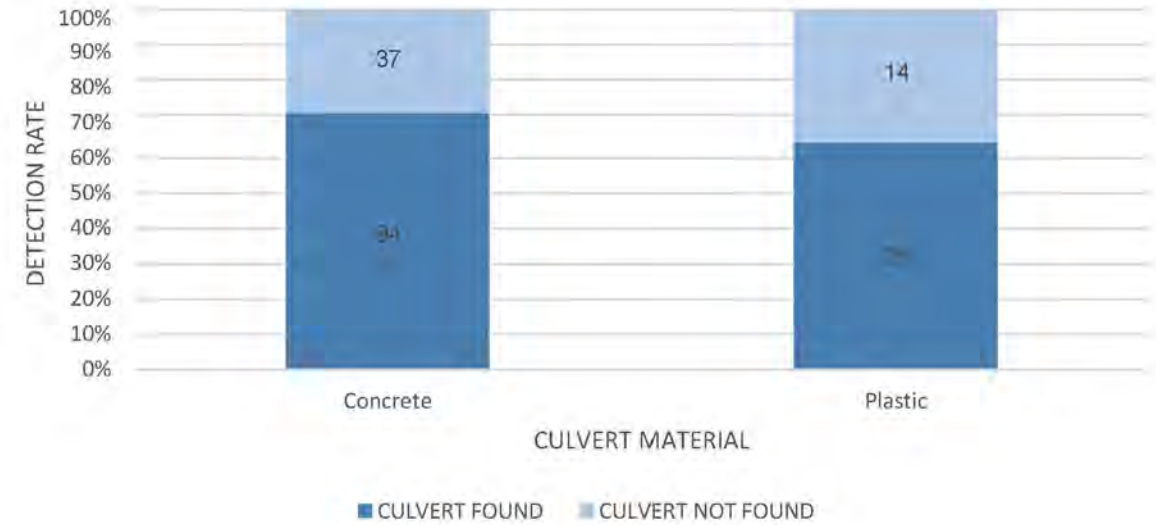
FIGURE 2 Success rate of finding a culvert in correlation with the material.

Conclusions

This study is investigating the suitability of GPR to detect and locate culverts on a road. GPR measurements on a 19-km-long stretch were conducted. The detection rate for culverts was 68.8%. The analysis, however, showed that the registration of culverts in the NVDB is not complete and that the documented location of the culverts is shifted in comparison to their real location. Further tests, such as visual inspections will be conducted to create a true data set and to validate the findings. The study showed that the success rate to find culverts is higher for an increasing inner diameter as well as for concrete as material. However, more culverts should be included in the study as the distribution for diameter size and material were unevenly distributed and can influence the results. Overall, the study showed that GPR as a method to detect and locate culverts on a road is a valuable contribution for road owners to gather more information about their assets. Being able to locate the culverts enables monitoring, by GPR or other methods, and ensures timely maintenance of the culverts.

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Wildlife-Friendly Drainage Structures

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Because of the rapid expansion of road networks worldwide, the decline of animal populations in most countries, millions of animals that are killed annually on highways, and the human cost of accidents, there is considerable interest in highway-wildlife crossings today. Wildlife overpasses and underpasses have been studied and built for over the past 25 years. The Federal Highway Administration (FHWA) published their document *Critter Crossings* in 2000 (*1*), and the U.S. Forest Service developed its Wildlife Crossings Toolkit in 2003. FHWA subsequently published other definitive documents such as their wildlife collision reduction best practices manual in 2008 (*2*) and their *Wildlife Crossing Structure Handbook* in 2011 (*3*). Banff National Park in Canada has been a success story with the numerous large wildlife crossing bridges built over a major park highway, and wildlife crossing structures have been shown to be very effective. Fish passage through culverts (*4*) and making culverts suitable for wildlife movement have also been promoted for years (*5*).

Despite the many efforts to prevent automobile–wildlife collisions and promote wildlife movement and connectivity through their natural habitats, millions of animals continue to be killed annually or their habitat fragmented, particularly in developing countries and tropical regions rich in biodiversity such as Costa Rica, Brazil, or Nepal. Research and studies are ongoing to understand the science behind wildlife movements and infrastructure impacts (*6*). Though wildlife crossing structures can be very effective where implemented, a general lack of understanding of the problem, lack of funds, and lack of political will are greater constraints today than the science. To date there is little mention of mitigation methods in basic engineering or environmental engineering curriculum, so most knowledge is gained through research and on-the-ground experience. Accommodating wildlife movement through drainage structures often requires modifications to traditional designs or creative engineering solutions to satisfy both site hydrology, hydraulics, and wildlife passage needs for specific target species. A discussion on practical drainage structure solutions follows.

Some wildlife considerations important for wildlife use and movement in typically engineered roadway drainage structures include the following:

- Most large bridges can accommodate wildlife movement, but the form of the channel ideally needs to include a dry area, such as a floodplain or terrace, to allow for wildlife movement most of the time.
- Box culverts and bottomless arch culverts are preferable to small round culverts to promote wildlife use. The size of the structure useful for wildlife crossings depends on the target species.
- Culverts may need to have a bench or platform built through the structure that is dry most of the time to allow for wildlife passage.
- For fish passage, a culvert is ideally at least as wide as the bankfull channel width and have a natural channel substrate through the culvert, as in a stream simulation design.
- Avoid box culverts with concrete slab bottoms at-grade because they can promote the formation of a waterfall at the culvert outlet, creating a barrier for some species.
- Avoid fords that have a significant drop off the downstream edge of the structure. A vented ford with culvert boxes and a natural material in the bottom might be needed.
- Energy dissipator structures or deep box inlet structures may form traps for animals moving along a drainage channel.
- Avoid steep-walled roadside ditches or trenches that may trap animals.

Methodology

This extended abstract is based on personal experience working and presenting training on wildlife crossing projects with the U.S. Forest Service and involvement in workshops and conferences regarding wildlife issues in various countries. Recent work in Nepal has involved wildlife crossing options and connectivity issues for a major highway widening project adjacent to two national parks (Chitwan National Park and Parsa National Park), famous for their wildlife diversity.

Training and project work have involved evaluation of bridges and drainage crossings for their suitability for wildlife movement, a review of wildlife crossing literature, appropriate structural designs, and discussions with non-governmental organizations, wildlife groups, and wildlife biologists. Information and observations presented herein are based on experiences and insights gained in working on this topic periodically over the past 20 years.

Findings

Bridges

Bridges are by far the most desirable wildlife crossing structure because of their relatively large size, with a span of at least 7 m, and the wide variety of animals that use drainages as travel corridors. Also, most bridges are necessary to cross over streams or rivers, so accommodation for wildlife typically involves minimal or no additional cost. Install bridges rather than culverts whenever possible when wildlife movement is an issue; the added benefit is that bridges have the most flow capacity and least risk of failure when properly designed and when considering climate resilience.

To be "wildlife friendly," design bridges to span the entire watercourse width, typically placing the abutments outside the active channel and using spill-through abutments with sloping sides that mimic natural streambanks. Ideally, a zone that is dry most of the time (except during floods), such as a floodplain or riverbank, provides a corridor for wildlife movement, as seen in Figure 1.

Maintain a natural stream channel bottom (substrate) through the bridge structure. If rock (riprap) is used for streambank protection and bridge abutment armoring, provide a bench through the riprap with a soil or gravel surfacing to allow for wildlife movement along the margins of the stream corridor.
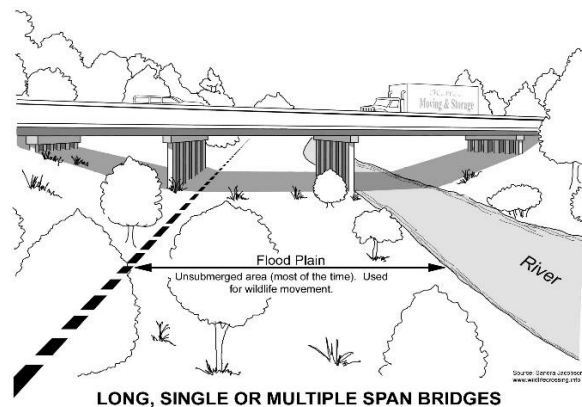
FIGURE 1 Photo and figure of major bridges with a dry floodplain terrace that is ideal for wildlife movement through the bridge.

Culverts

Box culverts (typically concrete) are generally preferred to round culverts since a box can have a flat, more natural (soil layer) bottom than in a round culvert. To have a portion of a round culvert with a natural soil bottom, the culvert has to be partially infilled with soil. A minimum culvert size (width) of 1 to 2 m is desirable both to accommodate wildlife movement and for culvert cleaning. Moderate-size structures, 6 x 3 m (width x height) are recommended for common medium-size animals, while up to 12 x 6-m structures may be needed for very large animals (*7*). Size should be based on the needs of specific target species.

Use boxes and bottomless arches that have a natural stream channel bottom material (substrate). This typically requires that a structure use spread footings for a foundation. Alternatively, for a "box" design with a concrete bottom, sink the structure deep enough below the natural channel elevation that it will partially aggrade and backfill with natural channel material (sand, gravel, boulders). An at-grade concrete bottom on a box culvert (or through a bridge) will accelerate stream flows and often form a waterfall or scour hole at the downstream edge of the structure, creating a barrier for some species (Figure 2).

For culverts that have perennial flows and are typically partially flooded, install benches through the structure, typically made of rock, masonry, concrete, or wood, so that there is a dry crossing area through the culvert most of the time (Figure 3). This may require oversizing the structure to still pass the design flow, debris, and sediment, as well as require additional maintenance and debris cleaning.

FIGURE 2 A box culvert with a smooth concrete bottom that accelerates streamflow and forms a waterfall that is a barrier to some small animals or fish.

FIGURE 3 Culverts with a raised bench constructed of concrete or retrofit with rock to provide a dry platform for wildlife movement through the culvert. (Photos from Adolfo Artavia and Randall Reeves).

Stream Simulation Culverts

In the past 15 years, many culverts have been replaced to improve aquatic organism passage and stream function, using open-bottomed arch structures or bridges to accommodate fish passage at a range of flows. Channel-design techniques that mimic natural stream-channel condition upstream, through, and downstream of the crossing are used (*8*). They promote both aquatic organism passage and potential wildlife movement (Figure 4). Additionally, this design concept often has a capacity for a 100-year flood event, useful for a climate-resilient design.

Design culverts to have a span at least as wide as the bankfull flow width of the natural channel and with a natural channel bottom substrate through the structure. Bankfull width is the limit to where flows reach about every 1.5 to 2 years ($Q_{1.5} - Q_2$).

Low-Water Crossings

Either a simple ford with a natural channel bottom or a vented ford with large vents (box culverts) and a natural channel bottom material can accommodate wildlife and fish movement. Since wildlife commonly move along drainage channels, avoid designs that create a wall (waterfall) or barrier at the downstream edge of the low-water crossing. In a steep channel that requires an elevation difference between the road level and stream channel bottom, install a vented ford with large, at-grade boxes, or install a bridge.
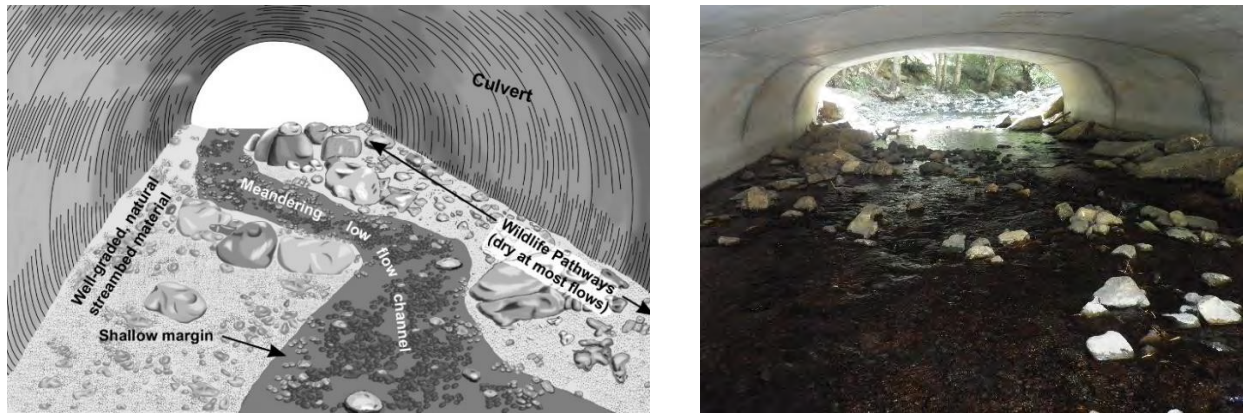
FIGURE 4 Drawing and photo of stream simulation designs through culverts, replicating a natural stream channel bottom substrate, and promoting aquatic organism and wildlife movement for some species. (Source: Keller and Ketcheson 2015).

Surface Drainage Features

Many surface drainage features such as deep ditches, drop inlet structures, stair-step energy dissipator or down-drain structures can either trap small animals or create barriers to their movement. Vertically walled ditches should be avoided. Ideally ditches should be built with a "V" shape, and side slopes should be built to be as gentle as possible. Grass-lined ditches may be suitable but also may provide an attractive breeding or feeding habitat that increases roadkill.

Large drop-inlet structures on culverts or for ditch-relief cross drains may be a trap for animals, so they should be built so animals can escape. Either an escape ladder or ramp can be built into the structure, or the animals can get out by exiting along the attached culvert. Sand traps may also be animal traps! Energy dissipation structures also can be built to prevent trapping of small animals or designed to allow animal movement.

Conclusions

Drainage structures, including bridges, culverts, ditches, and related structures, are fundamental along roads for hydrologic design and for stream crossings. Most bridges and stream simulation design culverts are ideal for wildlife movement and habitat connectivity or can be modified to facilitate wildlife passage. Proven methods are

available today, both in wildland and suburban areas, that can be wildlife friendly and promote wildlife movement, particularly along stream corridors.

Costs for wildlife friendly bridges typically involve minimal additional investment, other than possible fencing. Stream simulation culverts are typically larger than hydraulic design culverts, so their initial cost is more expensive, particularly for large structures. However, they are often more cost-effective and have lower life-cycle costs because of a longer life span, less armoring, less maintenance, avoided damage and repair costs, a lower risk of failure, and improved flood resilience (*9*).

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SPECIAL APPLICATIONS:
AIRPORTS AND COLD REGIONS

Application of Light Weight Deflectometer to Assess Structural Competency of Nontraditional Airfield Pavements During Contingency Aircraft Operations

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Airfield pavements in contingency environments may not meet minimum pavement thickness design standards or may be severely deteriorated due to aging or conditions that are not included in current pavement evaluation techniques. This paper discusses the use of the heavy and light weight deflectometer devices, hereafter referred to as HWD and LWD, respectively, to monitor the structural competency of airfield pavements at contingency environments during aircraft operations. Four airfield pavements like those encountered at contingency environments, referred to as nontraditional airfield pavements, were selected to conduct field evaluations with simulated C-17 and C-130 aircraft. Rutting was monitored, and both HWD and LWD tests were also conducted at selected traffic intervals. Both the HWD and LWD devices could indicate the pavement structural condition during accelerated aircraft traffic. An acceptable relationship was identified between the HWD and LWD, and test results indicate the LWD can be deployed as a rapid and practical pavement evaluation tool for nontraditional pavements during contingency aircraft operations. Based on current rutting failure threshold, a termination criterion in terms of a reduction in the stiffness parameter from the LWD test could be preliminarily used to aid in expediently determining acceptable aircraft operations on nontraditional airfield pavements.

The Role of Temperature Gradient and Soil Thermal Properties on Frost Heave

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In cold regions, the soil temperature gradient and depth of frost penetration can significantly impact roadway performance due to frost heave and thaw settlement of the subgrade soils. The severity of the damage depends on the soil index properties, temperature, and level of the groundwater table. While nominal expansion occurs with the phase change from pore water to ice, heaving is derived primarily from a continuous flow of water from the vadose zone to growing ice lenses. The temperature gradient within the soil influences water migration toward the freezing front where ice lenses form and grow. This study evaluates the frost heave potential of frost-susceptible soils from Iowa (IA-PC) and North Carolina (NC-BO) under different temperature gradients. One-dimensional frost heave tests were conducted with a free water supply under three different temperature gradients of 0.26°C/cm, 0.52°C/cm, and 0.78°C/cm. Time-dependent measurements of frost penetration, water intake, and frost heave were carried out. Results of the study suggested that frost heave and water intake are functions of the temperature gradient within the soil. A lower temperature gradient of 0.26°C/cm leads to the maximum total heave of 18.28 mm (IA-PC) and 38.27 mm (NC-BO) for extended periods of freezing. A maximum frost penetration rate of 16.47 mm/h was observed for higher temperature gradient of 0.78°C/cm and soil with higher thermal diffusivity of 0.684 mm^2/s. The results of this study can be used to validate numerical models and develop engineered solutions that prevent frost damage.

Development of Deceleration-Based Friction Prediction Models and Methods on Semi-Prepared Runway Surfaces

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Both civilian and military airfield managers require the safe operation of aircraft on runway surfaces. It has been shown in numerous research efforts that surface friction is an excellent indicator of runway condition. While a great deal of work has been done to quantify safe runway conditions for landing and takeoff operations on rigid and flexible pavement structures, limited research exists to extend such efforts to soil-based or semi-prepared runways. The objective of this research effort was to develop deceleration-based surface friction prediction models on unpaved surfaces with varied moisture conditions and soil types. Surface friction, in this study, was quantified using the Findlay Irvine Mk2-D GripTester. Deceleration was measured using four smartphone inertial measurement units (IMUs), one Bowmonk IMU, and one Xsens IMU. Tests were conducted in three ground vehicles: a high-mobility multipurpose wheeled vehicle, a civilian ½-ton pickup truck, and a civilian full-size sport utility vehicle. The various deceleration-based devices tested here adequately correlate (coefficient of determination > 0.6) to Mk2-D GripTester measurements collected on unpaved soil runways. The models and measurement methods detailed here are of considerable use to semi-prepared runway airfield managers around the world needing to measure safe landing conditions following inclement weather.

Innovative Air Convection Embankment for Cold–Arctic Region Low-Volume Roads

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Roadbed thaw settlement is a unique challenge for the durability of low-volume roads (LVRs) in permafrost regions. Air convection embankment (ACE) is an effective technique that acts as a semi-heat-transfer system to control temperature variation and reduce the thaw depth of subsoil. However, limited by the shortage of desired crushed rocks, building an ACE in Alaska is prohibitively expensive. Previous studies identified the feasibility of using cellular concrete for ACE and determined the optimized thickness of the cellular concrete aggregate interlayer for ACE. However, the economic efficiency, thermal, and mechanical performance of the optimized, innovative cellular concrete aggregate ACE need further investigation. Hence, two innovative cellular concrete ACEs with reasonable heights for Cold/Arctic Region LVRs were evaluated in this study. A thermal-mechanical coupling model was created using ANSYS Fluent and ANSYS Mechanical to evaluate the thermal and mechanical stability of the two optimized innovative cellular concrete aggregate ACEs by comparing with a typical Alaskan flexible pavement, a silty sand/gravel embankment, and a conventional crushed-rock ACE. The fatigue damage was predicted using the elastic-based Alaska Flexible Pavement Design (AKFPD) program. A life-cycle cost analysis was conducted using AKFPD to evaluate the overall long-term economic efficiency of the cellular concrete ACEs. The results showed that cellular concrete ACE could achieve better thermal and mechanical performance with much lower embankment height than crushed-rock ACE. The cost analysis showed that the proposed cellular concrete ACEs had a significant cost advantage over the conventional crushed-rock ACE.

PAVEMENT MAINTENANCE AND PRESERVATION

Assessment of Life-Cycle Benefits of Bio-Based Fog Sealant for Low-Volume Asphalt Pavement Preservation

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Bituminous pavement can become brittle and distressed over time due to oxidation of asphalt binder caused by exposure to air, and multiple preservation methods could be implemented to prevent bitumen oxidation and prolong lifespans of asphalt roads. Fog seal, a common preservation strategy, refers to applying a thin layer of asphalt emulsion on an existing road surface to mitigate oxidation and moisture penetration. During past decades, some innovative sealing agents derived from biomass have drawn considerable attention due to their economic benefits and eco-friendly properties, and RePLAY is such a proprietary soybean-derived sealer that has been successfully applied in many states. While some studies have investigated its performance and concluded that it could effectively preserve asphalt roads and potentially prolong their service life, few employed comprehensive life-cycle cost analysis (LCCA) based on the field performance and actual construction cost and made appropriate recommendations to local public agencies. To fill this gap, this study selected a low-volume asphalt pavement in Clinton County, Iowa, and during summer 2016 sprayed RePLAY on it for five consecutive years of investigation, with an annual distress survey conducted on both untreated and RePLAY-treated sections for comparison purposes. Field assessment during a five-year period indicated that RePLAY treatment could control crack growth of the RePLAY installation and road maintenance costs were combined and LCCA was employed, with results suggesting that, among the various options investigated in this study, three treatments within a 5-year interval during a service period was the most cost-effective option.

Performance of Cape Seal Treatments Used for Pavement Preservation

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Cape seals were originally developed in the Cape Province of South Africa in the 1950s. They are combination treatments that consist of a chip seal treatment followed by an application of a slurry seal or micro surface to partially fill the void space between the aggregate particles or completely cover the surface (*1*). Cape seals help protect the pavement structure and extend service life by sealing the surface, protecting the structure from oxidation and raveling, improving skid resistance, and restoring the pavement's appearance.

This two-step process approach combines the benefits from each treatment layer, increasing their effectiveness compared to stand-alone treatments. Cape seals have been found effective in delaying cracking reflection and raveling as a result of this dual action (*2*). In addition, cape seals are also capable of maintaining the pavement structure in sound condition for a longer time compared to untreated sections (*3*). Nair et al. documented the installation and short-term performance of three preventive maintenance treatments (cape seal, fiber-reinforced cape seal, and micro surface) applied to US 301 in Sussex County, Virginia (*4*). After 3 years, the cape seals exhibited better performance in terms of reflective cracking compared to the stand-alone micro surface. All treatments provided good surface characteristics such as friction, macrotexture, and ride values.

The estimated treatment life for a cape seal ranges from 6 to 15 years and is affected by factors like existing pavement condition, material properties, mix design, construction practices, traffic level, environmental conditions, etc. (*5–6*). In South Africa, cape seals have been reported to have excellent performance thanks to their ability to handle turning actions and cold temperatures without raveling. The expected life in the Western Cape Province is 10 years, although there have been reported instances of cape seals effectively protecting the pavement for up to 20 years (*7*).

Ozer at al. reported an average life extension of at least 7 years for cape seal treated sections and 11.4 years of service life before reaching the poor condition based

on the Illinois Department of Transportation condition rating survey (CRS) methodology (*8*). Sebaaly et al. reported an effective performance life of 3 to 3.5 years for slurry cape seals and 5 to 7 years for micro surfacing cape seals in northern Nevada (*9*).

In low-traffic-volume roads, cape seals can achieve long service lives at a relatively low cost. A research partnership established between the National Center for Asphalt Technology (NCAT) and the Minnesota Department of Transportation Road Research Facility (MnROAD) has been studying the field performance of various cape seal sections since 2012. More details on the NCAT–MnROAD partnership and the overall objectives of its pavement preservation research can be found elsewhere (*10*).

In current work, the three treatment variations evaluated include the following:

- Cape seal—Type II micro surface over chip seal
- Fibermat cape seal—Type II micro surface over Fibermat chip seal
- Scrub cape seal—Type II micro surface over scrub seal

The objective of this study was to evaluate the field performance of similar cape seal treatments applied to low-traffic roads in different climatic zones.

Methodology

To accomplish the objective, full-scale test sections were treated in two different roadways and their performance was monitored for an extended period.

Test Sections

Treatments were applied on two different roadways subjected to low traffic but with significantly different climatic conditions. Lee County Road 159 (LR-159) is located in Auburn, Alabama, which corresponds to a warm, wet-no-freeze environment. Conversely, County State Aid Highway 8 (CSAH-8) is near Pease, Minnesota, which is in the cold, wet-freeze climatic zone. LR-159 sections were treated in 2012, and CSAH-8 sections were treated four years later, in 2016. All sections remain in place after 10 and 6 years of service, respectively.

Both roadways are subjected to low traffic volumes, with an annual average daily traffic of approximately 500 vehicles per day. LR-159 sections are 100 ft in length and cover both travel lanes, while CSAH-8 sections are 528 ft in length and cover a single

lane. Climate conditions vary significantly between the two locations and have a direct effect on pavement performance and deterioration modes. In Auburn, the average annual high and low temperatures are 74°F and 53°F, respectively, with an average annual precipitation of 53 in. Conversely, Pease has an average annual high temperature of 53°F and an average annual low temperature of 31°F. Moreover, although the average annual precipitation is lower (29 in.), the area also experiences an average annual snowfall of 44 in (*11*).

Table 1 shows a summary of the materials used for each treatment and location. Materials were selected according to local climatic conditions and available aggregate sources. The aggregate gradations used for the chip or scrub seal layers had 100% passing the 3/8" sieve size. All micro surfaces were Type II with target emulsion rates of 12% and 13% for the LR-159 and CSAH-8 sections, respectively.

Data Collection

Field performance was measured periodically in terms of cracking (as a percent of the total lane area), average rut depth, and roughness [in terms of the International Roughness Index, (IRI)]. These three indicators can be used to evaluate the condition of the pavement based on the performance criteria established by the Federal Highway Administration, as shown in Table 2 (*12*).

TABLE 1 Treatment Description

Treatment Location	Cape Seal	Fibermat Cape Seal	Scrub Cape Seal
LR-159	Granite chip seal with CRS-2HP emulsion, limestone micro surface with CSS-1HP emulsion	Granite chip seal placed on fiber membrane with CRS-2L emulsion, limestone micro surface with CSS-1HP emulsion	Granite scrub seal with CMS-1P(CR) emulsion, limestone micro surface with CSS-1HP emulsion
CSAH-8	Granite chip seal with CRS-2P emulsion, granite micro surface with CQS-1HP emulsion	Granite chip seal placed on fiber membrane with CRS-2P emulsion, granite micro surface with CQS-1HP emulsion	Granite scrub seal with CMS-1P(CR) emulsion, granite micro surface with CQS-1HP emulsion

TABLE 2 Pavement Condition Categories (*12*)

Condition Rating	Cracking, %	Rutting, mm	IRI, in./mi
Good	< 5	< 5	< 95
Fair	5–20	5–10	95–170
Poor	> 20	> 10	> 170

Prior to treatment application, all LR-159 sections exhibited cracking and rutting in the "fair" category, and IRI in the "good" category. Conversely, the CSAH-8 sections fell mainly in the "good" category for all indicators, with the exception of the cape seal section which was in the "fair" category for IRI before treatment.

Once sections reach the "poor" condition, they may not be considered as candidates for pavement preservation and will likely require further intervention to restore serviceability and/or load-carrying capacity. The time required to reach this condition after treatment application may be used to assess treatment service life.

Findings

Current measured values for cracking, rutting, and IRI are shown in Figure 1. Due to the age difference between the two locations, LR-159 sections are also shown at year 6 for direct comparison to CSAH-8 sections.

Cracking

Cracking performance can be classified as "good" to "fair", as seen in Figure 1*a*. After six years, the LR-159 exhibited less cracking compared to the CSAH-8 sections. The northern region is mainly affected by thermal cracks, which due to their severity reappeared shortly after treatment. However, once these cracks were reflected, cracking progression slowed down. In the southern region, cracking appears primarily on the wheelpaths and has been increasing at a slow rate.

Currently all sections have cracking percentages under 9%, and, in general, older sections exhibit the most deterioration. The exception is the cape seal treatment, which has cracked to a higher extent in the cold location, despite having been treated more recently. The Fibermat cape seal exhibited the best cracking performance and both test sections remain in the "good" condition category.
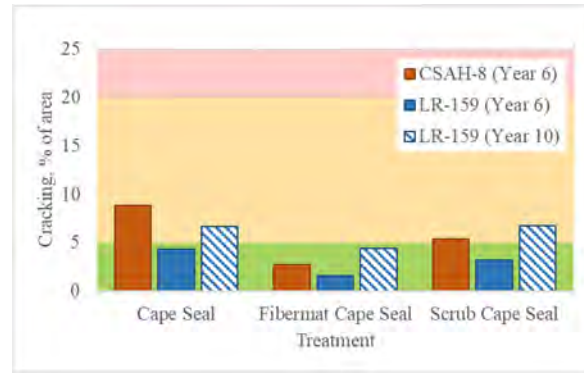
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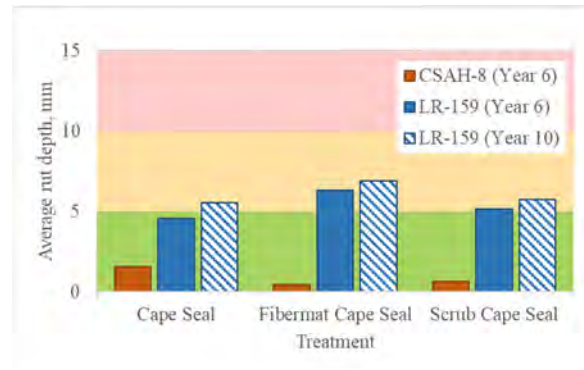
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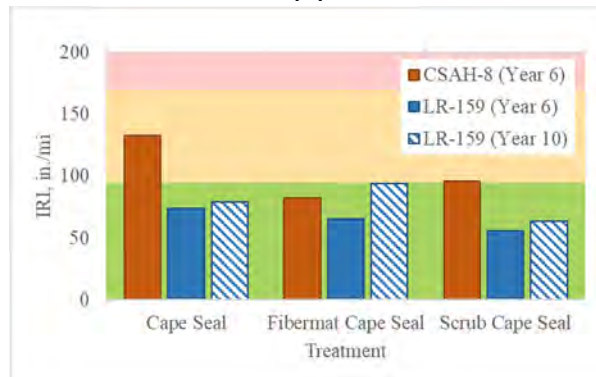
(c)

**FIGURE 1 Observed field performance in terms of
(*a*) cracking, (*b*) rutting, and (*c*) roughness.**

Rutting

Average rut depths are minimal in the CSAH-8 sections, with values under 2 mm. In LR-159, subjected to a warmer climate, rut depths are higher and by year 10 all sections have surpassed the "fair" condition threshold (Figure 1*b*). There is no clear trend between observed rutting performance and treatment type.

Roughness

In general, roughness is higher for the test sections located in the colder climate (Figure 1*c*), which relates to high severity thermal cracks present in the pavement prior to treatment application. Nonetheless, these sections remain in "good" to "fair" condition after six years of service. In the warmer location, all sections remain in "good" condition after 10 years of service.

General Condition

Over the analysis period, sections have maintained "good" to "fair" performance in terms of the MAP-21 indicators. The criteria establish that if all three indicators are "good" the overall condition of the pavement is "good," two or more indicators in the "poor" condition will result in an overall "poor" condition, and all other combinations are considered as in overall "fair" condition. At year 6 the CSAH-8 Fibermat cape seal and LR-159 cape seal had maintained an overall "good" condition. Since then, the LR-159 cape seal has been subjected to an additional four years of service and has fallen into the "fair" condition category. All other test sections are in "fair" condition. Figure 2 shows an overview of the current condition for all test sections.

Conclusion

This research has monitored the field performance of six cape seal test sections placed in two different climate regions. After being in place for several years, test sections have maintained "good" or "fair" condition in terms of cracking, rutting, and IRI. The treated pavements have been able to provide a satisfactory level of service and remain in place with no major issues. Based on the observed performance, these sections have reached service lives of at least 6 years in cold climate and 10 years in warm climate and are still far below any of the "poor" condition category thresholds.

Sections located in cold climate appear to be more susceptible to roughness deterioration. Sections in the warm climatic zone tend to develop more rutting and cracking. Treatments continue to be monitored, and further analysis is expected to quantify their life-extending benefit.

FIGURE 2 Overview of current test section condition: (*a*) Cape seal CSAH-8; (*b*) Cape seal LR-159; (*c*) Fibermat cape seal CSHA-8; (*d*) Fibermat cape seal LR-159; (*e*) Scrub cape seal CSAH-8; and (*f*) Scrub cape seal LR-159.

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Best Practices for Chip Sealing Low-Volume Roads in Ohio

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Many local agencies in Ohio construct chip seals using local work crews and application rates based on experience, or with contractors using local specifications or older version of the Ohio Department of Transportation (ODOT) specification, as newer specifications from 2002 have been difficult to implement for some. Consequently, success has been varied, with some excellent applications and some failures. The purpose of this research was to gain a better understanding of chip seal practices of the local agencies and identify best practices for low-traffic-volume roads. The goal of this research was to assess the current state of practice for chip sealing on county, township, and municipal-maintained roads. The objective was to develop a matrix of best practices for chip sealing low-volume roads in Ohio and design a study to aid in the future assessment of long-term performance creating protocols for data collection. As part of this study a literature review was completed, county and municipal engineers and township trustees were surveyed regarding their chip seal practices, and follow-up interviews were conducted with 11 agencies. Based on the information collected, a matrix of best practices for chip sealing on local roadways was developed, and a study to collect long-term performance was designed.

Introduction

In 2015, a workshop for Ohio's local agencies was held in Newark, Ohio, to discuss the design, rehabilitation, and maintenance of local roadways in Ohio counties. There are approximately 29,000 centerline mi (46,000 km) on the Ohio county road system. A survey of county engineers attending the workshop found approximately one-third of the roads under their jurisdiction are surfaced with chip seal (see Figure 1). Sargand, Mitchell, and Green (*1*) also reported the chip seal maintenance expenditure can be as high as 60% of the local agency's budget.

With the ODOT's 2002 edition of the *Construction and Materials Specifications* (CMS), Item 422, Chip Seal, was revised (*2*). The current version of the specification with polymer binder might be burdensome and costly for local agencies to adopt due to lack of inspection personnel and a required level of quality, which might not be necessary to achieve satisfactory performance on the local system. Many local agencies construct chip seals using local work crews and application rates based on experience or with contractors who use either local specifications or an older ODOT specification. Consequently, success has been varied, with some excellent applications and some failures. The purpose of the research was to gain a better understanding of chip seal practices of the local agencies and identify best practices for low-traffic-volume roads in Ohio. The research team created a synthesis of current practice through a review of the literature, a survey of Ohio county and municipal engineers and township trustees, and interviews of the local agencies responding to the survey (*3*). The report included a

Pavement surface, miles of pavement, percentage

18 counties and 1 city in Ohio completed survey
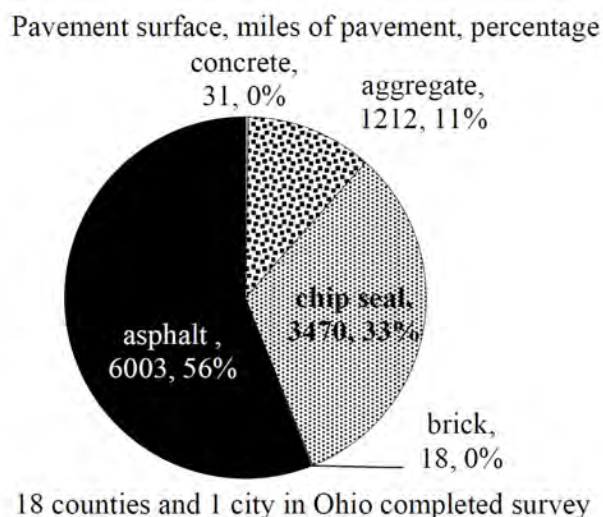
**FIGURE 1 Ohio County Road pavement surface types
by mileage and percent (*1*, Figure 2, p. 6).**

matrix of chip sealing best practices for Ohio and a plan to study the long-term performance of chip seals in the state.

Objectives

The goal of this research was to assess the current state of practice for chip sealing on county, township, and municipal-maintained roads. The objective was to develop a matrix of best practices for chip sealing low-volume roads in Ohio and design a study to aid in the future assessment of long-term performance, creating protocols for data collection. To fulfill the objectives, the following tasks were undertaken:

1. Create a synthesis of current practice.
2. Develop a matrix of chip sealing best practices.
3. Design a study of long-term performance that could be conducted using data collected by local agency personnel.

Synthesis Of Current Practices

Around the country, researchers have worked toward quantifying cost–benefits of preservation treatments. A study in Ohio found the pre-chip seal pavement condition rating (PCR) influenced the average increase in PCR after construction; in other words, the existing condition of the pavement will impact how much improvement is seen in performance by applying a chip seal (*4*). The PCR is a method used by ODOT to assess the condition of pavement. A score of 100 indicates new pavement, and deductions are made based on the types, severity, and prevalence of defects observed, including rutting, cracking, crack sealing, raveling, D-cracking, etc. The rating team drives the pavement at 40 mph (60 km/h) with a second pass with stops at every mile (1.5 km), where close inspection is conducted of a 100-ft (30-m) segment from the road shoulder. A PCR of 55–65 indicates a fair-to-poor condition, meriting an overlay or rehabilitation. Chip seals provide the most benefit when applied to pavements with a PCR in the range of 66–80 and have a service life up to seven years (*5*). A study in Kansas showed chip seals were effective in preserving roads and had the lowest annual cost compared to other similar surface treatments such as thin overlays, Nova-Chip, and modified slurry seal (*6*). Another study in Pennsylvania showed the annual cost of chip seal has proven to be less than half the cost of slurry seal and less than 20% the cost of thin hot-mix asphalt overlays. Another study conducted by Wang et al. quantified cost–

benefit of various preservation treatments in Pennsylvania with chip seals and micro-surfacing having the highest cost–benefit for low-volume roads (with average daily traffic < 2,000) (*7*).

A survey conducted by Gransberg and James (*8*) identified commonalities between agencies that achieve excellent chip seal performance. Many of these agencies use chip seals as a preventative maintenance (PM) tool about five years after construction, and they expect to achieve six-year service life. Other key similarities between agencies achieving excellent chip seal performance include the following:

1. Using formal design procedures as McLeod, modified Kearby, or locally empirically developed procedures based on input parameters from field surface conditions (*9, 10*).
2. Using modified binders with polymers, with crumbled rubber as the most common.
3. Using pavement condition rating as a base for selecting chip seal candidates.
4. Selecting roads with moderate to low distress level and structural stability rated as good to fair. Chip seals are not placed on pavements exhibiting structural distresses such as potholes, cracking in the wheel path, etc.
5. Using chip seal as a PM technique rather than repair/corrective technique.
6. Using quality control, quality assurance, and performance monitoring programs.

Minnesota's seal coat handbook and well-developed specification were the result of chip seal research. By 1997 there was a decline in the use of chip seals due to repeated failures (*11*). The Minnesota Department of Transportation worked in partnership with the Minnesota Local Road Research Board to modify their chip seal program. This study helped implement a chip seal design, required aggregate characterization, changed how chip seal bid items were paid, and enhanced training efforts (*11*). The Minnesota chip seal study revitalized the chip seal program, and most chip seals performed better than expected (*11*). The following points summarize the important considerations (*11*):

1. Use a rational design of chip seal;
2. Make sure the pavement surface is clean;
3. Use the highest-quality specified materials;
4. Apply the proper amount of asphalt binder;
5. Place only a single layer of chips;
6. Minimize the distance between the distributor and the chip spreader;
7. Place chips before the emulsion starts to break;

8. Operate a minimum of three pneumatic rollers with a speed under 5 mph;
9. Control traffic speed on the fresh chip seal;
10. Sweep as soon as possible;
11. Quality does not cost—it pays; and
12. Remember that details count.

Most DOTs in the United States follow the Norman McLeod method developed in the 1960s and adopted by the Asphalt Institute (*8*). The McLeod design considers the depth of aggregate embedment into the binder, aggregate loss, and bleeding (*9*, *12*). McLeod found that chip seals with an embedment depth of less than 50% are more prone to aggregate loss. Chip seals with embedment depth of more than 70% tend to have surface bleeding. Therefore, McLeod developed a formula to calculate binder application rate, ensuring that there is sufficient binder to hold the aggregates in place but not so much as to overly reduce the macro texture. The McLeod chip seal design approach developed correction factors to the application rates to account for field conditions. The correction factors include aggregate-related characteristics, traffic volume, existing pavement condition, and embedment depth.

The second widely used method is the Kearby method, which was adopted by Texas. Kearby developed a nomograph to be used as an easy tool to calculate the binder application rate (*10*). The nomograph considers percent of voids, desired embedment, and size of aggregate/mat thickness. Aggregate properties required for the Kearby method are unit weight, specific gravity, percent of voids, gradation analysis, and use of a square-yard-test-board to determine the effective mat thickness and spread ratio (*10*). However, the Kearby method has limitations, for example, traffic and aggregate toughness properties are not considered in the approach, however, these are addressed in an expanded method by Epps et al. (*13*). Yet, the Kearby method includes many considerations for the design process that are still in use today, such as uniformly graded aggregate, flat and elongated particles limitations, and adjustments for embedment based on surface texture and traffic volumes.

The New Zealand Transportation Authority has continually studied and refined their chip seal design method. This method is currently practiced in New Zealand. The design pays close attention to existing pavement surface texture and binder application rate. Pavements with large texture depth require higher binder rates. Texture depth is usually measured using the sand circle test in accordance with TNZ Specification T/3, which is similar to the ASTM sand patch test (*14*). Moreover, the New Zealand method considers traffic factors and its effect on aggregate orientation and embedment into the substrate.

The condition of the pavement prior to the application of a chip seal affects the seal's overall performance (*15*). Treatments should generally be applied to pavements in good condition to maximize the treatment effectiveness (*16*). Studies commonly report negative experiences with chip seal performance and reduced pavement life extension when chip seals are applied to pavements in poor condition (*17–21*). Chip seals are sometimes used as a "holding strategy" to keep pavements from deteriorating below an acceptable service condition. Sometimes holding strategies are required when budgets are tight and engineers are left with limited alternatives (*22*). In this case, roadway distresses often require pre-treatment such as patching or crack sealing before applying the surface treatment (*23*). The following information related to pavement pre-existing condition should be gathered to evaluate a roadway's suitability for chip seal application (*24*):

- Pavement surface type and/or construction history;
- An indication of the functional classification and traffic level;
- At least one type of condition index, including distress and/or roughness;
- More specific information about the type of deterioration present, either in terms of an amount of load-related deterioration or the presence of a particular distress type; and
- Geometrics, to indicate whether pavement widening or shoulder repair should also be required.

The literature emphasizes using high-quality chip sealing materials on good pavement candidates to achieve the best performance (*15*, *18*, *25*, *26*). Important material characteristics include aggregate size, shape, gradation, cleanliness, and quality of binder. The purpose of the asphalt binder is to adhere to the roadway substrate and to bond/embed the chip seal aggregates. Binder application must be applied uniformly and in sufficient amounts and ideally be based on a chip seal design method. Typically, hot applied asphalts or emulsions and used. Polymer-modified asphalt is used for a majority of chip seals. Other binder additives may include viscosity modifiers, anti-stripping additives, coating improvers, and emulsion stabilizers.

Ohio Local Agency Survey

County, township, and municipalities in Ohio were surveyed to obtain details about the use and performance of chip seals within their jurisdiction and to select agencies for follow-up interviews. An online survey was developed using the Qualtrics survey platform. A link to the survey, as well as hard copies for local agencies without access to the internet, was distributed by the County Engineers Association of Ohio, the Ohio Township Association, the Ohio Municipal League, and the Ohio Local Technical Assistance Program through their respective newsletters and/or e-mail lists. Responses were received from 41 counties, 25 townships, 11 cities, 2 villages, ODOT, and 1 contractor, for a total of 81 responses. The survey questions and complete responses are contained in the project report (*3*, Appendices B and C, pp. 72–95).

The responding cities had an average network of 233 centerline miles, the counties an average network of 339 centerline miles, the townships an average network of 40 centerline miles, and the villages an average network of 28 centerline miles. The percentage of roads with a chip seal surface ranged from 0% for villages to 46% for counties. In addition, roads with an asphalt surface, which could be a candidate for chip seal, ranged from 34% for the township to 82% for the cities. Of the 79 local agencies responding, 65 agencies, or 82%, indicated they use chip seal as a treatment for their pavements. By jurisdiction, 100% of the responding counties, 76% of the townships, 45% of the cities, and 0% of the villages indicate they use chip seal as a pavement maintenance technique. The cities, counties, townships, and villages that do not use chip seal were excluded from the analysis.

More than half of the agencies apply chip seal as part of a routine preventive maintenance plan. Other high-ranking reasons for using chip seal include to prevent water intrusion and to provide a wearing surface.

The respondents indicated the most common chip seal being used is single chip seal, used by 62 (78%), of the agencies. The second most common chip seal type being used is double chip seal, by 26 agencies (33%), followed by Cape seal by 6 agencies (8%), and fog seal by 3 agencies (4%).

A majority of the responding cities (80%) and counties (76%) fully or partially use ODOT's *Construction and Materials Specifications* (CMS) for chip seals, while slightly less than half (44%) of the townships use ODOT's CMS. More than half the agencies apply chip seals as part of a routine preventive maintenance plan. Of the responding agencies, 40 use contractors and 28 use in-house crews to construct chip seals. The results of the survey also indicate in-house and contractor chip seal projects have approximately the same expected life span, with a majority having a life of 4–7 years, as shown in Figure 2.
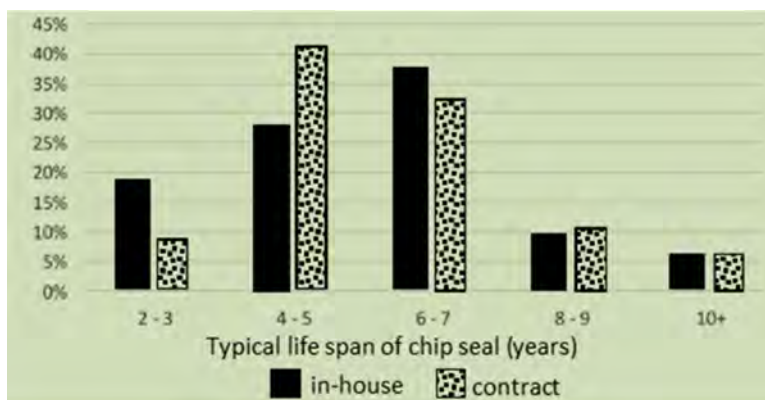
FIGURE 2 Ohio local agency survey responses regarding service life of chip seals.

The most critical factors determining service life, as identified by local agencies, were original condition of the road (32%), the underlying structure (25%), and the quality of the chip seal (22%), validating the importance of scheduling the project at the right time, preparing the project for chip sealing, and the quality control/quality assurance during construction.

Based on the survey results, the most prevalent causes of chip seal failures include dirty or dusty aggregate, weather conditions during construction, and snow plow damage. The survey results are discussed in more detail in the project report and helped shape the matrix discussed later (*3*).

Ohio Local Agency Interviews

A sample of agencies were selected from the survey response for follow-up with detailed interviews. Only agencies that construct chip seals were selected. Agencies were selected to obtain a geographic distribution as well as a distribution of agencies that construct chip seals by contract and with in-house crews. The three cities and eight counties shown in Table 1 were interviewed.

The research team traveled to the agency's location for the interview. During the interview, the research team asked the agency for a list of projects recently constructed. After the interview, the research team would visit projects on the list as time permitted to view and document the chip seal condition.

Three of the agencies routinely construct double chip seals, seven routinely construct single chip seals, and one will routinely construct a double chip seal on routes with "severe problems" and single chip seals elsewhere. One county that routinely constructs a double chip seal does so based on the 5-year evaluation of a test section

TABLE 1 Ohio Local Agencies Selected for Follow-up Interviews, with Centerline Mileage of Road Network and Chip Seal Portion

Agency	Total Road Surface (mi)	Total Road Surface (km)	Chip Seal Surface (mi)	Chip Seal Surface (km)	In-house or Contract	Types of Chip Seal
City of Circleville	60	97	2	3	Contract	Double
City of Cleveland Heights	128	206	32	51	Contract	Single
City of Toledo	1125	1811	146	235	In-house	Single
Butler County Engineer	278	447	28	45	Contract	Double
Columbia County Engineer	169	272	113	182	In-house	Both
Darke County Engineer	521	838	104	167	Both	Single
Henry County Engineer	406	653	325	523	In-house	Single
Knox County Engineer	400	644	200	322	Contract	Both
Licking County Engineer	421	678	156	251	In-house	Single
Tuscarawas County Engineer	466	750	280	451	In-house	Both
Washington County Engineer	341	549	143	230	Both	Single

that showed double chip seal has a longer service life. Two of the agencies are trying to build structure with the double chip seals. One agency uses the double chip seal to address bleeding.

Six agencies use chip seal as preventive maintenance, one as reactive, and four for both preventive and reactive. Deviations from the common chip seal treatment include the following:

- One of the cities is considering the use of a Cape seal.
- Two agencies, both cities, fog seal newly constructed chip seals. One county indicated some townships within the county fog seal newly constructed chip seals.
- The same city is also considering the use of No. 9 (9.5-mm) aggregate instead of No. 8 (12.5-mm) aggregate because they believe smaller aggregate would reduce the amount of binder needed.
- One county has been using the chip seal as a "poor man's" stress absorbing membrane interlayer and placing a thin hot-mix overlay on top. County crews were used to place the chip seal. A 1-in. (25-mm) asphalt overlay is then placed by contract.

- One city plans to mill the surface, use double chip seal, and surface with microsurfacing as a replacement for a 2-in (50-mm) mill-and-fill.

Other pavement preservation treatments used include the following:

- Crack sealing is used by seven of the 11 agencies interviewed. One of the counties that does not crack seal stated the treatment is labor intensive and easily removed by snow plows during snow and ice control.
- Three counties have tried fog seal as a pavement preservation treatment. They did not find the treatment to be cost effective.
- Three agencies use microsurfacing. One agency plans to use microsurfacing in the future. Two counties have tried microsurfacing and do not plan to use it in the future due to cost effectiveness.
- Only one county has used slurry seal.
- All but one agency uses thin asphalt overlays. The one agency that does not use thin overlays is in northern Ohio. That agency indicated the thin overlays do not perform well during the harsh winters typical of the area. Overlay thickness for nine of the agencies ranges from 1-in. (25-mm) to 1.5-in. (38-mm) thickness, typically on a scratch leveling course. One county's overlay thickness ranges from 1.25 in. (32 mm) to 3 in. (76 mm).

While no agency had a policy for placing a chip seal on an aggregate road, some reasons given for chip sealing aggregate roads in the past include an increase in daily traffic, elimination of a pothole problem, elimination of the "checkerboard" effect resulting from sealing in front of homes to reduce dust, and reduction of maintenance cost. Four of the agencies had no aggregate roads. One county was converting chip sealed roads to aggregate roads due to lack of funding.

Typical advantages of chip seals cited were

- Low cost (approximately 1/3 cost of hot-mix overlay);
- Economical, can rehabilitate more miles than with hot-mix overlay;
- Construction with agency crews;
- Extended pavement life;
- Pavement flexible over a range of temperatures;
- Pothole prevention;
- Pavement waterproofing, crack sealing;
- More skid resistant in winter; and

- Ability for cracks to "heal" during warm weather.

Typical disadvantages of chip seal cited were

- Public acceptance, complaints, viewed as a lower level of treatment;
- Bleeding;
- Dust (fine particles sent airborne by passing traffic);
- Loss of aggregate, which can also damage cars;
- Large commitment of workforce if constructing with agency crew;
- More difficulty applying and maintaining pavement markings;
- Rough texture (especially in urban areas);
- Less life than a thin hot-mix overlay;
- Noisy (from passing traffic);
- No improvement in ride quality;
- Appearance (streaking, etc.);
- Not bicycle friendly;
- One city assesses the residents on a street when they chip seal, residents not happy with the assessment; and
- Not suitable for high-traffic, high-speed roads.

The information from the literature search, survey, and interviews was collated into a lengthy matrix of best practices, presented in the project report (*3*, pp. 11–25). A sample excerpt is presented in Table 2 (the full matrix is available from the authors). The matrix has the following heading items. For each one, considerations are noted, including those from literature and those from surveys and interviews; where warranted, specifications are proposed.

- Aggregate selection/properties;
- Binder selection/properties;
- Pavement management/project selection;
- General;
- Pre-project;
- Surface preparation;
- Construction;
- Curing; and
- Pre-application.

TABLE 2 Example Excerpted from Matrix (3, p. 11)

Item		Considerations	Specifications
Aggregates Selection/Properties	**Literature Search**	• Washed, hard, durable, clean, free from coatings and deleterious material • Washed limestone/dolomite • Binder-compatible • Damp for emulsions • Dry for hot binders • Electrostatic chip testing before design • Preferred lightweight aggregate to minimize vehicle damage • Otta seals allow the use of lower-cost, well-graded local gravel, which typically is less costly than single-size aggregate used for chip seal.	• The quantity of chip seal aggregate per square yard shall be agreed on with the project manager. • Only one source of aggregate shall be used and shall conform to the following gradations. **Gradation Table: Chip Seal Aggregate** | Sieve Size	% Passing
---	---
1/2" (13 mm)	100
3/8" (19 mm)	90–100
#4 (4.75 mm)	5–25
# 8 (2.36 mm)	0–10
#16 (1.18 mm)	0–5
#200 (0.075 mm)	1.5 max. washed value	 • Limited flaky particles (< 12%) → ASTM D4791 • Abrasion < 20% → AASHTO T96 • 100% fractured faces
	Survey/Interviews	• Washed slag aggregate—lighter than limestone & gravel; less dust; limited availability (northern Ohio) • Washed limestone/dolomite—good adhesion, reduces bleeding from underlying layer • Washed gravel—lowest cost aggregate type, performance improved if at least 80% crushed • Smaller size aggregate requires less binder and does less damage due to loose aggregate, but more susceptible to bleeding • Larger size aggregate/double chip seals reduce reflective cracking • Use pre-coated aggregate to promote ionic bond with emulsion.	

Item		Considerations	Specifications
Binder Selection/Properties	Literature Search	• Affected by surface temperature – If high → Asphalt binders – If low → Emulsions • Polymer-modified binders for performance • Otta seals use a soft asphalt binder to allow the binder to migrate into the gravel matrix. A benefit of using the soft asphalt is cracks tend to "heal" during warm weather.	• RS-2 emulsified binder • The specification for binders should be in accordance with the material properties and test methods → ASTM, AASHTO, and ODOT. • The emulsion standing undisturbed for a minimum of 24 h shall show no white, milky separation but shall be smooth and homogeneous. • The emulsion shall be pumpable and suitable for application through a distributor.
	Survey/Interviews	• MC 3000 provides a more flexible chip seal that "heals" during the summer but is more likely to flush or bleed. • HFRS provides good adhesion, especially with larger aggregate. • Binders with polymers provide good stone retention, extending the life of the chip seal an estimated 2 to 3 years. • MWS has performed well with respect to bleeding and aggregate bond.	• EPA limitations on the time of year MC 3000 can be used • Aggregate must be dry when using MC 3000

- Post application
- Preliminary responsibilities
- Distributor
- Aggregate spreader
- Rollers
- Sweeping
- Traffic control
- General weather conditions

Long-Term Performance Study Protocol

A protocol was developed to guide local agencies on collecting performance data before, during, and annually after chip seal placement. It is anticipated data collected as part of the long-term performance study will enable local agencies to more accurately estimate the life of their chip seals and improve their chip seal programs by better understanding the effect of pretreatment conditions, construction quality, and materials on chip seal performance. Also, documenting activities before and during construction will enable local agencies to evaluate the implementation of identified best practices.

Data pertaining to maintenance activities, surface texture, and existing pavement condition are to be collected before the placement of the chip seal. Performance monitoring is to be conducted annually after chip seal placement for a minimum of 3 years, although ideally performance monitoring would be conducted until the end of the service life had been reached. Performance is monitored by documenting surface texture, pavement distresses, and maintenance activities. In addition to data collection before and after chip seal construction, the protocol includes documentation of pertinent information collected during construction of the chip seal.

This protocol serves as a uniform data collection procedure for local agencies in Ohio to monitor performance of chip seals. To support this protocol, which is further described in Appendix G of Green et al. (*3*, pp. 212–226), a spreadsheet provides Excel worksheet forms to collect the following:

1. Maintenance activities prior to chip seal,
2. Pavement condition prior to chip seal,
3. Daily construction and materials information form,
4. Pavement/chip seal performance monitoring survey (to be completed annually), and
5. An application rate design check worksheet.

Table 3 shows a form for recording pavement condition before applying ship seal and Table 4 shows a form with a checklist for recording daily construction work. The complete spreadsheet is available from the authors.

To organize all project information, each chip seal construction project should have an electronic file to store important construction and performance information in an easily accessible manner. The file should include the following items:

- Project information (number, county, route);
- Date, air temperature, pavement temperature, and humidity;
- Binder type used and temperature;
- Beginning and ending stations;
- Binder tests results to qualify the asphalt binder/asphalt emulsion;
- Aggregate properties (gradation, moisture content, and station location);
- Aggregate tests results required to qualify aggregates;
- Target binder and aggregate application rates;
- Actual application rates using yield checks on binder and aggregate (minimum three per day);
- Construction field notes documenting any concerns; and
- Field data collection sheets developed in this study.

Before chip seal placement, any maintenance activities conducted, such as partial-lane- or full-lane-width repairs, full-depth pavement repairs, pothole repairs, crack sealing, tree trimming or brush removal, berm cutting, culvert repairs, grader patching, or ditch repairs should be documented. The pavement condition of the existing roadway should be determined before chip seal placement by documenting pavement distresses (including severity and extent) following the Long-Term Pavement Performance Program's *Distress Identification Manual for the Long-Term Pavement Performance Program* (*27*). The surface texture of the existing pavement should be determined following the sand circle test TNZ T/3 (*14*). Traffic volumes on chip sealed roads are generally low in Ohio, making a sand circle test feasible with some traffic control, though where volumes are too high, a laser measurement could be made (*28*).

During construction of the chip seal, daily construction reports should be completed to document general information like weather conditions and work area for the day. Also, yield checks for binder and aggregate application rates should be conducted and recorded, and aggregate should be sampled and tested for gradation daily. A spreadsheet tool was created to estimate binder and aggregate application rates to check the design rates based on details for the specific roadway conditions.

TABLE 3 Form for Recording Condition of Pavement Prior to Chip Seal (adapted from *3*)

Each chip seal studied should have at least three 500-ft test sections for statistical comparisons to be developed. A form is filled out for each 500-ft chip seal section. Ideally, a form should also be completed for each section prior to chip seal construction to document the pavement condition and maintenance work leading up to chip seal construction.

General Information			Sand Circle Test—Outer Wheel Path (O.W.P)									
Route number	Ohio Route		Outer Wheelpath (O.W.P)	Station	Milepost	Starting Odometer Reading	Odometer Reading at Sand Patch Location	Rut Depth (mm)	d1(mm)	d2 (mm)	MTD (mm)	Average MTD (mm)
Test section name			O.W.P. #1									
Rater name			O.W.P. #2									
Weather condition (sunny/cloudy/partly)			O.W.P. #3									
Ambient temperature (°F)			O.W.P. #4									
Pavement temperature (°F)			O.W.P. #5									
Wind speed (mph)			Sand Circle Test—Between Wheel Paths (B.W.P)									
Humidity (%)			Between Wheel Paths (B.W.P.)	Station	Milepost	Starting Odometer Reading	Odometer Reading at Sand Patch Location	Rut Depth (mm)	d1(mm)	d2 (mm)	MTD (mm)	Average MTD (mm)
Other notes:			B.W.P. #1									
			B.W.P. #2									
			B.W.P. #3									
			B.W.P. #4									
Pavement Distress Codes and Units			B.W.P. #5									
Distress/Activity	Code	Unit	Pre Chip Seal Repair Locations and Pavement Distress Survey <u>Before</u> Chip Seal									
Transverse cracking	1	ft.	Station (for Longitudinal Distresses: Station-to-Station)	Milepost	Starting Odometer Reading	Odometer Reading at Distress	Measurements	Area/Length/Unit	Severity (High/Med/Low/NA)	Notes		
Long. cracking	2	ft.										
Fatigue cracking	3	ft²										
Pothole	4	ft.										
Patching	5	ft²										
Bleeding	6	ft²										
Aggregate loss	7	ft²										
Raveling	9	ft²										
Other	10	Specify										
Pre–Chip Seal Repairs												

Partial-lane-width repair	11	yd²										
Full-lane-width repair	12	yd²										
Full-depth pavement repair	13	yd²										
Pothole repair	14	yd²										
Crack sealing	15	% length										
Tree trimming, brush removal	16	% length										
Culvert replacement	17	number										
Other	18	Specify										
Other	19	Specify										
Other	20	Specify										
Other	21	Specify										
Other	22	Specify										

Please verify the following by checking the boxes below:			**Document any needed or completed corrective work and location**
Finished surface has minimal tears and binder streaking.	☐		
Joints appear neat and uniform without buildup, uncovered areas, or unsightly appearance.	☐		
Longitudinal joints have less than a 2 inch (50 mm) overlap on the adjacent passes.	☐		
Transverse joints have no more than 0.25 inch (6.5 mm) difference in elevation across the joint as measured with a 6 foot (2 m) straightedge.	☐		
Chip seal edge is neat and uniform along the roadway lane, shoulder, and curb lines.	☐		
Chip seal edge has no more than 2 inch (50 mm) variance in any 100 feet (30 m) segment along the roadway edge or shoulder.	☐		
Surface patterns including alternate lean and heavy lines (ridges or streaking over the surface) are minimal to non-existent	☐		
Bleeding/flushing (excess binder on surface, not subject to wearing off quickly) is minimal to non-existent	☐		
Loss of cover aggregate (patches or lines of aggregate lost from surface) is minimal to non-existent	☐		

TABLE 4 Form for Recording Daily Construction Work (adapted from *3*)

Obtain and label a binder sample from distribution truck. Test one sample from the aggregate spreader box randomly during the day (221 lb). Yield checks should be done three times per day. Aggregate testing should include one sample from spreader box at production start, one random sample during the day, and any other samples when directed by the engineer. Aggregates not meeting gradation requirements should be rejected.

General Information					Gradation Requirements		
Project No., County, Route	Ohio Route						
Inspector name					Gradation Requirements		
Date				No. 4 (5-mm) sieve from JMF	±5%		
Air temperature (°F)				No. 8 (2.5-mm) sieve from JMF	±3%		
Pavement Temp (°F)				No. 200 (0.075-mm) sieve from JMF	±1%, 2.05% upper limit		
Humidity (%)				Aggregate Moisture	4% max. for aggregates with absorption > 2%,		
Binder Temp (°F)				Content (by dry weight)	3% max for aggregates with absorption ≤ 2%		
Other Notes:							
Chip Seal Date and Location							
Date /Time	Begin Station	End Station					
Yield Checks							
Target Aggregate Application Rate (lb/yd²)		Yield Check 2	Yield Check 3				
Target Binder Application Rate (lb/yd²)							
Yield checks on binder (min 3 per day)				Tolerance: ±0.02 gal/yd²			
Aggregate yield check (min 3 per day)							

Aggregate Station 1		Aggregate Station 2		Aggregate Station 3	
Length		Length		Length	
Width		Width		Width	
Total Area		Total Area		Total Area	
Moisture Content		Moisture Content		Moisture Content	
Sieve Size	% Passing	Sieve Size	% Passing	Sieve Size	% Passing
1" (25 mm)		1" (25 mm)		1" (25 mm)	
3/4" (19 mm)		3/4" (19 mm)		3/4" (19 mm)	
1/2" (12.5 mm)		1/2" (12.5 mm)		1/2" (12.5 mm)	
3/8" (10 mm)		3/8" (10 mm)		3/8" (10 mm)	
#4 (5 mm)		#4 (5 mm)		#4 (5 mm)	
#8 (2.5 mm)		#8 (2.5 mm)		#8 (2.5 mm)	
#16 (1.2 mm)		#16 (1.2 mm)		#16 (1.2 mm)	
#30 (0.6 mm)		#30 (0.6 mm)		#30 (0.6 mm)	
#50 (0.3 mm)		#50 (0.3 mm)		#50 (0.3 mm)	
#100 (0.15 mm)		#100 (0.15 mm)		#100 (0.15 mm)	
#200 (0.075 mm)		#200 (0.075 mm)		#200 (0.075 mm)	

Lastly, data like that collected prior to the placement of the chip seal should be collected on an annual basis to monitor performance. Distresses, including the severity and extent, should be documented as well as the surface texture of the chip seal using the sand circle test. Additionally, maintenance activity conducted on the chip seal should also be documented.

Conclusions

The need to effectively manage the pavement network with limited funding has led to the widespread use of chip seals by local agencies in Ohio. Implementation of best practices as identified in this project has the potential to result in significant savings. The literature search conducted for this study identified best practices that are applicable not only to local agencies in Ohio, but also for agencies across the country and internationally. The best practices identified from the local agency survey and interviews are most applicable in Ohio.

Based on the findings of this project, the following conclusions can be drawn:

- Chip seals are typically used by local agencies in Ohio for these purposes:
  – As a reactive, stopgap treatment to provide a serviceable pavement until funds for a hot-mix asphalt overlay are available or
  – As a preventive treatment for a class of road, i.e., low traffic volume, in the local agency's network.
- There is no single right combination of aggregate and binder type and application rates for all local agencies in Ohio. Through trial and error, each agency has developed a chip seal design that meets their needs. Some local agencies were willing to accept lower quality material, and therefore decreased performance, to reduce cost, but still provide a pavement acceptable to the public.
- However, the research team did observe issues, such as excessive aggregate, bleeding, etc., that may be resolved with a review and revision of design and/or construction techniques.

Practices common to most of the agencies surveyed or interviewed include

- Combine all sections in a county (include township projects if the township is willing) into one project.

- Let contract projects in the spring, when contractors are trying to fill their schedule and may bid a lower price.
- Prepare the pavement and roadway prior to the chip seal.
    – Repair localized structural (base) failures.
    – Repair potholes.
    – Crack seal the pavement.
    – Remove tree limbs and brush that shade the road and prevent evaporation of moisture.
    – Cut berm and clean ditches to provide surface drainage.
    – Replace failed culvert.
    – Restore cross slope with scratch course or grader patch where necessary.
    – Cover manhole covers, catch basins, valve boxes, etc., with a debonding material.
- Use washed, hard, durable aggregate.
- Maintain a steady pace during construction; only travel as fast as the slowest machine in the chip seal operation.
- Monitor binder and aggregate usage during construction, adjust as needed.
- Use experienced inspectors.

Recommendations

The best practices matrix in the project report (*3*, pp. 10–25) identifies best practices found in the literature and/or used by local agency personnel. Many of the practices can be implemented with minor changes to current procedures or specifications. The scope of the research did not include collection of traffic, design, and construction data; sampling and testing of materials; application rates; performance history; etc. Therefore, the information gathered from the survey and interviews are subjective, with limited verification by site visits. The research team recommends the identified best practices be adopted by local agencies as a specification, policy, or procedure after local verification.

Excessive aggregate or binder was observed on several projects, which could be an indication of incorrect application rates. The research team recommends that the local agencies consider a formal design procedure. The design procedures discussed in Appendix A of Green et al. (*3*, pp. 36–72) require a minimal investment of time and money and could result in savings in materials cost, which would easily offset design costs. The local agencies should consider sharing the cost and use of equipment

needed to design chip seals like the hard ball test (ball penetrometer test) equipment (*29*, *30*), sieves and sieve shaker, flakiness index thickness gage, etc.

With regard to the bleeding and flushing, sandwich seals have been effectively used by New Zealand to correct bleeding and flushing of chip seals. A sandwich seal consists of placing a layer of aggregate chips directly on the bare pavement, topped with binder and a second layer of smaller aggregate chips (*31*, pp. 60–61). Consideration should be given to evaluating the use of sandwich seal to rehabilitate chip seals with a binder rich surface.

Consideration should be also given to evaluating the use of Otta seals in areas where the cost of single-size aggregate used in chip seals costs significantly more than locally available well-graded gravel aggregate. Otta seals originated in Norway (*32*, *33*) and have been successfully placed internationally, including Nepal (*34*) and Botswana (*35*, *36*).

A plan for the long-term monitoring and evaluation of chip seals was provided in Appendix G of the report (*3*, pp. 212–226). The research team recommends that local agencies collect and record construction and annual performance data on all, or a sample of, chip seals constructed in their jurisdiction. The research team recommends the monitoring continue for the service life of the chip seal. It is recommended that agencies use the collected performance data to evaluate the life of their chip seals. In doing so, agencies will have more accurate information to use for the planning and budgeting of chip seals as part of their pavement management program. Agencies may be better informed to determine when to apply a chip seal based on existing pavement conditions and have a more accurate estimate of the life of the chip seal. It is recommended agencies evaluate the information collected to also determine the effect of implementing best practices recommended herein.

Acknowledgments

Author Contribution Statement

The authors confirm contribution to the paper as follows: study conception and design, Shad Sargand, Roger Green, Ashley Buss; data collection, Roger Green; analysis and interpretation of results, Shad Sargand, Roger Green, Ashley Buss, Minas Guirguis; draft manuscript preparation, Roger Green, Andrew Russ. All authors reviewed the results and approved the final version of the manuscript.

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Clinton County Pilot Demonstration of Cape Seals for Pavement Preservation

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Cape seals have been applied in several different countries as well as in the United States for many years and are considered a proven pavement preservation strategy. Seal coats and microsurfacing are the two basic components of a Cape seal, and both treatments are promoted as pavement preservation techniques by a variety of agencies, including the Federal Highway Administration (FHWA). Currently, local public agencies mainly use other pavement preservation treatments such as slurry seals, crack sealing, chip seals, and other thin surface treatments. Traditional pavement preservation practices by counties in Iowa would be crack sealing and filling, mill-and-fill overlays, and cold-in-place recycling (CIR) with hot-mix asphalt (HMA) overlay. Very little seal coating and few microsurfacing treatments have been applied on roads in good condition in Iowa.

As part of Iowa's State Transportation Innovation Council effort to promote the Every Day Counts (EDC) Pavement Preservation When, Where, and How, Clinton County is constructing a Cape seal (double chip seal with microsurfacing wearing surface) as a pavement preservation method on Clinton County Road F-12. The project is being funded through an Advanced Innovation Deployment Grant from FHWA and the US Department of Transportation (USDOT).

The proposed construction project was let for construction in the spring of 2022 and was built in September 2022. The Cape seal is being proposed as a new pavement preservation and holding strategy for lower-traffic-volume pavements that are still structurally sound. Cape seals were identified as a pavement preservation innovation that is not currently in use in the State of Iowa. The goal of this project is to demonstrate a "middle-of-the-curve" type of pavement preservation method to prolong the need for major rehabilitation.

Methodology

According to the U.S. Forest Service, Cape seals can extend the life of a pavement from 6 to 8 years and increase the life of a chip seal by enhancing bonding of the chips and by protecting the surface (*1*). Cape seals will provide a pavement preservation tool for those roads that are a little farther down the condition curve and require a more substantial preservation treatment method.

Pavement Preservation (When, Where, and How) is an FHWA EDC-4 initiative. As part of Iowa's implementation of Pavement Preservation, this project will demonstrate using Cape seals as a pavement preservation treatment and as a holding strategy for pavements. This project will help demonstrate a preservation tool to fill the gap for roads that are past the condition necessary for "top-of-the-curve"–type treatments (fog seals, slurry seals) and not yet deteriorated to the point of implementing "bottom-of-the-curve"–type treatments (CIR, hot-in-place recycling).

This project also addresses the USDOT Technology and Innovation Deployment Program goals (*2*):

1. Significantly accelerate the adoption of innovative technologies by the surface transportation community;
2. Provide leadership and incentives to demonstrate and promote state-of-the-art technologies, elevated performance standards, and new business practices in highway construction processes that result in improved safety, faster construction, reduced congestion from construction, and improved quality and user satisfaction;
3. Construct longer-lasting highways through the use of innovative technologies and practices that lead to faster construction of efficient and safe highways and bridges;
4. Improve highway efficiency, safety, mobility, reliability, service life, environmental protection, and sustainability; and
5. Develop and deploy new tools, techniques, and practices to accelerate the adoption of innovation in all aspects of highway transportation.

The Cape seal pilot demonstration project is located in Clinton County, 5 mi northeast of DeWitt, Iowa (shown in Figure 1). The pilot section on Clinton County Road F-12 is 4.4 mi long and was originally constructed as a 6-in.-thick portland cement concrete pavement in 1968. It has a traffic count of 840 vehicles per day. In 2001 it was cracked and sealed and then overlaid with 5 in. of HMA. The road was crack sealed in 2005 and 2011 and is in good condition. The surface has a slight amount of wheel patch rutting and oxidation but is structurally sound. The Cape seal construction procedure for
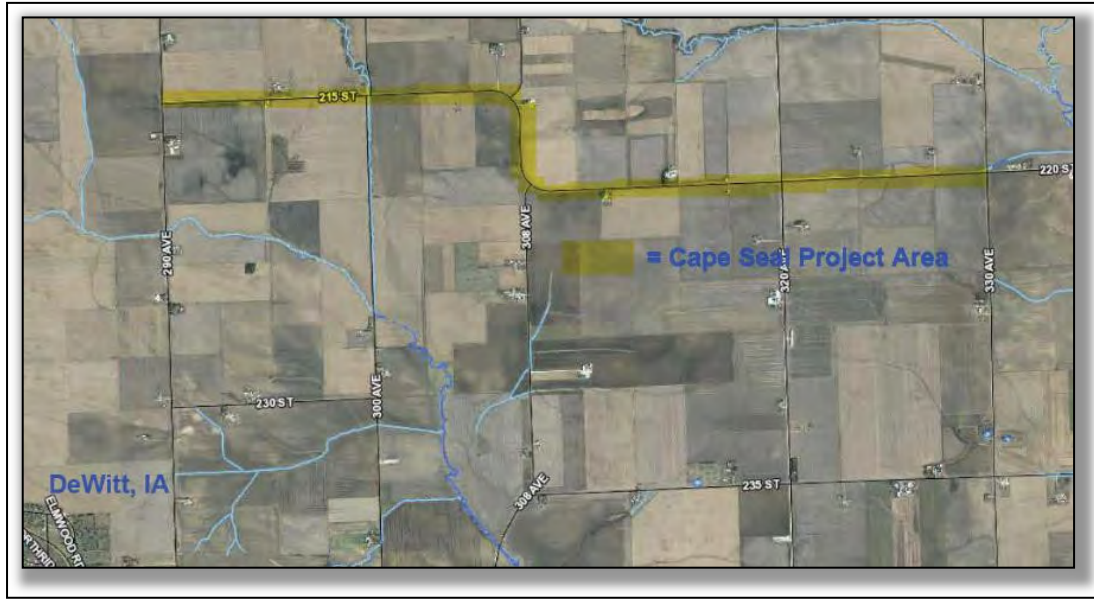
**FIGURE 1 Proposed Cape seal demonstration project location.
Image from Google Earth.**

the pilot demonstration project will be documented in detail. The construction activities and pavement performance measurements to be documented during Cape seal construction follow:

- Any repairing activities immediately before construction
- Visual evaluation of the existing surface condition, including cleanliness, drying, distress, and drainage in accordance with FHWA guidelines (*3*).
- Checking the actual field application rates of the aggregate and asphalt binder in accordance with FHWA guidelines (*3*).
- Checking the weather requirements of Cape seal, including the air and surface temperature, precipitation, humidity, and wind speed, per FHWA guidelines (*3*).

The Cape seal will seal the surface cracks, deter further oxidation, and fill in the minor rutting issues. The major performance goals of this project are to reduce severity of cracks, correct current rutting, and improve International Roughness Index (IRI) parameters. Pavement performance of the constructed pilot demonstration project will occur over a period of 5 years. Existing pavement condition has been collected for all paved routes in the State of Iowa via the Iowa Pavement Management Program (IPMP). Clinton County will participate in the IPMP data collection program and have additional data collection cycles completed every 2 years after the cape seal has been constructed

(2021, 2023, 2025, 2027, and 2029) and will conduct visual inspection of the treatment on an annual basis to document any surface failures (e.g., Cape seal debonding, loss of aggregate, and so on). This data will serve as a baseline from which the performance of the treatment will be measured over the 5-year period. The performance measurements to be monitored are listed below:

- IRI,
- Rut depth measurements using a straightedge (commonly called a "rut bar"),
- Cracking severity and other distress types in accordance with the *Distress Identification Manual for the Long-Term Pavement Performance Program* (*4*), and
- Any maintenance and repair activities.

The data collected will be compiled for data analysis to explore the effectiveness of Cape seals as an innovative pavement preservation method. A detailed statistical analysis, including an *F*-test based analysis of variance, paired *t*-test, and multivariate linear regression analysis, will be conducted on the compiled data in the developed database to scrutinize if Cape seals could reduce severity of cracks, correct current rutting, and improve IRI parameters when compared to pavement performance data before Cape seal application.

Findings

The Cape seal demonstration project was completed in October of 2022 at a cost of $530,000. Cape seal design was as follows:

1. First layer chip seal:
   ½-in. aggregate (30 lb/yd^2) + CRS-2P binder (0.35 gal/yd^2)
2. Second layer chip seal:
   ½-in. aggregate (25 lb/yd^2) + CRS-2P binder (0.30 gal/yd^2)
3. Emulsion control:
   Diluted CSS-1 binder (0.12 gal/yd^2), dilution ratio is 1:7
4. Microsurfacing (single layer):
   100% passing 3/8 sieve (L-4 aggregate) 19.8 lb/yd^2, CQS-1H emulsion 0.3 gal/yd^2, portland cement mineral aggregate 1% by dry weight of aggregate, Application rate 24 lb/yd^2

Construction procedures were as follows (Figure 2):

Step 1: Surface milling and crack filling
Step 2: Site cleaning
Step 3: First chip seal: Spray emulsion + spread aggregate + compaction + sweeping
Step 4: Second chip seal: Spray emulsion + spread aggregate + compaction + sweeping
Step 5: Emulsion control
Step 6: Microsurfacing

Conclusion

The outcomes resulting from this project will be of great benefit for meeting the Iowa DOT and local transportation agencies' needs for deploying innovative pavement preservation tools. A number of successful applications of cape seals have been reported so far by case studies, not only within the United States but also in several different countries. Cape seals have never before been used by Iowa counties, and its effectiveness as a pavement preservation strategy has not been evaluated in the state of Iowa. This demonstration project will be helpful in documenting the performance of a cape seal application over a 5-year period and in providing the necessary data for the Iowa DOT and local transportation agencies who are considering deploying cape seals

FIGURE 2 Cape seal construction. Photo by Clinton County.

as a standard pavement preservation practice. Construction details and preliminary performance findings will be presented as part of this extended abstract.

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Author's Note

Optimizing the Effective Use of Recycled Asphalt Pavement in Local Roadways

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This paper involved the construction of eight test sections in Columbus, Ohio, to evaluate the effects of using different recycled asphalt pavement (RAP) contents and types of rejuvenator on the performance and properties of asphalt mixtures used for local roads. The first section (control section) had a mix of 20% RAP and PG 64-22 binder. While three sections had mixes with 30% RAP, 40% RAP, and 50% RAP, PG 64-22 binder, and Sylvaroad rejuvenator (tall oil), three other sections had mixes with the same RAP percentages and binder but used Hydrolene (aromatic extract) as the rejuvenator. The last test section was constructed using a mixture with a 30% RAP and PG 64-28 binder (softer binder). Specimens were compacted in the laboratory from loose mixtures that were obtained during the construction of each test section. Laboratory tests were conducted to evaluate the resistance of the prepared samples to fatigue cracking, low-temperature cracking, rutting, and moisture damage. The test results showed that Hydrolene was more effective than Sylvaroad in improving the fatigue cracking resistance of RAP mixes with more than 30% RAP. In addition, the tests results showed that the 30%, 40%, and 50% RAP mixes had similar low-temperature cracking resistance to the control. The laboratory test results also showed that mixes with 30% and 40% RAP had acceptable resistance to moisture damage. All mixes had acceptable rutting resistance. Finally, preliminary field evaluation showed that there were no observed distresses in the test sections after 7 months of construction.

DATA DRIVEN SAFETY AND OPERATIONAL ANALYSIS

Systemic Safety Screening in San Juan National Forest
A Practical Approach with Limited Crash Data

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As a national forest, the San Juan National Forest (SJNF) falls under the jurisdiction of the United States Forest Service (USFS), an agency of the United States Department of Agriculture. SJNF covers 1.8 million acres in southwest Colorado and contains three regions:

- Mancos-Dolores,
- Columbine, and
- Pagosa.

The Office of Federal Lands Highway (FLH), in partnership with the Federal Highway Administration (FHWA) Office of Safety, developed a forest road safety plan (FRSP) to assess policies, identify relevant risk factors, and recommend key countermeasures to reduce roadway departure crashes in the forest. However, crash data in SJNF are limited; only 38 crash records on SJNF roads were available, provided by the Colorado Department of Transportation (CDOT), for the years between 2010–2018. Furthermore, the FLH project team could only generally locate 28 of these records (i.e., not to a specific location, but at the intersection of two roads or at a rounded distance offset from an intersection). As a result, the project team needed a practical method for assessing risk on the forest's 2,498 mi of roadway to deploy effective low-cost countermeasures.

Al-Kaisy and Huda (*1*) published a framework for screening low-volume roads in Montana that does not necessarily require crash data (although the presence of a crash within the last 5 years is a risk factor). Segment-level risk factors noted in this research include the following:

- Road width,
- Horizontal curve radius,
- Vertical grade,
- Driveway density,
- Side-slope steepness,
- Fixed objects on roadside,
- Unpaved road,
- Poor pavement condition,
- Posted speed limit, and
- Traffic volume.

Although these risk factors are substantiated in the existing literature (*2,3*), many of these risk factors do not necessarily apply to SJNF or are difficult data to obtain. Almost all roads in SJNF have a historic traffic count of less than 500 vehicles per day, and roads are typically only wide enough to accommodate a single direction of traffic. Furthermore, only 17 of the forest's roughly 2,500 mi are paved. This limited the potential risk factors present (as well as applicable countermeasures) for roads in the forest. However, the project team was able to identify readily available data that could support risk-factor screening in SJNF (e.g., horizontal curvature, vertical grade, traffic volume, and exposure). This extended abstract documents a practical approach to systemic screening using readily available data for most low-volume road contexts in the United States.

Methodology

The FLH project team had access to the following data:

- Digitized roadway centerlines provided through the USFS Geodata Clearinghouse that includes
    - Maintenance level (*4*),

- – Functional class,
- – Number of lanes, and
- – Surface type.
- Vertical elevation data available through the United States Geological Survey's *National Map* (10-meter resolution).
- Sporadic traffic counts on SJNF routes (2008–2015).
- Other contextual data, including
  - – Trails and trailheads and
  - – Campground sites with monthly occupancy statistics.

The FLH project team partnered with university researchers to develop the following suite of risk factors:

- Horizontal curvature: The University of Wisconsin Traffic Operations and Safety Laboratory applied its Curve Finder methodology to derive curve characteristics using digital centerline inputs. This includes curve type, length, radius, and degree (*5,6*).
- Vertical grade: The Add Surface Information tool in ArcGIS Pro allowed the FLH project team to derive vertical slope associated with individual curves; this produced the minimum, maximum, and average slope along each curve. This information can be combined with horizontal curve characteristics to develop compound risk on horizontal curves (Figure 1).
- Exposure: Although there was no single comprehensive method for assessing exposure (e.g., annual average daily traffic), the FLH project team had access to some traffic count data, as well as several surrogates that can assist with prioritization:
  - – Maintenance level is a planning-level indicator that corresponds to the form, function, and level of user comfort associated with the roadway. Higher maintenance levels (e.g., 5) indicate paved roads and a high degree of user comfort, while lower levels (e.g., 2 and 1) prohibit passenger cars or public vehicles altogether.
  - – Functional class and number of lanes, like maintenance level, indicate the function of the road. Like most state systems, arterials and collectors tend to carry more vehicular traffic than local roads.
  - – Surface type, although mostly unpaved, indicated higher or lesser use by motor vehicles. Crushed aggregate and gravel surfaces cover higher trafficked segments as opposed to native material surfaces.
  - – Campground reservations are a readily available indicator of visitation and the desirability of certain destinations. Although SJNF provided its own campground
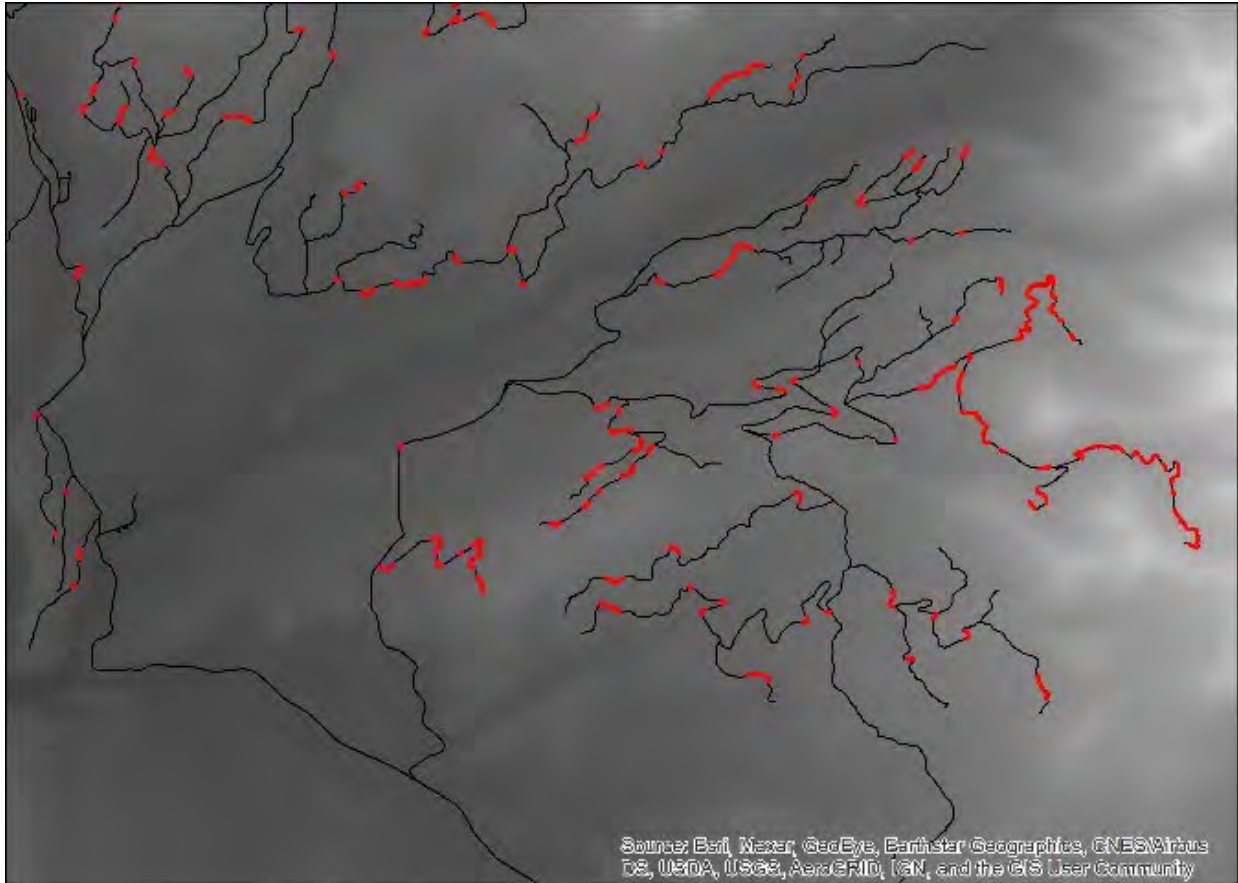
FIGURE 1 Example of horizontal curves that meet high-risk criteria in SJNF.

reservation data, reservation data are available nationally through the recreation.gov portal.

The FLH applied these data to identify corridors and specific curves that could be prioritized for treatment:

- Horizontal curves longer than 50 ft with an estimated radius of less than 300 ft.
- Vertical grade with a maximum slope estimated at greater than 4 percent.
- Operational maintenance level of 3 (suitable for passenger cars) or 4 (moderate degree of user comfort).
- Functional classification of arterial or collector.
- Crushed aggregate or gravel surface type.
- Two travel lanes indicated in USFS centerline records.

Findings

These data allowed the project team to compile a series of risk factors on the network and use these risk factors to develop countermeasure quantities. Figure 2 demonstrates a corridor that the FLH team prioritized based on the number of exposure surrogate risk factors, as well as the frequency and density of individual high-risk curves.

After location prioritization, the FLH project team used a focused list of countermeasures to develop installation quantities and potential costs. The focused list of countermeasures, using CDOT standards, included:

- W1-1 and W1-2 signs are considered Class I signs and paid by the square foot under item number 614-00011.
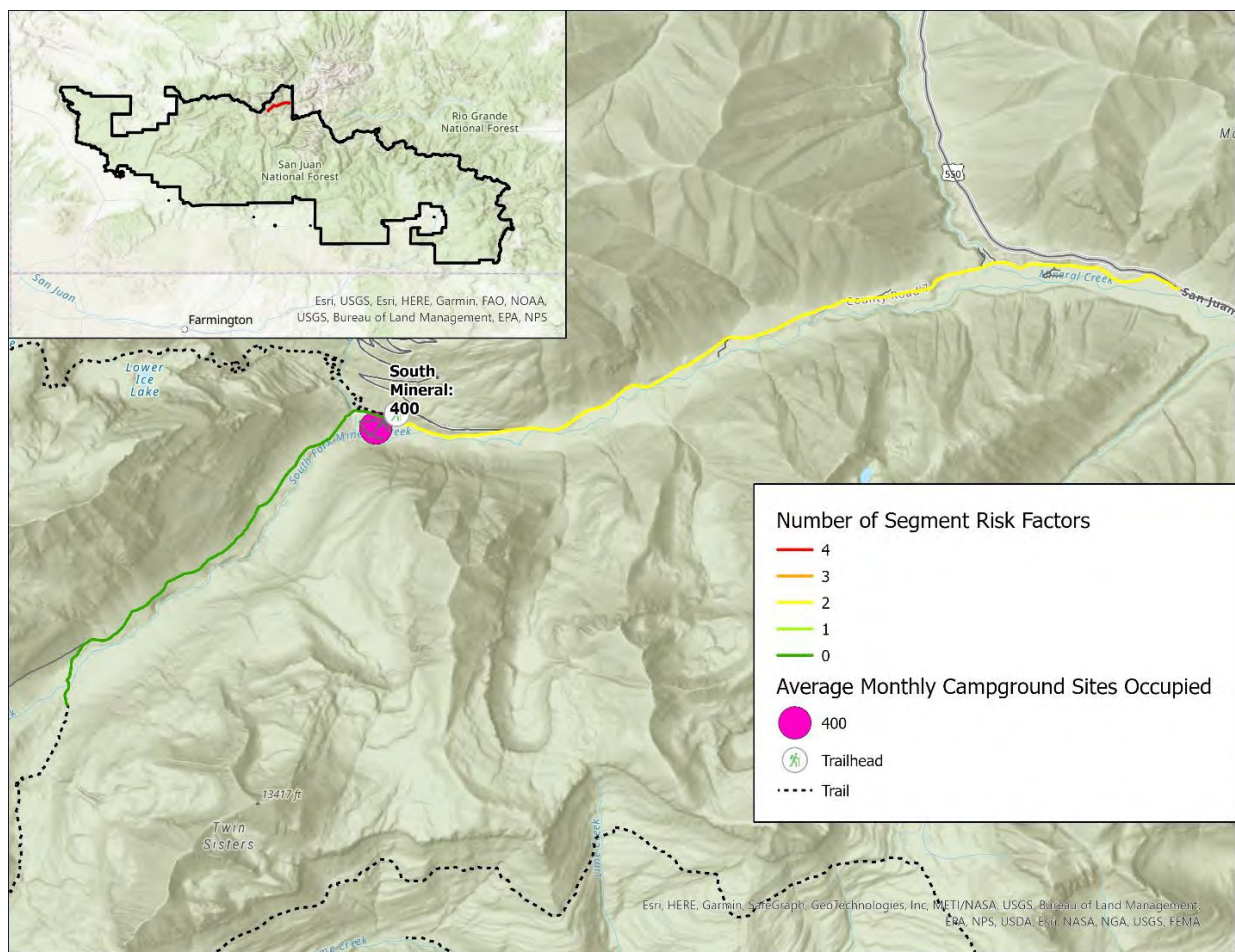
FIGURE 2 Example of a high-priority corridor based on curve, grade, and exposure risk factors.

- W1-6 signs are considered Class II signs and paid by the square foot under item number 614-900012.
- Both Class I and Class II signs are placed on 2-in. round steel sign supports under item number 614-01503. The item is paid per signpost.
- Post-mounted delineators are paid by each item under item number 612-00081. Barrier mounted delineators, assumed to account for a small percentage of the total estimated number of delineators, are paid per delineator under item number 612-00021.

Table 1 provides countermeasure screening criteria for horizontal curves and intersections on forest roads.

The FLH project team used CDOT item costs based on the 2021 average bid and awarded bid prices to generate estimated costs associated with countermeasure quantities. This analysis, at the network and priority-corridor level, produced a prioritized, planning-level estimate of costs associated with Table 1 improvements (Table 2).

TABLE 1 Countermeasure Application Criteria for Horizontal Curves and Intersections

Curve Countermeasure Criteria	
Radius < 300 ft; Vertical grade > 10%	• 4 approach delineators • 2 Class I signs (W1-1, W1-2, etc.) at 6.25 ft^2 each* • 2 Class II signs (W1-6, etc.) at 8 ft^2 each* • 1 sign support and foundation per sign • Number of on-curve delineators based on spacing guidelines in Table 3F-1 of CDOT's standard drawing (S-612-1; issued July 31, 2019)
Radius 300–650 ft; Vertical grade > 10%	• 4 approach delineators • 2 Class II signs (W1-6, etc.) at 8 ft^2 each* • 1 sign support and foundation per sign • Number of on-curve delineators based on spacing guidelines in Table 3F-1 of CDOT's standard drawing (S-612-1; issued July 31, 2019)
Radius > 650 ft; Vertical grade > 10%	• 4 approach delineators • Number of on-curve delineators based on spacing guidelines in Table 3F-1 of CDOT's standard drawing (S-612-1; issued July 31, 2019)
Intersection Countermeasure Criteria	
Intersection of 2+ Roads of Maintenance Category Level 3 or Higher	• 6 Class I (wayfinding) signs at 2 ft^2 each • 2 sign supports and foundati on per intersection (3 signs per support)

*All sign classifications refer to the applicable *Manual on Uniform Traffic Control Devices* code (*7*).

TABLE 2 Estimated Cumulative Countermeasure Costs Prioritized by Corridor Risk Factor Count

Risk Factors (No.)	Cumulative Curve Cost ($)	Cumulative Intersection Cost ($)	Cumulative Paved Road Cost ($)	Cumulative Total Cost ($)
0	$25,928,231	$384,356	$21,479	$26,334,067
1	$2,829,723	$2,474	$53,648	$2,885,846
2	$2,380,768	$34,641	$87,465	$2,502,875
3	$3,669,401	$69,283	$50,485	$3,789,169
4	$383,133	$23,919	—	$407,053
Total Cost ($)	$35,191,256	$514,675	$213,078	$35,919,010

Conclusion

The SJNF FRSP is a practical example for how several thousands of roadway miles can be screened with very limited data, particularly limited crash data. The combination of horizontal curvature, vertical grade, and exposure surrogates allowed the FLH project team to prioritize corridors in SJNF and develop countermeasure packages that can address safety at these locations. Using costs obtained from CDOT bid information, improvements for the highest priority corridors in the forest can be addressed for within the cost range of a typical Colorado Highway Safety Improvement Program project. FLH plans future data analysis and systemic screening improvements so that additional risk factors can be addressed in future projects, particularly those associated with roadside safety and sight distance.

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Acknowledgments

Development of Safety Performance Functions for Iowa's High-Speed Paved Secondary Roads

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Iowa's secondary (county) roads make up approximately 78% of the total miles of roads within the state. High-speed paved secondary roads account for almost 18,000 mi total, which over a 5-year period saw approximately 23,000 crashes. These crashes tend to be widespread, with some sections of road not seeing any crashes, and therefore network screening provides a useful process to help identify candidates for safety improvements (*1*). In order to support this effort, a set of safety performance functions (SPFs) were developed for the entire population of paved secondary roads within the state of Iowa. This was done instead of calibrating *Highway Safety Manual* SPFs as it would likely result in better predictive reliability (*2*)*.* The Empirical Bayes (EB) method, which accounts for regression to the mean, can then be conducted with SPFs. EB combines the predicted crashes for the segment from the SPF with the observed crashes. These EB estimated crashes can then be compared to the number of crashes that were predicted for the segment from the SPFs, and then these two values together can be used to calculate a potential for crash reduction (PCR) value, which can be used in prioritizing sites.

Methodology

Development of the SPFs started by determining the homogenous road segments. Determining segmentation can impact the results of the models. Original segmentation started with basic criteria that included segments with a speed limit ≥45 mph, those located outside city limits, and only undivided roads. Before segmentation could begin,

however, the data needed to be cleaned, including determining an annual average daily traffic (AADT) to use. As 5 years of data were included in the models to be consistent, the AADT from the middle year (i.e., 2018) would be the AADT used. In some cases, the AADT for segments had been collected in years before or after 2018, and therefore the AADT data were calculated using expansion factors to determine an approximate 2018 AADT. Initially, to be considered homogenous each segment would have the same ROUTEID, be adjacent, be located within the same county, have the same speed limit, and have the same expanded 2018 AADT. Once homogenous segments were developed, crashes were then assigned to each segment. This included all secondary road crashes 2016–2020, excluding intersection-related crashes as these were accounted for in previous intersection SPFs the Iowa DOT had developed.

However, after attempting to develop models using the SPF-R code (*3*), which was developed by Eric Green of Kentucky, this segmentation would not yield adequate models because of poor fit. Metrics and tools used to assess fit included percentage cumulative residual (CURE) deviation, maximum absolute CURE deviation, maximum absolute deviation, Akaike information criterion (AIC), and visual inspection of CURE plots. From there, roads were further segmented by shoulder-type categories as it was thought this would have a significant influence on crash frequency on secondary roads. This included paved shoulders, gravel shoulders, and earth shoulders, or unknown shoulders based on the right or outside shoulder. Crashes were then again reassigned to this new set of homogenous segments. This second segmentation was used to develop models based on speed limit and shoulder type, which yielded models with better fit than the regular segmentation, but there were still issues with some of the fitness indicators (e.g., high CDP values, large AIC, large moving average convergence/divergence values). This improvement led the team to try further aggregating the segments by including surface width in addition to the shoulder type, speed limit, and other segmentation variables, including lane-width categories and the number of lanes, which all proved to reduce the fit of the models.

Next, an indicator of curvature within the segmentation was included. However, due to issues with the available curve dataset (e.g., some curves not identified, some broken up, some very large radius curves), breaking up curves into their own segments was not feasible. Additionally, to address some issues within the dataset, cleaning was needed before using. One of the major issues was that the mobile data collection picked up turns at intersections as curves.

To account for this issue, along with those mentioned earlier, queries were set up to exclude the data collection on vehicles turning at intersections as well as on the very large radius curves. The criteria that were used included the following:

- Remove any curves with a radius less than 250 ft and a length less than 160 ft that are within 250 ft of an intersection.
- Remove any curves with length less than 50 ft.
- Remove any curves with radii greater than 1,800 ft and central angle between –5° and 5°.
- Remove any curves with radii greater than 4,500 ft.

From there modeling began using the SPF-R (*3*) statistical package using the base model ($Y = Length * e^{\alpha}AADT^{\beta}$). Segments were broken up into curve segments, or those that had any curvature present (i.e., % of segment that was part of a curve >0) and those that had none (tangent segments). Additional analyses found a more sophisticated model, which included an interaction variable between AADT and length, resulted in better fit. The form of the models utilized included

$$Y = e^{\alpha} * AADT^{\beta} * Length^{\beta_2} * e^{\ln(AADT)*Length*\beta_3 + Length*\beta_3 + \frac{1}{AADT}*\beta_5}$$

for all models except the low-speed tangents, which used the model

$$Y = e^{\alpha} * AADT^{\beta} * Length^{\beta_2} * e^{AADT*Length*\beta_3 + Length*\beta_3 + \frac{1}{AADT}*\beta_5}.$$

The final breakdown of models by subcategories can be seen in Figure 1. When developing the SPFs, no segments with lengths less than 0.1 mi were included as these are so short, they may skew the results.

Curve SPFs

Segments with curvature were modeled first. The percentage of segment length that was curve was used as the curvature indicator in our SPFs. The other available variables for curves were curve length, but that varied based on the length of the segment and the other variable was number of curves, which we weren't as confident in due to some of the curves being broken up into multiple parts. Due to some uncertainty with the dataset, an attempt was made to break up the curve SPFs into as few sub models as possible. After performing sensitivity analyses on the data, it was found that two groups (<25% curvature and 25%+ curvature) performed best.
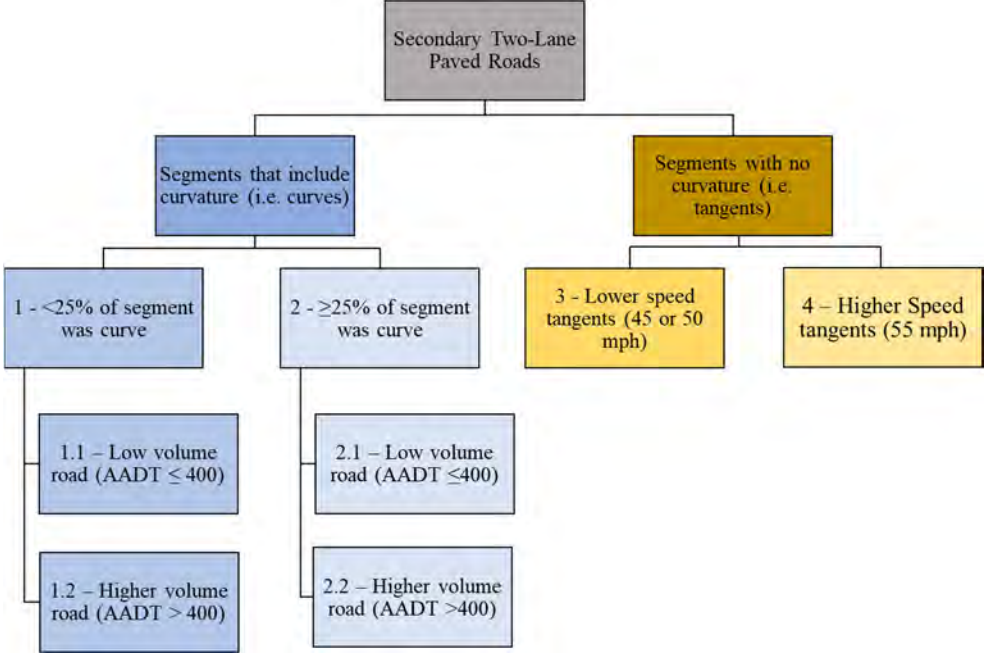
FIGURE 1 Categories of developed SPFs.

After attempting as a single SPF for each category, it was determined the SPFs would need to be broken into further subcategories based off sensitivity analysis of the AADT. The AADT break was decided to 400 vehicles per day (vpd) because that is one of the common definitions for a low-volume road. The goodness-of-fit indicators and the estimates of the best-fit models for the curve SPFs can be seen in categories 1 and 2 in Tables 1 and 2.

Tangent SPFs

Tangent SPFs were modeled by separating out by speed. As most secondary paved roads have speed limit of 55 mph, these segments were modeled separately from 45- and 50-mph segments. The 45- and 50-mph segments were combined because these would often be in the transition area into slightly more urban or suburban areas and would therefore have similar performance.

Modeling found that utilizing the full AADT range for the tangents resulted in the best fit. The best-fit models and goodness-of-fit measures for the tangent SPFs can be seen in categories 3 and 4 in Tables 1 and 2.

TABLE 1 SPF Models Goodness of Fit Indicators

Category ID	Category Description	AADT Range	Sample	Total Segment Lengths	Crashes	Pseudo R^2	Percentage CURE Deviation	Max. Absolute CURE Deviation	Max. Absolute Deviation	Standard Error
1	Segments with <25% of its length made up of curves	≤400	1,496	2,369.54	1,535	0.5903	0.2005	32.7582	0.8348	0.3627
		>400	1,507	2,873.42	6788	0.6072	0.5309	105.5497	2.5156	0.2235
2	Segments with greater than 25% of its length made up of curves	≤400	558	587.58	575	0.6291	2.3297	38.6172	0.8437	0.4359
		>400	851	943.28	3239	0.7196	0.7051	53.1838	2.1062	0.3554
3	Tangents with speed limit equal to 45 or 50 mph	All	728	342.21	844	0.7053	2.1978	34.3117	0.9307	0.5892
4	Tangents with speed limit equal to 55 mph	All	7459	10688.64	10304	0.6598	0.6034	124.0713	1.0159	0.1518

TABLE 2 SPF Estimates by Category

Category ID	Category Description	AADT Range	ln (α)	β1	β2	β3	β4	β5	Theta
1	Segments with <25% of its length made up of curves	≤400	−4.786249	0.822283	0.993061	0.056759	−0.374686	17.626772	2.619737
		>400	−2.719588	0.595734	1.068889	−0.006331	−0.008085	−294.1980	3.134331
2	Segments with greater than 25% of its length made up of curves	≤400	−6.269610	1.102943	1.016725	−0.053281	0.287678	41.215839	2.095238
		>400	−2.232265	0.546889	1.026789	0.021408	−0.158937	−268.8672	3.223368
3	Tangents with speed limit equal to 45 or 50 mph	All	−3.921364	0.627419	0.772551	0.000156	0.188232	−69.50889	2.888941
4	Tangents with speed limit equal to 55 mph	All	−5.625346	0.869250	0.775311	0.029081	−0.134446	22.515372	2.743776

Findings

Six unique SPFs were developed for the paved secondary road segments. As mentioned previously, two categories involved segments with some curvature present, while the other two categories involved tangents with lower speed limits (i.e., 45 or 50 mph) and higher (55 mph) speed limits. The segments with curvature models had sub-models developed breaking data up into low-volume and higher-volume ranges. Table 1 lists the categories, sample sizes, total length of segments (in miles), number of crashes, and goodness-of-fit measures of the final models, while Table 2 lists the estimates for the SPF.

Conclusion

Six unique SPFs were developed for Iowa's high-speed paved secondary roads. The SPFs were developed using homogenous segments that had the same RouteID, were adjacent, had the same speed limit, were within the same county, had the same AADT (2018 expanded), and had the same shoulder type. Due to issues with the curve dataset, developing SPFs specific for just curves and just tangents was not possible. Instead, the percentage of a segment's length that included curvature was used. In the future, if a better dataset were available, these should be modeled separately. However, the developed SPFs will be able to help Iowa better perform network screening on their paved secondary roads to identify roads with highest potential for crash reduction, which will help to better prioritize safety dollars and ultimately save lives.

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Contributing Factors for Crashes on Low-Volume Unpaved Roads

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Unpaved roads have different roadway characteristics than paved roads. One key way to investigate potential safety barriers is to analyze traffic crashes that have occurred on unpaved roads. The objective of this study was to estimate the influence of different factors on crash occurrence on unpaved roads. For this, 5 years of traffic crash data (2016–2020) from Kansas were collected and analyzed by applying binary logistic regression. Results showed that 23 factors were statistically significant among the 25 analyzed factors categorized under driver, roadway, and crash characteristics. Factors with higher likelihoods of occurrence included the presence of farm equipment, crashes occurring at yards and fields, and collisions with trains. Several factors were found to have low likelihoods of occurrence, such as roads with more than two lanes, interchanges, ramps, median existence, and multiple-vehicle collisions. However, widening the roadway may not guarantee crash reduction on unpaved roads. Effective steps could be proper educational programs targeting teen drivers' experience on unpaved roads. It is also recommended to post traffic control devices at railroad crossings and locations with nearby access to farms and agricultural lands. In addition, further study can be conducted to estimate the site-specific roadway width and speed limits.

Comparison of Analytical and Simulation Results for One-Lane Operation on Low-Volume Two-Lane Highways

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Work zones with one lane closed on two-lane highways require sharing the open lane with traffic from closed directions. In such work zones, the traffic control resembles to operating a two-phase signal and in rare cases a three-phase traffic signal. Temporary traffic signals (or flaggers) allow the open lane to be used in alternating manner.

Signal timing (green times and cycle length) directly affects the delay and queue, and one of the most influential variables is the operating speed of vehicles in the work zone. The operating speed is affected by work zone speed limit, work intensity and speed control technique, lane and shoulder width, acceleration capability of vehicles, and work zone length. Additionally, delay and queue computations must consider queue build up that often happens in oversaturated conditions. WorkZoneQ-Pro (WZQ-Pro) is developed with a new signal timing method that considers the above-mentioned factors. The procedure can handle multiple hours of analysis with two- or three-phase signal operation. Test scenarios are real-world work zone examples from three different states and were used to compute signal time variables and use them to compute queue and delay. These values were also computed using HCM 2016 procedures and are compared. In addition, the computed values were input to Vissim simulation software, and the results were compared. It showed that the WZQ-Pro results are reasonably close to Vissim simulation results, and that further validated that acceptable agreement existed between the analytical and simulation results.

Geotechnical, Stabilization, and Dust Control

A Case Study on the Implementation of the Unstable Slope Management Program

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Unstable slopes, both natural and constructed cut and fill slopes, are a common occurrence on roads and trails on federal lands. These unstable slopes create a serious managerial problem for the agency responsible for maintaining transportation. Left unaddressed, unstable slopes can block the use of roads and trails; cause loss of life, injury, and property damage; and cost millions of dollars.

Agencies that manage transportation systems often oversee aged transportation assets (bridges, pavements, retaining walls, etc.) with limited funding and personnel to effectively address all their assets. Because of these factors, geotechnical transportation assets such as unstable slopes are often addressed on a "worst first basis," meaning they are targeted for maintenance or rehabilitation only when they have already caused major interruptions to transportation use. This reactive approach is not cost-effective, as it misses significant opportunities to be proactive and implement lower cost repairs that can extend the life of geotechnical transportation assets (FHWA, 2019).

It is in recognition of this problem that the Unstable Slope Management Program (USMP) for Federal Land Management Agencies was created to aid in the management of unstable slopes as a geotechnical asset. The USMP is based on transportation asset management and geotechnical asset management (GAM) principles, meaning the use of policy-driven and performance-based approaches to the handling of assets. These approaches provide quality information for cost–benefit analysis, allowing the managing agency to make sound decisions regarding the allocation of limited money and personnel across their transportation system (FHWA, 2019).

The Western Federal Lands Highway Division (WFLHD) of the Federal Highway Administration was integral in the creation of the USMP and has had ample opportunities to implement it for a variety of local, state, and federal partners. It is the intent of this paper to provide an overview of the USMP and to highlight a case study where the USMP has been used to make proactive and informed decisions related to the management of unstable slopes.

Methodology

In order to develop a GAM program for unstable slopes, the USMP uses the following steps in a "roadmap" to success (FHWA, 2019):

1. Geotechnical Performance Goals and Objectives. This step involves determining the maintenance, operations, or safety goals that are meant to be achieved by the implementation of the USMP across a given area, whether it be a single road or a whole transportation network. This may involve identifying existing agency-specific programmatic guidance that can be applied to slope management or developing unique performance measures. Examples of metrics used to evaluate the efficacy of unstable slope management include slope condition surveys, road/trail closure frequency caused by unstable slopes, and the condition of unstable slopes adjacent to high-traffic recreation sites, trails, or facilities.

2. Inventory, USMP Rating, and Condition Assessment. Once performance goals and objectives are set, the agency must develop an inventory of their unstable slopes and conduct condition assessments. Within the USMP, the condition assessment of unstable slopes is done using a rating form. Ratings are collected on the basis on whether the unstable slope is classified as a landslide (includes translational and rotational failures, debris flows, etc.) or a rockfall (includes planar/wedge/toppling failures, rock avalanches, etc.). Each of the rating categories are assigned a score based on an exponential scale. The higher the overall total score, the higher the relative hazard and risk that unstable slope poses. The rating information gathered on the form is divided into four primary areas:

   a. Site Information. This includes location information, length of affected road or trail, hazard type, and speed limit, among others.

   b. Preliminary Rating. A preliminary rating is developed by gathering information on the severity of the landslide or rockfall, how that unstable slope affects the use of the road or trail, the traffic volume, and the economic or recreational importance of the road or trail affected. A preliminary rating of 15 to 21 total points is classified as "Good," and no further information is required for the rating. Slopes that are classified as "Fair" (22 to 161 total points) or "Poor" (>161 total points) are evaluated further.

   c. Detail Slope Hazard Rating. For "Fair" and "Poor" slopes, more slope information is gathered, including slope drainage observations, annual rainfall, landslide movement history, maintenance frequency, and structural condition.

d. Detailed Risk Rating. For "Fair" and "Poor" slopes, an in-depth evaluation of risk factors, such as human exposure, sight distance, impacts to right of way, environmental and cultural impacts, and maintenance complexity and costs is performed.

3. Performance Modeling and Measures. Once the inventorying and rating of all slope assets are complete, then the agency can rank all the slope assets by total USMP score and can conduct a prioritization of unstable slopes that need to be addressed with maintenance or remediation. The method of prioritization will depend on the anticipated funding available for unstable slope work and the overall performance goals and objectives set by the agency.

4. Cost and Economic Analysis. This step consists of professional geologists and engineers developing conceptual designs and cost estimates for maintenance or remediation for the prioritized slopes identified in Step 3. Once costs are estimated, a benefit–cost analysis can be performed for the prioritized slopes. The USMP includes a Conceptual Design and Cost Estimate Form and guidance on how to perform a benefit–cost analysis.

5. Decision Support. Having a list of prioritized unstable slopes based on the USMP ratings with completed benefit–cost analysis allows the agency to make proactive decisions on both the short- and long-term allocation of funding to perform maintenance or remediation of unstable slopes, and it gives supporting information for funding requests. The USMP also includes procedures on how to conduct a quantitative risk assessment that can be used as another decision support tool by providing an estimate of the risk associated with an unstable slope so it can be compared with other societal risks.

6. Monitor Performance. The USMP is intended to be a living system. This is emphasized by this step, which is the periodic reevaluation and rating of slopes. Repeatedly rerating slopes gives the agency direct feedback on their chosen performance goals and objectives and their decisions on selection of unstable slopes for maintenance or remediation. It also allows them to track the performance of slopes over time and be proactive in selecting unstable slopes for maintenance or remediation. To aid in this effort, the USMP includes a New Slope Event Form to record new unstable slope events and a Maintenance Form to track maintenance activities performed at unstable slopes.

Findings

At WFLHD, we have successfully utilized the USMP to aid our partners in making decisions regarding the management of unstable slopes. The USMP has been used on projects in national parks, national forests, and local municipalities across several states covered by WFLHD, including Alaska, Washington, Oregon, Idaho, and Wyoming. The following case study highlights one specific example.

Case Study:
Icicle Creek Road, Okanogan-Wenatchee National Forest, Washington

WFLHD was asked to evaluate the unstable slope hazards along a portion of United States Forest Service (USFS) Road 7600 (Icicle Creek Road) owned and maintained by Okanogan-Wenatchee National Forest. This roadway is located outside of Leavenworth, Washington, and provides access to numerous campgrounds and trailheads for the Alpine Lake Wilderness.

Unstable slopes along Road 7600 were identified as a key safety issue during a roadway safety audit performed by the USFS. The USFS performance goals and objectives were to increase safety to roadway users and to reduce maintenance burden, which is Step 1 of the methodology outlined previously (FHWA 2017).

In 2017, WFLHD engineering geologists identified and evaluated a total of 39 unstable slopes along Road 7600 (Step 2 above). These ratings were informed by conducting maintenance interviews with the USFS personnel responsible for Road 7600. Following the ratings, the sites were ranked by USMP total score, and the top 13 unstable slopes with total scores above 500 were identified as priorities for increasing the performance goals and objectives of the USFS (Step 3 above). WFLHD then developed conceptual design and cost estimates for the top 13 unstable slopes. In the process, they highlighted slopes with relatively low benefit–cost ratios, such as a debris flow that was rated highly, but had a low event recurrence interval and would require substantially more money to remediate compared to the other unstable slopes (Step 4 above) (FHWA, 2017).

The USFS took this information and selected a number of unstable slopes that they would target for maintenance or remediation (Step 5). The cost estimates developed by WFLHD were put together into a funding request for the Federal Lands Access Program (FLAP). In 2019, USFS was granted $3.2 million in FLAP funding to address unstable slopes along Road 7600.

In 2020, WFLHD engineering geologists reevaluated the top-rated slopes (Step 6) and refined the cost estimates and conceptual designs. Ultimately, three sites were selected for rockfall risk-reduction work (FHWA, 2022). This project is currently in design at WFLHD and construction is planned for 2023 or 2024.

Conclusion

The USMP is not without limitations. Like most asset management programs, it requires a substantial amount of labor to inventory and rate the assets before any decisions can be made. However, labor costs for the initial inventorying and rating process can be made more affordable by employing interns or seasonal workers to perform the initial USMP ratings with minimal training.

Another limitation is the USMP was made primarily for a rural context and for roads or trails with low-traffic volumes. However, it can be modified to account for higher traffic volumes by adjusting the rating categories for human exposure. And, even in its current form it is very flexible, able to be used on as small a scale as a single road or trail all the way up to entire transportation networks.

As shown in the case study, the USMP provides a direct vehicle to more proactive and cost-efficient management of unstable slopes. It does this by applying asset management principles in a methodical process that is policy and data driven. This process and the included suite of decision support tools gives agencies responsible for managing transportation infrastructure a basis for making sound judgements regarding the effective maintenance and remediation of unstable slopes.

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Modeling Dust Generation on Low-Volume Roads Based on Vehicle Speed and Surface Fines Content

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This study analyzes the role of vehicle speed and surface fines content on dust emission. Accordingly, 50 unpaved road sections in Iowa were evaluated, surface loose aggregate samples were collected, and dust was collected using Colorado State Dustometer at three speeds: 25 mph, 40 mph, and 55 mph. The data were analyzed using analysis of variance (ANOVA) test. Several dust prediction models were developed utilizing multiple linear regression (ML), nonlinear regression with an interaction term (NLI), nonlinear Beta regression (NLB), nonlinear curve fitting regression (NLCF), and a multilayer neural network (MNN). The model predictors included vehicle speed and surface fines content. When models were evaluated using synthetical data and compared using a post-hoc analysis, it was found that dust increases exponentially as vehicle speed increases and increases linearly as surface fines content increases. Also, at higher speeds, dust values will converge independent of the fines content in the surface materials. The ANOVA test results revealed that vehicle speed, surface fines content, and their interaction significantly affected dust emissions. The accuracy of models ranged from acceptable to good. The coefficients of determination (R^2) for ML, NLI, NLB, NLCF, and MNN training models were 0.736, 0.748, 0.704, 0.730, and 0.829, respectively. Evaluation of the models showed that independent of the R^2 value, the MNN model was the most accurate in predicting dust emissions, followed by the NLCF model, the ML model, the NLB model, and lastly the NLI model. The post-hoc test showed that MNN training, NLCF, and ML models produced comparable results.

Investigations on Strength, Durability, and Shrinkage Characteristics of Stabilized Silty Sand for Low-Volume Roads

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Low-volume roads (LVR) consist of less traffic (traffic < 2 million standard axles) and are located in rural areas. These roads are the connectivity of several village areas in India and play a vital role in the socioeconomic activities of any region. The availability of superior-quality materials near construction sites for rural road construction is limited and a real challenge for engineers and practitioners. Although the materials are available in some places, it could be uneconomical for construction due to the long haulage distance. The poor-quality material typically exhibits undesirable physical properties such as lower strength, high shrinkage, low bearing capacity and less durability (*1*). One such soil is silty sand (SM). It is the type of soil that has poor geotechnical properties. To improve the properties of such soil, suitable soil stabilization needed to be adopted. Literature demonstrated that the traditional techniques may be followed by the use of lime, cement, fly ash, and bituminous material (*2–4*). Other nontraditional techniques that are commercially available in the markets, such as salts, polymers, acids, resins, etc., also were observed. These may involve the high cost of construction (*5*). However, the study with the combination of cement and fly ash at optimum dosage with the suitable proportion as stabilizing agents in silty sand is found to be limited. Also, the linear shrinkage cracking with such soil is not known in great detail. Thus, the present study aims to find out the optimum dosage of cement and fly ash to be used for silty sand in the stabilization process. The strength parameters such as unconfined compressive strength (UCS), indirect tensile strength (ITS), California bearing ratio (CBR), and linear shrinkage cracking have been evaluated for the cylindrical silty soil samples. Previous studies reported that the use of traditional techniques, especially with silty soil, has been successful in gaining strength and durability characteristics (*6, 7*). The study with the use of lime with cement showed improved mechanical properties and showed proper agglomeration of particles in

microstructural organization with the stabilizers (*8*). The use of cement as a stabilizing agent has been dominant in the enhancement of strength properties in silty soil (*9*). The cementitious property has been added with different pozzolana materials and industrial wastes resulting in the formation of calcium-silicate-hydrated (CSH) gel. This gel formation showed improved mechanical and durability properties in stabilized soils (*7, 10*). The study was also made with the use of industrial waste such as bagasse ash with lime. It reported strong phases of CSH gel formation and was found to be an alternative to cement replacement (*11*). The use of coal fly ash with cement and polyester fiber was evaluated for compressive strength (UCS). It was reported that the fiber-reinforced cement fly ash soil specimens showed a decrease in the brittle behavior by inducing ductile property (*12*). The use of fly ash with coir showed significant improvement in the UCS value of stabilized silty sand specimens. The study reported optimum content of fly ash was 15% and coir fiber was found to be 3% (*13*). The stabilization with cement stabilized in silty sand imparts high strength, durability, and good resistance to fatigue behavior (*14*). It was observed that the use of cement significantly influenced the strength properties. The use of fly ash has been also effective in enhancing the mechanical properties of soil. However, the optimum dosage using both combinations has been found limited. Thus, the present study aims to find out the actual dosage require for silty sand.

Research Significance

India is one of the largest producers of fly ash in the world after the United States and China. It is estimated to produce 600 MT by the year 2031–2032 (*15*). Currently, the fly ash generation is 95–100 MT every year, which requires a large amount of space, i.e., 65,000 acres of land, which affects the agricultural land as well as human health (*16*). Thus, increasing environmental concerns and global warming are serious challenges. The application of such waste in various engineering sectors is alarming to budding engineers. The present study deals with the state of Bihar, an eastern province of India that has five national thermal power corporations. A huge amount of fly ash is generated every year and getting affected by cultivated lands. The fly ash was collected from Kahalgaon thermal plant, which was found to be a class F division. The soil was collected from the Banka district of state Bihar, India, and evaluated for strength and other characteristics.

Silty Sand

The soil was obtained from the Banka district, Bihar, India. Table 1 shows the physical properties of natural soil.

Methodology

The soil sample was collected from a southern district of the state Bihar, namely the Banka district. The soil was found to be silty sand per USCS soil classification. This soil was stabilized with cement and fly ash in different proportions at the ratio of 1:4 per the AASHTO 2009 guidelines (*17*). Cylindrical samples were prepared with a diameter of 50 mm and 100 mm in height. The cement was varied from 2% to 8% at a 2% increment by the weight of the total soil mass. The fly was in the ratio of 1:4 (i.e., 8%, 16%, 24%, and 32%, respectively) by the weight of the total mass of the soil sample. The strength characteristics were determined by conducting UCS and CBR tests on all the mix variants. The durability was assessed by exposing the prepared samples to wet–dry cycles. The linear shrinkage was determined by fixing the relative humidity in the humidity chamber. Table 2 shows the mix variation adopted for the study.

TABLE 1 Physical Properties of Natural Soil

SI No.	Test Name	Results	Codes Referred
1	Liquid limit test, %	24.34	ASTM D4318-10
2	Plastic limit test, %	1.06	ASTM D4318-10
3	Plasticity index, %	23.28	ASTM D4318-10
4	Type of soil	SM-silty sand	ASTM D2487-11
5	Optimum moisture content, %	12.02	ASTM D698
6	Maximum dry density, gm/cc	1.88	ASTM D698
7	CBR, %	4.26	ASTM D 1883
8	Unconfined compressive strength test, (N/mm^2)	0.35	ASTM D 2166

TABLE 2 Mix Proportions

Stabilizing Material	Variation of the Stabilizing Material, %			
	Mix 1	Mix 2	Mix 3	Mix 4
Cement (C)	2% C	4% C	6% C	8% C
Fly ash (FA)	8% FA	16% FA	24% FA	32% FA

Findings

The results of the UCS are shown in Figure 1. The cylindrical samples were exposed to curing for 7, 14, 28, 56, and 90 days. Then the UCS was evaluated by the load taken on the area at failure. Figure 1 shows that the UCS values have a drastic change with the adopted stabilization technique. The normal soil UCS was in the range of 0.28–0.38 MPa. The stabilized soil reported the UCS value from 0.82 MPa to 3.68 MPa. The maximum UCS value was attained at 3.85 MPa for mix 3. The 7-day UCS value was 2.17 MPa for mix 3, i.e., at 6% cement and 24% fly ash content. The 7 days required value for the granular sub-base (GSB) layer is 1.7 MPa per the guidelines of IRC:SP:89-2018. Therefore, it is the judging criteria for the 7-day UCS value for the prescribed layer. If the attained value is higher, it acts as a replacement for the GSB layer in the pavement. Similar trends and results were also observed with the value of the ITS test, CBR, and durability results. Figure 2 shows the ITS and durability plots.

The ITS value was 60 KPa for the control sample and 330 KPa for mix 3 stabilized soil after a 7-day curing period. After 90 days of curing, it was 82 KPa for the control sample and 575 KPa for the mix 3 sample. The CBR value for normal soil was 4.2% and for mix 3 was 32.4%. There was a drastic change in the CBR value, with a 7.72-times increment in the stabilized soils. The durability value after a 7-day curing period was 56% for the control sample and 83.24% for the mix 3 sample. After a 90-day curing period, the value obtained was 71% for the control sample and 91.68% for the mix 3 sample. The maximum values for ITS, CBR, and durability were attained by mix 3 stabilized soil samples.
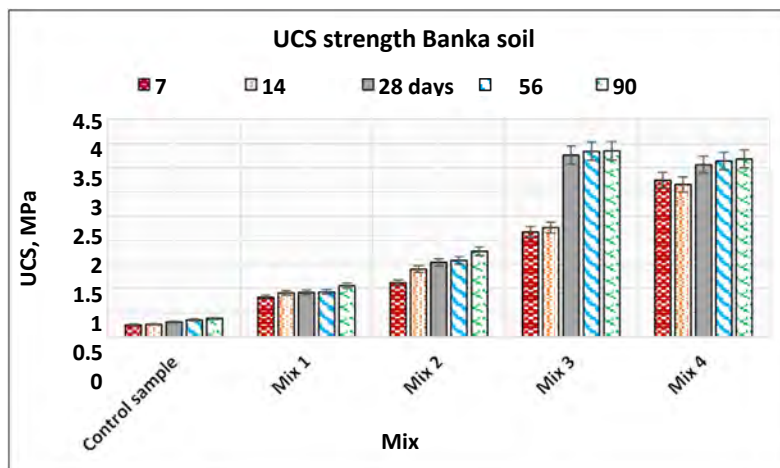
FIGURE 1 UCS test results.
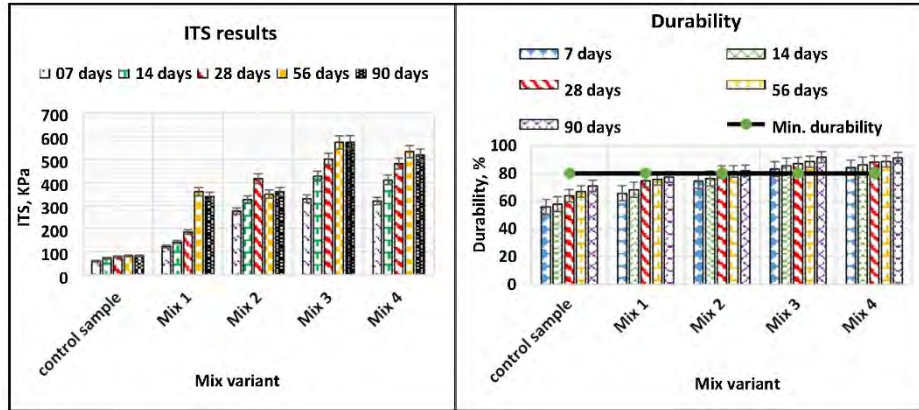
FIGURE 2 ITS results (*left*) and durability test results (*right*).

Dry Shrinkage Analysis

The linear shrinkage was conducted for the samples up to a period of 21 days for all the mix variants. The relative humidity was maintained constant for all the samples to assess the mixture behavior. The results obtained from the test are shown in Figure 3.

The value of linear shrinkage for normal soil was 2.974% and mix 3 was 1.186%, indicating a decrease of 150.75% with the mix 3 proportion. This might be due to the curing effect of the stabilizing material. The shrinkage value remains constant after 16 days with mix 3. A similar result was reported in the study by Biswal et al. (*18*).
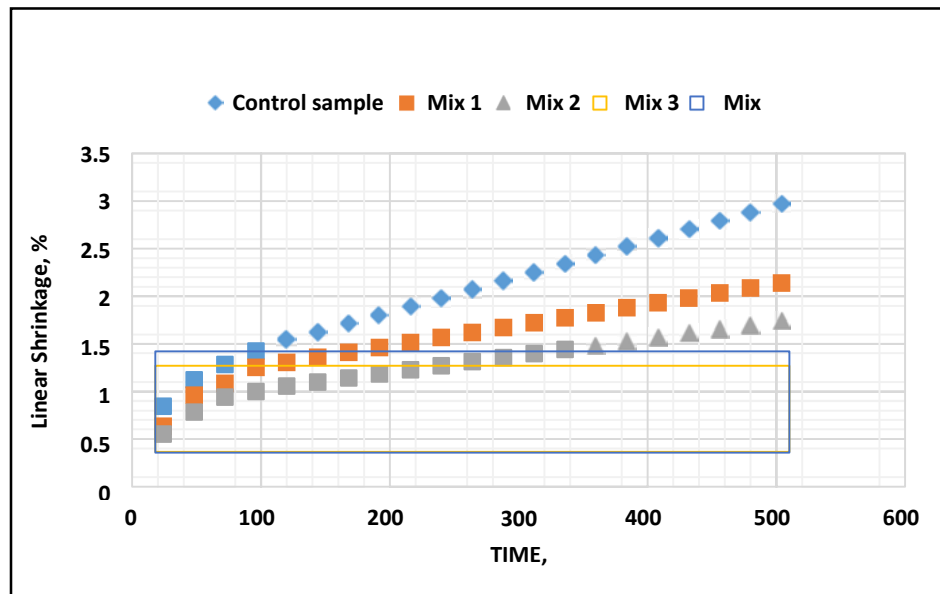
FIGURE 3 Linear shrinkage results obtained for all the mix variants.

Microstructural Analysis

A scanning electron microscopy (SEM) was used to evaluate the microstructure. Figure 4 shows the images taken for fly ash and normal soil at 7-day and 28-day curing periods. The shape of the fly ash was found to be round, spherical, and hollow, similar to that of cenospheres. The particle surfaces were smooth and densely packed close to each other. It was observed that the agglomeration and micropores were very high in the untreated soil sample. The image of soil treated after 7 days shows the formation of CSH gel and minimized microspores. The 28-day curing image shows the completion
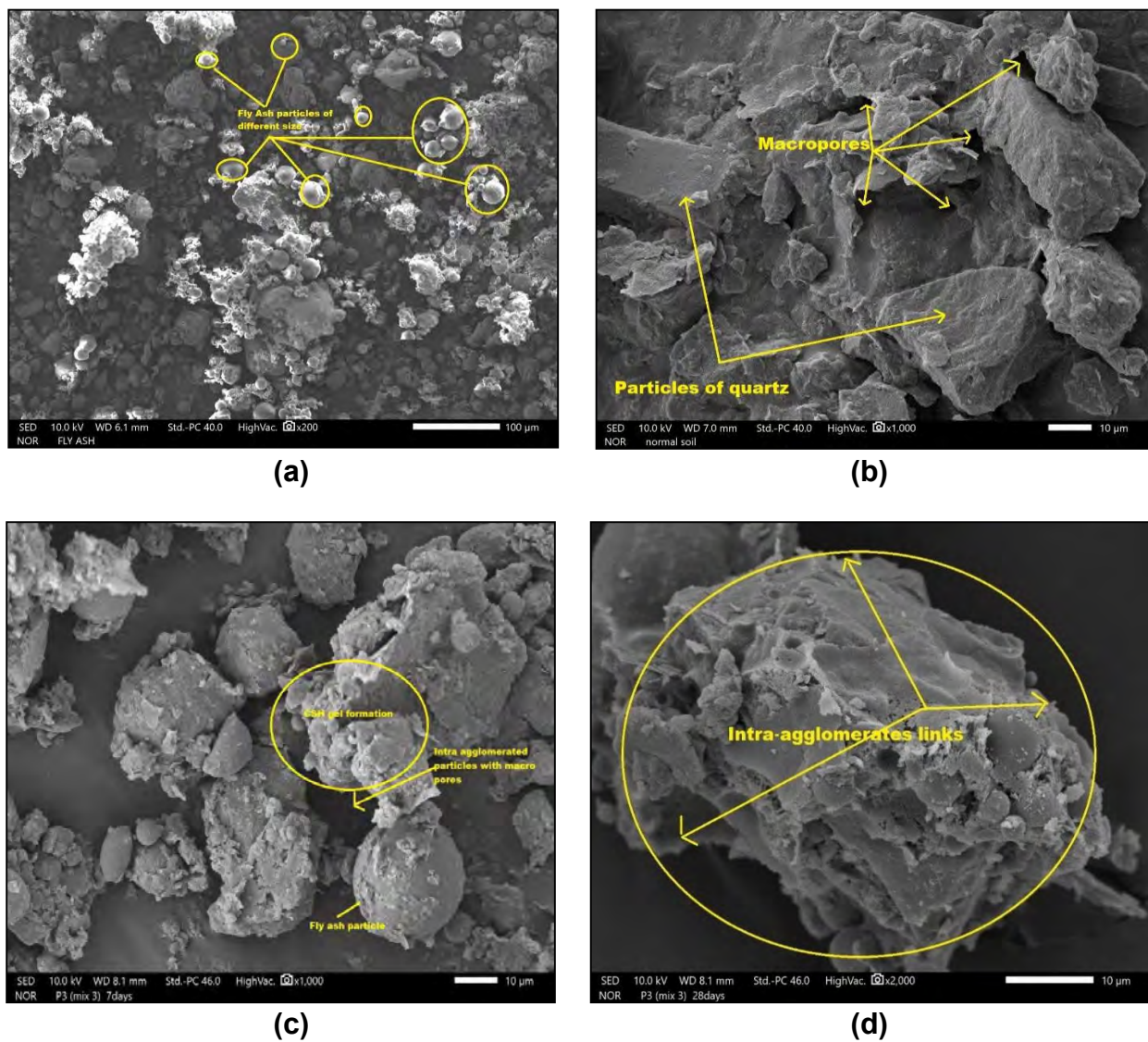
FIGURE 4 SEM images: (*a*) fly ash particles; (*b*) soil particles; (*c*) 7 days curing; and (*d*) 28 days curing.

of the hydration process and all the particles agglomerated. It can be said that these CSH gels are responsible for gaining high strength and durability characteristics. A similar result was also observed in line with Horpibulsuk et al. and Lemaire et al. (*7*, *8*).

Conclusions

The silty sand was stabilized with cement and fly ash with different proportions by fixing the ratio of 1 cement to 4 fly ash. It was seen that the strength, durability, and shrinkage characteristics were not known in great detail. The study attempts to address these issues with the application of class F fly ash. Based on the results, the following conclusions were drawn.

- The stabilization technique used in the present study has provided impressive results. The UCS value of mix 3 has achieved such a high strength that it could act as a replacement for the sub-base layer in rural road construction per the code specifications.
- A significant change in the ITS value was observed with the soil treated with cement and fly ash. Similarly, the results were followed by the CBR value with the treated soil. The maximum strength was obtained for the mix 3 proportions.
- The durability test results reported a higher value for the mix 3 proportion; the highest value was found to be 91.68%.
- The dry shrinkage test reported the least value for the mix 3 proportion and was found to be very effective at this proportion.
- The microstructural analysis using SEM images showed that the particles get densely packed and the agglomeration of all the particles leads to a magnificent increase in the strength.

It can be concluded that the usage of cement at 6% and fly ash at 24% could be the better proportion when working with silty sand. The cost of construction can be significantly reduced since the higher strength leads to replacing the sub-base layer and reducing the pavement thickness. Also, it leads to a decrease in the environmental load since 24% of fly ash can be utilized in such soils.

Acknowledgments

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Disaster Risk Preparedness and Repair Activities of Low-Volume Roads in Selected Developing Countries

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In the aftermath of natural disasters, the goal of international donors has been to restore safe road accessibility, create employment, and build local capacity of low-volume-road (LVR) investments. Reliable risk assessment is crucial to building emergency response and risk management capabilities before the disaster strikes (preparedness). Risk management of possible catastrophic events includes the development of susceptibility risk maps, planning of preparedness activities, and designing guidelines of cost-effective prevention work. Risk management assessment includes (1) mapping of populated areas (affected communities and poverty classification) and determining the type, severity, and occurrence probability and the related disaster impacts, which include landslides, erosion, flooding, and failure of structures; (2) selecting cost-effective engineering tools for the preparedness works, considering the project-level soils, geology, drainage, topography, and earthquake/shaking characteristics; and (3) evaluating the meteorological and rainfall intensity and duration records. Key preparedness activities include research, training, developing monitoring and alert indicators of possible catastrophic events, and generating related cost-effective engineering solutions. Urgent job order contracting procedures are used to cover all types of emergency works; these works are competitively bid with fixed costs and with related performance-based indicators. This paper emphasizes engineering components of planning affordable, preventive, and remedial measures for vulnerability reduction by applying lessons learned from the Ecuadorian Andean 1998 El Niño flooding damages of the local LVR system. This damage was associated with rainfall storm intensity and duration of 20% to 35% above the previous registered storms and thus were considered to be related to the ongoing climate change along the Andean region.

Introduction of Most Common Natural Disasters

Worldwide trends of natural disasters such as hurricanes, tropical storms, and related significant floods damages have been increased in terms of frequency or destruction severity, associated with the ongoing climate change that has caused irreversible impacts (*1–17*). The world's most common disaster-prone region is the Asia-Pacific region. The United Nations and the Thomson Reuters Foundation indicate that more than 2 million people, an average of 43,000 per year, have been killed by natural disasters since 1970 (*1, 2*).

The region encompassing Latin America and the Caribbean (LAC) is the second most disaster-prone region in the world. The LAC region has a population of 152 million, and it has been affected by 1,205 natural disasters (2000–2019). In the 1990s, the LAC region saw 45,000 fatalities and more than $20 billion (USD) in damages, experiencing approximately 40 significant disasters per annum. Between 2000 to 2019, the LAC region suffered 226,000 fatalities, 339,000 injured persons, and 14 million affected people, along with $54 billion (USD) in total damages from 75 earthquakes. Inadequate disaster-preparedness works, including the lack of infrastructure strengthening and inadequate surface and subsurface drainage facilities, were the most important factors contributing to these catastrophic losses in LAC.

Notably, about 70% of the natural disasters in the LAC region were meteorological events such as intense rainfall, hurricanes, and strong windstorms, likely related to climate change. By comparison, the remaining 30% of the natural disasters were related to geological phenomena such as earthquakes and volcanic eruptions (*1, 7–9*). The costliest catastrophic events in the LAC region were the 1985 Mexico City earthquake and the 1988 El Niño-related flooding in Ecuador, Argentina, and Peru. Each one of these two events caused an estimated $6 billion (USD) in damages (in 1999 dollars).

It is critical to recognize that natural disasters have had far more devastating effects on low- and middle-income countries than on high-income countries. In fact, approximately 95% of the deaths caused by natural disasters occurred in low- and middle-income countries. Therefore, international donors decided to support and finance the planning, design, and construction of natural-disaster preparedness activities, especially with regard to infrastructure and facilities works. As noted earlier, investing in the strengthening of infrastructure and facilities is crucial to mitigating the effects of natural disasters. Such preemptive activities can save lives, time, and money; prevent displacement; preserve communities; prevent the loss of livelihoods; build local capacity; and enhance resilience.

Key to the success of disaster preparedness is ensuring on-time availability of adequate institutional and technical capabilities, sustainable funding, on-time affordable designs, and a clear definition of the roles and responsibilities of national and international agencies. Finance ministries (FMs) of LAC countries are important players within the administration of natural disasters in terms of economic planning and shaping the private and the public sectors financial decisions. FMs decide whether to invest effectively in disaster preparedness or in disaster prevention rather than in costly works during or after the occurrence of the emergency. Strategic plans of preparedness emphasize a culture of prevention, reducing the vulnerability of people with low incomes, providing reliable risk information for decision-making, fostering leadership, and building partnerships that include the private sector, public sector, and affected communities.

The following information is provided to support efforts toward cultivating a culture of preparedness and disaster mitigation. This paper presents engineering-planning tools, socioeconomic considerations, issues and challenges of preparedness, disaster risk management aspects of LVR stability, and vulnerability reduction from catastrophic natural events in LAC countries. Since the frequency of excessive rainfall and flooding related to catastrophic events appears to be on the rise due to the ongoing impact of climate change, the severity of the economic damages from these events is also on the rise. Therefore, the development and implementation of affordable and cost-effective preparedness tools are essential (*1, 2, 5–15*).

Socioeconomic Considerations of LVR Investments

While the disaster events are natural in origin, the extent and severity of the damages are mostly a consequence of human activity and inactivity (*3–18*). Poor planning of affordable prevention works has aggravated the socioeconomic impacts of these catastrophic events, which includes the accelerated deterioration of LVR accessibility services. These impacts are measured in terms of damages to the economic and social infrastructure, a destruction of economic assets, environmental deterioration, inadequate income distribution, and acceleration of economic inflationary processes. The result is a negative long-term macroeconomic impact that is reflected on a downtrend in the per capita income. The experience of the LAC region confirms that there is a high correlation between the gross domestic product growth and the frequency, magnitude, and severity of natural disasters (*3, 7–14*). One of the most important impacts is the worsening of the national living standards, including worsening

of the LVR accessibility services to local schools, clinics, and socio/economic community centers. Also, the severity of natural disasters usually affects a country's entire population, and could affect neighboring countries in terms of migration, vector transmission, deterioration of watersheds, reduced demand for imports, and interrupted communications.

In the aftermath of a large disaster, countries often face declining exports and rising imports, a deceleration of economic growth, and a reduction of the per capita income. A decline in tax revenues could prolong fiscal instability and increase the country's level of indebtedness. In such cases, donors have helped low- and middle-income countries move out of the emergency stage back into their socio/economic development trajectory, as efficiently as possible (*4–15*). Donors' financing high priority is directed to benefit people with low incomes. This includes smaller projects to repair and reconstruct LVR and bridges, water, sanitation, drainage and related flood-control and slope-stabilization works in low-income communities. In addition, donors have financed programs that protect public expenditures and improve living conditions and economic opportunities for people with low incomes (*3–15*). These programs have traditionally been replaced by reconstruction needs when disaster strikes. As an example, an Inter-American Development Bank (IADB) reconstruction program in the Dominican Republic after Hurricane George made up the fiscal shortfalls to safeguard programs aimed at poverty reduction and children's welfare. IADB financing helped selected LAC countries to manage the adverse macroeconomic impacts, which included financial support to augment shortfalls of recurrent public expenditures for vital social programs, including infrastructure such as LVR, water, and sanitation services. Other IADB socioeconomic supporting activities include the countries' balance of payments and restructuring their national debts. Therefore, to reduce the long-term socioeconomic damages of catastrophic events, it is essential to consider the following infrastructure mitigation and reconstruction strategies: (1) classifying financial resources earmarked for preventing and mitigating the impact of natural disasters as high-yield and long-term investment in economic, social, and political terms; (2) developing catastrophic risk classification maps and affordable national and regional natural-disaster preparedness plans that progressively reduce the degree of vulnerability and, therefore, improve the prospects for future development.

The reasons for the high vulnerability to natural disasters are varied and complex. In many LAC countries, people with low incomes, and among these, women, children, and ethnic minorities, are the most fragile and vulnerable population groups. People with low incomes live in greater risk areas, use environmentally damaging farming activities or work marginal land, and have limited access to information as well as to basic services

of disaster protection. In many ways, poverty exacerbates the vicious cycle of disasters. Therefore, effective vulnerability reduction must follow the strengthening of the national and the regional macroeconomic capacity, the introduction of active policies that reduce social distortions, provide poverty alleviation, and improve the coordination of regional policies and international aid activity.

Issues and Challenges of Preparedness Activities

Preparedness involves building an emergency response and management capability before a disaster occurs. Key disaster-preparedness activities include training programs; availability of citizen information and education programs; reliable hazard detection and warning systems; developing cost-effective engineering tools for risk assessment of vulnerabilities; and building in resilience measures for disaster preparedness. Key challenges of preparedness activities include the following:

1. Develop interinstitutional and interdisciplinary coordination to break the cycle of destruction and reconstruction. The long-term objective is more institutional preparedness activities, more investments in preventive works, and less repair or remediation works (*10–17*). The challenge is to prioritize prevention activities as high-return investments that support significant vulnerability reduction, including controlling urban expansion, land-use planning, and productive activities, and averting misuse and degradation of natural resources.

2. Develop private–public sector collaboration among financial institutions, insurance agencies, and governmental and intergovernmental entities. FMs are important players in terms of economic planning and shaping private and public financial decisions. The FMs leadership helps ensure funding, facilitates the incorporation of disaster management into national development policy, and provides incentives for financing preparedness activities, including drainage improvement, slope stability, and infrastructure-strengthening works.

3. Develop susceptibility risk maps for preparedness planning and prediction of lifeline disruption in future earthquake or heavy flooding. These maps define five to six levels of risk-susceptibility, considering flooding, hydrogeological, and earthquake-shaking characteristics (*14–26*). As an example, along the Ecuadorian mountainous area of the Cuenca-Molleturo-Empalme (C-M-E) road, see Figure 1, the slope-failure risk varied from extremely stable slopes with a factor of safety (FS) of over 2.0, to unstable slopes with FS < 1.2. By comparison, FS = 1.2 is the minimum recommended

FS value needed to avert or minimize the risks of slope failures (*21–24*). Slope susceptibility risk maps incorporate the estimated FS values along the road into digital-layer-topographical-maps, usually in a 1:100,000 scale. These susceptibility risk maps incorporate local hydrogeological, rainfall, flooding, and earthquake characteristics in terms of location, flooding severity, peak acceleration, and intensity (*21–26*).

(a)   (b)
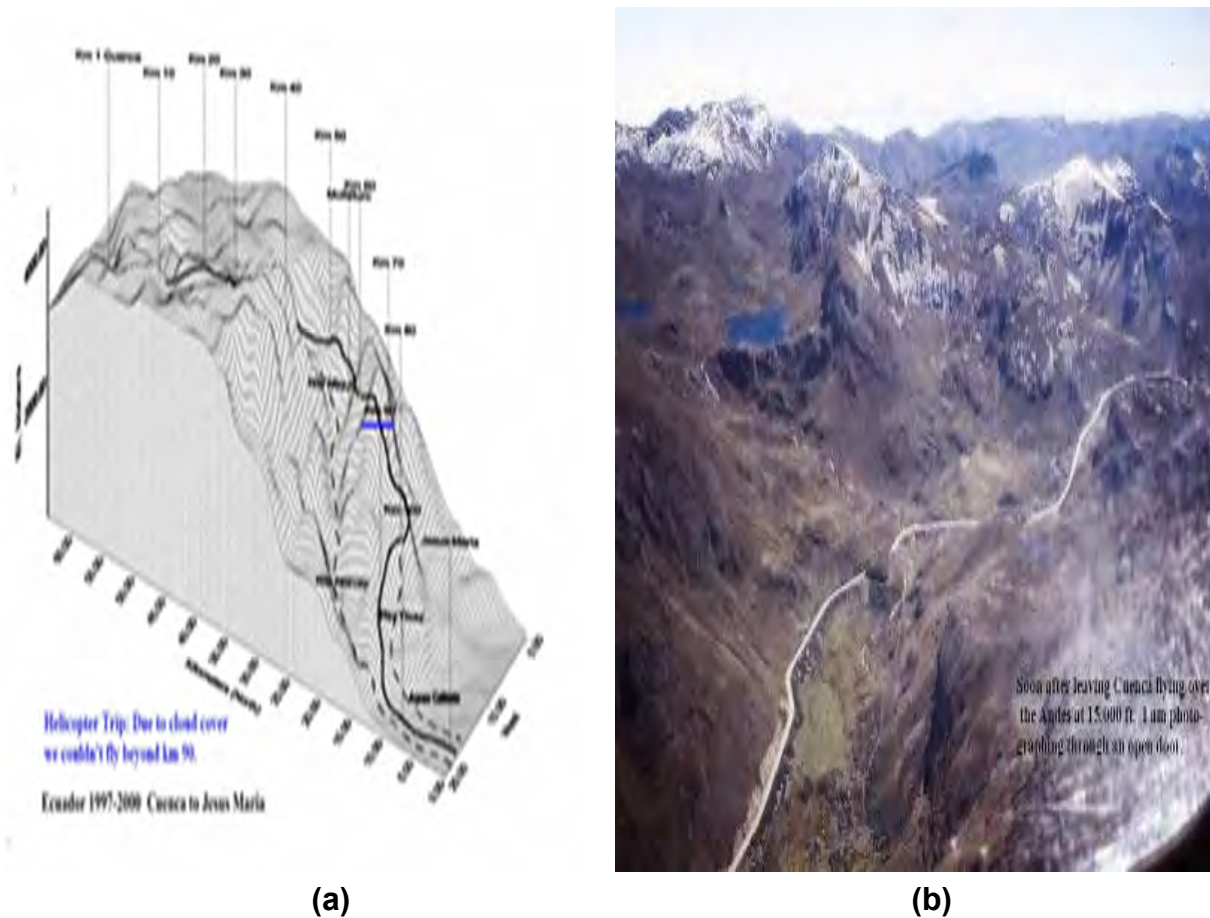
FIGURE 1 Repair and stabilization works of the slope's failures along the Ecuadorian Andean C-M-E road caused by the 1998 El Niño storm. The intensity and duration of this storm were 20% to 35% over the intensity of previous registered storms. Therefore, it is concluded that the ongoing climate change contributed to the storm's high intensity and to the significant slope damages along the road; (*a*) helicopter trip along the C-M-E road; (*b*) Flying at elevation of 15,000 above the C-M-E road (*continued on next page*).

(c)   (d)

FIGURE 1 (*continued*) (*c*) Typical slope failure of approximately 25*10⁵ M³; and (*d*) Typical slope failure of approximately 25*10⁵ M³. Notes: The photos present the general locations and typical slope failures along the Ecuadorian C-M-E mountainous LVR taken on November 1999, after the end of the Ecuadorian Andean 1998 El Niño flooding damages. Also, during this visit the road was already open following continuous road repair and maintenance activities, including clearing and removing of cohesive-granular and rocky slopes fallen materials.

What's more, to produce reliable susceptibility risk maps at areas with a high risk of occurrence of natural disasters such as the Ecuadorian 1998 El Niño storm period along the C-M-E road, it was necessary to survey the related engineering data at close locations of intervals of less than 100 meters (*21, 22*). Donors have supported producing susceptibility risk maps to optimize network level planning of disaster-preparedness works (*11–15*).

4.   Produce cost-effective contracting procedures for job orders (UJO) for urgent works that will be implemented by a multi-skilled project team. These reconstruction/rehabilitation and disaster-prevention works are competitively bid with indefinite quantities, indefinite delivery periods, and with fixed unit prices or fixed costs along with related performance indicators. The UJO contracting procedures are different from traditional contracts in terms of being in place before the completion of the designs, using more affordable and more achievable quality indicators (*11–13*). UJO cover all the types of disaster-related works, including construction, repair, maintenance, and rehabilitation of roads, bridges, slopes, seawalls, surface, and

subsurface drainage works, under one single contract. See Tables 4 and 5, which present data on preventive and remedial measures of road-slope failures in selected low- and middle-income countries (*14–26*). Also, the advantages of the UJO contracts include the use of (1) competitive contractual procedures of fixed unit prices, performance-based specifications, transparent pay equations, and related quality control procedures, and (2) qualified local contractors with transparent and financial incentives to quickly produce superior quality products and, thus, to continue to be qualified to receive more work. Also, UJO contracts are structured to encourage the participation of indigenous small businesses that cannot compete for larger projects.

5.   Develop institutional preparedness capabilities, seeking to ensure that road/transport agencies will have adequate institutional capacities to administer the planning, programming, construction, maintenance, quality control, reporting, and monitoring procedures of cost-effective disaster-prevention activities. Most of these activities are contracted out, see selected LVR slope stabilization treatments activities in Tables 4 and 5.

6.   Carry out natural-disasters preparedness training programs, seeking to support the road/transport agencies, contractors, and affected communities to produce cost-effective and affordable preparedness works. A good reference for an effective and efficient administration of natural-disasters preparedness planning and implementation activities is the Japan International Cooperation Agency (JICA) (*27*, *28*). These references indicate that the number of fatalities from typhoons in Japan had gone down from approximately 45,000 to almost zero during the last 60 years. Nevertheless, these impressive capabilities were not sufficient to avert the damages of the 2011 Japan's magnitude-9 earthquake and tsunami (with water-waves height of around 9.3 m) that caused 18,000 fatalities and significant destruction to infrastructure and facilities. In this regard, it is important to indicate that Japan has experienced 20% of the earthquakes with magnitude-6 and higher that occur in the world and has been exposed to uncountable natural disasters like typhoons and active volcanoes. Japan has a long history of awareness of the importance of disaster risk reduction, and today, Japan is one of the few countries in the world where there is widespread initiative-taking vis a vis risk-reduction measures to mitigate disasters. In fact, Japan is using its experience and expertise in the field of natural-disaster preparedness to help other countries determine how best to incorporate disaster risk reduction into their society's cultural norms.

7.   Measure the effectiveness of disaster management organizations, including IADB and the U.S. Agency for International Development, by supporting transparency, reliable reporting, and cost-effectiveness of LVR investments and maintenance works (*4–15,18*). Donors and their member countries seek to transfer more responsibilities to

the private sector; areas of responsibilities include emergency evacuation, power supply, water and food services, and ambulance and medical trauma services. A recent Israeli study on the effectiveness of the organizational administration of natural and man-made disaster management concluded that a disaster organization's stated goals, on which most measures of organizational effectiveness are based, are not necessarily those perceived or even used by its related stakeholders to gauge effectiveness (*26*). Moreover, one of the lessons learned from the Israeli case study indicated that a third of the population is willing to pay directly for the disaster management services performed by the private sector. Considering this lesson, donors continue to evaluate how to best involve all the stakeholders—including the private and the public sectors and especially the affected communities—in the management of natural and man-made disasters in selected LAC and African low- and middle-income countries.

Risk Management Considerations of Mountainous Landslides of LVR Along the Ecuadorian Andean Region

Thunderstorms, excessive rainfall, and the quick rise of ground water levels often trigger avalanche or debris flow that caused the closure of rural roads in the Andeans mountains areas of Ecuador. Similar conditions triggered the closure of selected mountainous sections of the Interstate-70 in Colorado in the United States (*16–18*, *20–25*). In both cases, the mountainous instability occurred at altitudes between 2,200 and 3,500 m above mean sea level. Interstate 70 was closed for 25 h. In Ecuador, sections of the C-M-E road were closed for several days to 2 weeks (see typical slope failures in Figure 1*c* and *d*). The rainfall and hydrogeological characteristics of both cases indicate that they are susceptible to debris-flow hazards. Typically, debris flows are initiated in tributary drainage basins, and the debris accumulation and downward acceleration occur toward the basin mouth. In the Ecuadorian case, the debris flow covered extensive lower areas, clogged rivers and creeks, and caused heavy agricultural damages. Historic observation of the debris flows along the Ecuadorian C-M-E road and Interstate 70 has shown that the mean recurrence interval in years (*y*) is approximately: $y = 19,400/EXP^{(4.67x)}$, where *x* is the Melton's number (*24, 25*), defined as: $x = H/(A)^{(0.5)}$, H is the basin height above the fan, and A is the basin area above the fan. Basins that are small and steep have higher Melton values than basins that are large and have low to moderate H values (moderate relief). In the C-M-E road case, the Melton numbers along 15 locations, during the 1998 El Niño storm and rainy season, varied between 2.0 and 2.6, indicating that the mean recurrence interval of debris flow

varies between 1 to 18 months. Eventually, about 15 major debris avalanches occurred during this period and blocked the C-M-E road for several days to a couple of weeks.

The probability (P), defined as the percentage chance of one or more major debris flows occurring on an individual location during a specific time (t) and a mean recurrence interval (y), is calculated using the Poisson probability model, $P = 1-e^{-(t/y)}$. The probability in terms of the percentage chance of occurrence of one or more debris flows is shown in Table 1. Considering these probability indicators, together with other affordability concerns during the maintenance and remediation/reconstruction works of the C-M-E road, drove the selection of a mean recurrence interval of 50 years with an occurrence probability of less than 40% during a life expectancy of 25 years. This 25-year slope stability life expectancy design assumes that the new hazard monitoring procedures along the C-M-E road will further reduce accidents and major road closures. Lessons learned, in terms of 22 years of monitoring the road's slope stability performance, have confirmed this conclusion.

The most common methodology used to determine the FS of the stability of road mountainous slopes considers stress-strain and limit equilibrium relationships. However, viscous-plastic engineering laws, critical velocities and critical accelerations of the rocks, soils, and debris materials are the newer procedures to evaluate the probabilities and predict catastrophic landslides (*6–18*, *19–22*). The definition of critical values of the velocities (V_{crit}) and of the accelerations (a_{crit}) of catastrophic landslides are as follows: $V_{crit} = 5$ cm/day and $a_{crit} = 5$ mm/day^2. Therefore, when the V_{crit} or when the a_{crit} reaches these values, an uncontrolled or catastrophic landslide could be imminent. Thus, in a populated mountainous area, when early signs of a possible landslide appear, monitoring of slope movement is critical. Landslide early signs include development of tensile cracking and soil-rock-mass movements (SRMM) of 2 mm/day to 5 mm/day. In these cases, Table 2 recommends a slope evaluation of at least once a month. Also,

TABLE 1 Probability: Percentage Chance of One or More Debris Flows on an Individual Slope Location During a Specified Time

Mean Recurrence Interval (years)	1 year	10 years	25 years	50 years	100 years
10	9.52	63.21	91.79	99.33	100.00
50	1.98	18.13	**39.35**	63.21	86.47
100	1.00	9.52	22.12	39.35	63.21
200	0.5	4.88	11.75	22.12	39.35

**TABLE 2 Monitoring and Alert Indicators of Possible
Landslides (SMRA < 2 mm/day^2)**

Alert Level	SRMM (mm/day)	Status	Monitoring
0	2<	Normal	1/month
0	2–5	Normal	1/10 days
1	5–15	1st alert	1/day
2	15–25	2nd alert	2/day
3	25–40	1st warning	(2–5)/day
4	40–50	2nd warning	1/hour
5	>50	Consider evacuation	Continuously

Table 2 presents a useful guideline of monitoring and alert procedures of possible catastrophic landslides. For example, when the SRMM is between 5 mm/day and 15 mm/day, a daily monitoring and first alert is recommended. Evacuation of the affected population should be considered when SRMM > 50 mm/day or when the soil-rock-mass acceleration (SRMA) > 5 mm/day^2. Also, when the SRMA is over 2 mm/day^2 to 3 mm/day^2, the monitoring and the alert procedures should be more conservative, and an hourly monitoring should be considered. The most practical procedures to monitor the SRMM and the SRMA values was using a satellite-based very-small-aperture-terminal (VSAT) with centrally managed hub. The VSAT system monitored and reported movements and changes of inclinometers, piezometers, and other defined benchmarks. The VSAT used a remote site dish antenna with a diameter of less than 1.2 m, and the central management unit could monitor unlimited locations of unstable slopes. The VSAT (*11, 12, 20, 21*) and other new technologies (including drones and related phone-based apps and web-based software tools) are independent from terrestrial infrastructure and could provide consistence and continuous data about ground and slope movements. The bureau of seismology in countries such as Ecuador, Peru, and China had implemented VSAT networks for earthquake monitoring, including Intranet information exchange and voice applications, essential for risk management of catastrophic events (*29, 30*).

In critical conditions, the relationship linking the critical time (Tc), just before the occurrence of the slope failure, to the SRMM is LOG (Tc) = (A-LOG (SRMM))/B (*14, 15*). Where, A is a site-specific technical parameter and B is an engineering correlation factor that varies between 0.5 and 1.0. In the specific case of the Ecuadorian C-M-E road, during the 1998 El Niño heavy rains period, the SRMM varied from 25 mm/day to 45 mm/day, and the estimated Tc values varied between 50 h to 200 h. In this specific geological and climatical area of the Ecuadorian Andean region, the volume of the landslide-V, in units of cubic meters-M^3, is usually a function of the friction factor (FF) of the local cohesive-friction soils (*20, 21, 23*). The approximate relation between FF and V

TABLE 3 Relationship Between the FF and the Landslide Volume (V-in 10^5 m^3) Along the Ecuadorian Cuenca-Molleturo-Empalme (C-M-E) Road

V (M^3)*10^5	1	2.5	5.0	10.0	25.0	50.0
FF	0.79	0.77	0.75	0.74	0.72	0.71

is given in Table 3. As an example, an FF value of 0.72 was associated with a landslide volume of approximately 25*10^5 cubic-meter along the Ecuadorian C-M-E road. In this regard, typical slope failures of the C-M-E road, taken in November 1999, are given in Figure 1.

Other Slope Stabilization Works of LVR in Selected Developing Countries

Practical slope stabilization works were implemented in the seismic areas of the Andean region encompassing Ecuador, Colombia, and Peru, that are prone to the flooding damages of the El Niño-related storms; these slope-stabilization efforts include cutting and changing the geometry characteristics of existing unstable surfaces with low values of the FS and applying the Janbu method of slope stability analysis to reach FS values of over 1.2 (*18–21*). Also, in areas with unstable residual soils surface laying over weathered or unweathered rock layers, slope-surface drainage improvements are used to reduce the pore pressures (pore air and pore water) within the unstable soils and thus increase the FS to over 1.2. Techniques to reduce pore pressures are used in seismic areas of Ecuador, Colombia, and Peru to address cyclic loads induced by earthquakes that have decreased the soil strength and the related stability of slopes (FS < 1.2) by increasing pore pressures (pore air or pore water). Also, in extreme cases, increase of pore pressures could lead to liquefaction.

Other road-slope stabilization activities include low-cost solutions of sediment-debris traps used successfully in the hilly and mountainous areas of the LAC countries to avert clogging of the drainage system networks (*19*). These sediment-debris traps include check dams and gross sediment-pollutant traps, consisting of a combined sediment basin and a trash rack. These debris traps are usually located at the upstream or downstream of the constructed stormwater pipe or the drainage channel networks, as applicable to avert uncontrolled slope-surface and related erosion failures. Table 4 presents typical preventive and remedial measures of slope failures used in selected LAC countries, including Colombia, Peru, Ecuador, Bolivia, Barbados, and Dominica.

**TABLE 4 Preventive and Remedial Measures of LVR Slope
Failures in Selected LAC Countries (*15–25*)**

Failure Definition	Preventive Activities During Construction	Remedial Measures After Road-Slope Failures
Rock fall	Install base erosion protection; control blasting excavation; install rock bolts and straps, cables, and concrete support of large masses; remove loose blocks; shotcrete weak strata.	Permit fall, clean roadway; install rock bolts, straps, and concrete supports; remove loose blocks; install impact walls.
Soil fall	Install base erosion protection.	Install retention and drainage structures (*18*).
Planar rockslide[*]	Small volume: Remove or bolt. Moderate volume: Provide stable inclination or bolt to retain. Large volume: Install or relocate drainage systems to avert rockslide.	Permit slide, clean roadway; remove debris to stable inclination or bolt. Install internal drainage or relocate to avert rockslide, seek FS ≥ 1.2, see the A notes.
Planar debris slides	Provide stable inclination, and surface drainage control (*18*); provide retention for small to moderate volumes and relocate the road alignment for large volumes.	Allow failure and clean roadway; implement preventive measures, including retention structures and surface and subsurface drainage facilities (*18*).
Rotational rockslide[*]	Implement stable inclination, and surface drainage systems (*18*), seek FS ≥ 1.2.	Implement stable inclination, drainage systems; install internal drains, seek FS ≥ 1.2.
Rotational soils slides[*]	Provide stable inclination and surface drainage control (*18*), and/or install retaining structures, seek FS ≥ 1.2.	Permit failure, clean roadway, remove soils, stable inclination. Install surface and subsurface drainage systems, seek FS ≥ 1.2.
Debris avalanche	Prediction and prevention difficult: Treat as debris slide. Avoid and postpone working in high-hazard areas with FS <1.2.	Permit failure, clean roadway. Remove soils and rocks, and stable inclination. Install surface and subsurface drainage systems.
Debris flows	Prediction is difficult: Avert or postpone working in susceptible areas.	Remove debris or relocate the road for small and large scales flows, respectively.

NOTES: The Ecuadorian slopes along the C-M-E road are mainly anisotropic cohesive-frictional materials. In this regard, to produce a practical design of cost-effective preventive and remedial measures of road-slopes failures, the theory plasticity is most appropriate (*17*, *19*, *21*, *22*). Thus, at a state of a slope failure, where FS = 1, it is shown that both the equilibrium equations and the velocity (displacement) equations are hyperbolic, and the stress characteristics coincide with the velocity characteristics (*19*). Therefore, for a practical application, once the strength characteristics of the slope's cohesive-frictional materials have been found for these anisotropic and non-uniform materials, the conventional methods for the isotropic case have been used, with the calculation of the maximum shear stress that produces the lowest FS of the rotational slope failures. Thus, it was concluded that at the state of slope failure, along the Ecuadorian C-M-E road, the approximate relationship between friction factor FF and the landslide volume (V) was determined, and the related results are given in the table.

**TABLE 5 Other LVR Slope Stabilization Activities Used in
Selected LAC Countries**

Activity	Selected LVR Slope Stabilization Activities
Change slope geometry	Reduce the height of the rotational slides. Reduce the inclination of all the soils/rockslides. Add weight to the toe of the cohesive-frictional soils, seek FS ≥ 1.2.
Control surface water	Cover soils surfaces with vegetation. Seal cracks of soil/rock surfaces. Install drainage systems into soils and decomposed rocks surfaces.
Control internal seepage	Implement: (i) Deep wells to rock masses; (ii) Vertical gravity drains and/or sub-horizontal drains to cohesive-frictional rock/soil masses; (iii) Galleries to rock and high-strength cohesive-frictional soils masses; (iv) Relief wells and or toe trenches to cohesive-frictional soils; (v) Blanket drains to fills of soils; (vi) Electroosmosis to cohesive Silty soils; and (vii) Chemical to Clay soils.
Retention elements/structures	Implement the following retention structures into the following road slopes: (i) Concrete pedestals to rock overhanging; (ii) Rock bolts to jointed or sheared rock; (iii) Concrete straps and bolts to heavily jointed or soft rock; (iv) Cable anchors to dipping rock beds; (v) Wire meshes to steep rock slopes; (vi) Concrete impact walls to moderate slopes; (vii) Shotcrete to soft surface or jointed rock; (viii) Rock-filled buttress to strong or soft rocks; (viii) Gabion walls to strong soils/soft rock; (ix) Crib wall to moderately strong soils; (x) Reinforced earth walls to soils and decomposing rocks; (xi) Concrete gravity walls to soils and rocks; (xii) Anchored concrete curtain walls to soils and decomposing rocks; and (xiii) Bored or root piles to soils and decomposing rocks.

Financial Preparedness: Risk-Transfer Considerations

An effective risk-transfer mechanism could support the national or the regional financial preparedness of possible/probable catastrophic events. Risk exposure associated with natural catastrophes is characterized by low frequency and high impact (*4*). This reference provides an in-depth presentation of how losses due to natural disaster catastrophes are insured and who absorbs the costs of compensating the insured assets. The IADB evaluated the areas of financial planning and risk-transfer instruments that support understanding the mechanism of these financial instruments that provide financial protection for the private and public sectors in the LAC region (*3–15*). In this regard, the principal objectives of catastrophic risk-transfer instruments follow:

1. Develop affordable insurance premiums to cover infrastructure and facilities damages associated with natural disasters. This policy supports collaboration among LAC governments, academia, and insurance companies in the development and administration of a reliable database, emphasizing code and safety compliance, risk-

probability, magnitude, and severity of expected catastrophic events. This database addresses local, national, and regional risk assessments that are useful to practitioners and researchers.

2.   Incorporate catastrophic risk exposures into infrastructure and facilities investment projects. Manage, on network and project levels, catastrophic risks through mitigation or specific insurance that could reduce specific projects exposures, including possible flooding and seismic disasters.

3.   Produce country and project-level risk management plans (RMP) of infrastructure and facilities and establish financial cover for higher catastrophe risk layers. Implement the RMP, including affordable and reliable risk-sharing procedures to mitigate risks and attract insurance mechanisms of risk-transfer cover. The lower-level disaster risks could be covered by tax-funded calamity funds as the main source for disaster relief and rehabilitation. Cover for higher-level disaster risks could be carried out through the international financial markets in terms of cat-bonds, risk-swaps, and contingent capitals.

4.   Establish national insurance pools that require mandatory insurance policies. Governments must enforce stringent initiatives of risk management, such as enforcing building codes and effective property registration. Once these policies are implemented, local insurance companies could support local market involvement of the lower risk layers. Insurance pools could cover part of the higher risk layers in international financial markets, through reinsurance contracts, risk-linked securities, contingent and surplus notes.

5.   Integrate risk exposure across neighboring LAC countries. This international insurance pooling collaboration could provide a natural first line toward risk diversification that engages local primary insurance companies in the development of regional insurance companies. In addition, it could provide scale economies to risk financing arrangements in the international financial markets.

Summary and Conclusions:
Vulnerability Reduction and Engineering
Risk Management Considerations

1.   Vulnerability measures the probability of a catastrophic event on people, infrastructure, and facilities due to insufficient prevention, mitigation, and warning procedures. The development pattern of most countries with high rates of poverty, socioeconomic exclusion, and environmental damages, is the leading factor of high vulnerability to natural disasters. This vulnerability has revealed itself during natural-

disaster events, when not enough has been invested in prevention works and alert procedures. Catastrophic failures of roads, bridges, mountainous slopes, and earth dams are usually associated with uncontrolled surface and underground water flows. Once large masses of saturated soils and rocks generate a speed of over 50 mm/day or acceleration of over 5 mm/day^2, a catastrophic event could be imminent, and evacuation of affected communities should be considered in selected Ecuadorian mountainous areas. The planning activity is associated with the use of susceptibility flooding and slope-failure risk maps. These risk maps are essential to reducing the probability of a possible catastrophic event and to reducing its magnitude and severity. In engineering terms, producing an FS > 1.2 is needed to ensure affordable and safe road slopes with a life expectancy that ranges from 5 years to 50 years in the LAC region.

2.   International donors have financed projects aimed at building countries' institutional disaster risk management capacity to reduce vulnerability to natural disasters. As an example, between 1990 and 1999, the IADB financing in Central America disaster mitigation, prevention, and preparedness activities accounted for $350 million. For the most part, this IADB disaster-related lending was not part of emergency or reconstruction lending, but rather for programs that reduce vulnerability in the absence of an event. To further reduce vulnerability, donors have supported using the best practices of preparedness works, considering affordable financial and institutional capacities, to resolve critical problems of risk reduction. The selected priorities have included the following:

    (a) Reconstruction projects aimed at risk reduction and related institutional strengthening (legal, technical, and financial), as well as improvement of land use planning, and enforcement of construction and maintenance codes;

    (b) Modernization of the administration of emergency works, including decentralizing/deconcentrating of responsibilities and the related financial funding given to local governments/municipalities to build knowledgeable consumers and reliable database for risk monitoring and reduction; and

    (c) Strengthening public sector financing and optimizing the mobilization of private resources to support disaster risk reduction.

3.   To further reduce the vulnerability of catastrophic events, the IADB introduced a financial-sector facility of disaster prevention (*12*). This financial instrument assisted LAC countries to adopt an integrated approach toward reducing and managing their risk of natural hazards before a disastrous event occurs; this was done through the following components:

    (a) Risk identification and forecasting, to understand and quantify the vulnerability to natural disasters;

(b) Mitigation, to address the structural sources of vulnerability;

(c) Preparedness, to enhance a country readiness to cope quickly and effectively with an emergency;

(d) Risk-transfer measures to spread financial risks over time and among different actors, including insurance and capital market schemes; and

(e) Transition to new policies, legal and regulatory frameworks, and institutions to building effective national systems that support disaster risk reduction during the ongoing climate change.

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Preliminary Study on Directional and Stress Dependency of Stiffness Characteristics of Granular Roadway Materials

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Stress–strain characteristics of granular geomaterials have a major impact on response to traffic loading of unpaved roads or roads with thinner surface thicknesses [portland cement concrete (PCC) or hot mix asphalt (HMA)] compared to high-volume roads. To provide adequate support under continuous traffic loading, reliable characterization of geomaterial behavior under moving-wheel loads is necessary to help guarantee designed roadway service life without potential damage. Since resilient modulus (M_R = the ratio of cyclic stress to elastic strain) of unbound materials is being used as an input parameter in roadway design procedures as a surrogate for the elastic stiffness modulus, accurate M_R characterization plays an important role in understanding mechanical behavior of granular roadway layers corresponding to various stress states observed in the field. To this end, laboratory experiments have been extensively used to determine M_R values corresponding to different cyclic and confining stresses in simulating continuous traffic loading. For simplicity, the majority of past studies have been limited to conventional testing of geomaterials subjected to cyclic stress only in the vertical direction; this assumes that the major principal stress ($\sigma1$) is in the vertical direction for all possible stress states. Laboratory characterization of granular materials through conventional testing, however, represents only loading conditions in which the moving load is stationary on top of the tested geomaterial location where stress path slope, m, has a constant value of 3 ($m = \Delta q / \mathrm{U}p$; where Δq is the change in the cyclic

stress and Δp is the change in mean stress due to cyclic stress application). Stress states that occur in the field under moving wheel loads, however, have more complex characteristics as a result of continuous rotation of the σ1, and during laboratory testing this can only be accurately simulated by accounting for different stress path slopes (*1–5*). As a result of such rotation, geomaterials experience major principal stress both in the vertical direction and in the horizontal direction [*m* = (–3)] with wheel load becoming an approaching/receding load with respect to the reference geomaterial location. To impose stress states to a geomaterial corresponding to principal stress rotation, cyclic true-triaxial testing systems have emerged as useful tools because they have capability for applying various stress path combinations.

Research studies have shown that geomaterials exhibit cross-anisotropic behavior, i.e., stress–strain characteristics are different in the vertical and horizontal directions (directional dependency) as a result of natural stratification and vertical loadings such as compaction efforts and continuous traffic loading (*6–11*). One result of the cross-anisotropic nature of geomaterials is that their mechanical properties in both vertical and horizontal directions play an important role in roadway performance where they are constantly subjected to principal stress rotation. Cyclic true-triaxial testing is therefore necessary for use in reliable characterization of the stiffness characteristics of unbound materials for advanced stress path testing where geomaterials are exposed to the various stress combinations most likely to occur in the field. Stress dependency of the M_R^v values of unbound roadway layers has been demonstrated in numerous studies (*12–15*). Depending on applied stress levels and gradation, materials may exhibit stress-softening or stress-hardening behavior in vertical and horizontal directions as a result of their deformation characteristics during cyclic stress application. While it has been shown that the M_R^v of unbound granular materials exhibits stress-hardening while stress-softening behavior for the M_R^v of subgrade materials has been observed (*16–17*), there is little in the literature related to softening/hardening behavior of M^h of granular roadway geomaterials, probably because advanced testing equipment is needed for such investigation. Considering the cross-anisotropic and continuous principal stress rotation issues, the investigation of softening/hardening behavior of unbound materials in the horizontal direction has become more necessary for improved characterization of mechanical properties of geomaterials using advanced testing systems such as cyclic true-triaxial testing systems.

In this study, the directional and stress dependency of the stiffness characteristics of two granular roadway surface materials (dolomite and limestone) were investigated. The geomaterials were tested by cyclic true-triaxial equipment (SPAX-3000) to determine vertical M_R (M_R^v), horizontal M_R (M_R^h), and anisotropy ratios (ratios of M_R^h to M_R^v). The

results revealed that unbound materials are cross-anisotropic due to their high M_R^v compared to the M_R^h. Anisotropy ratios for coarse aggregate materials were determined to be 0.04–0.18.

Methodology

A series of M_R tests were performed to investigate the directional and stress dependency of two coarse aggregates (dolomite and limestone) through cyclic true-triaxial testing. Table 1 shows the results of the material characterization test results (sieve analysis, specific gravity, Proctor test) including the soil classification according to the Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO).

Advanced cyclic true-triaxial testing equipment with rigid-rigid-flexible boundary, the SPAX-3000, was used to conduct M_R testing on compacted prismatic specimens. The SPAX-3000 has four load actuators, two vertical (x) and two horizontal (z), that can be independently controlled to enable simulation of principal stress rotations by applying cyclic stresses to the compacted specimens through rigid plates. The system also enables application of confining pressure (σ_y) through an air medium and uses linear variable displacement transformers in each direction to record deformations corresponding to the applied cyclic stresses. Details of the SPAX-3000 testing system can be found in (*18*).

TABLE 1 Physical Characteristics of Materials

Material	Granular Surface Materials	
	Dolomite	Limestone
Gravel (%)	64	64
Sand (%)	31	31
Fines (%)	5	5
G_s	2.65	2.69
MDU (kN/m³)	22.5	22.8
OMC (%)	4.4	3.5
USCS	GW	GW
AASHTO	A-1-a	A-1-a

NOTE: Fines = silt and clay, G_s = specific gravity, MDU = maximum dry unit weight; OMC = optimum moisture content; USCS = Unified Soil Classification System; AASHTO = American Association of State Highway and Transportation Officials.

Two specimens of dimensions 152 mm × 152 mm × 304 mm were prepared at optimum moisture content using a vibratory hammer in 6 equal layers to reach maximum dry unit weight. Following specimen preparation, the compacted geomaterials were transferred to the cyclic true-triaxial cell to complete the SPAX-3000 assembly procedure. To simulate the principal stress rotation as a result of approaching/receding wheel load, specimens were tested for M_R^h [m = (–3)] following M_R^v (m = 3). For the dolomite and limestone unbound granular materials, stress levels in the AASHTO T307 corresponding base aggregate layer were used to determine stiffness in both directions. Testing sequences for the materials are shown in Table 2 for m = 3 and (–3). Following M_R testing, Equations 1 and 2 were used to calculate M_R^v and M_R^h, respectively.

$$M_R^v = \frac{\Delta \sigma_x}{\varepsilon_x} \tag{1}$$

$$M_R^h = \frac{\Delta \sigma_z}{\varepsilon_z} \tag{2}$$

TABLE 2 Loading Testing Sequences

| Sequence No. | Granular Road Surface | | |
| | σ_y | m = 3 $\Delta\sigma_x/\Delta\sigma_z$ | m = –3 $\Delta\sigma_x/\Delta\sigma_z$ |
	kPa	kPa	kPa
0	103	103/0	103/0
1	21	21/0	0/21
2	21	41/0	0/41
3	21	62/0	0/62
4	35	35/0	0/35
5	35	69/0	0/69
6	35	103/0	0/103
7	69	69/0	0/69
8	69	138/0	0/138
9	69	206/0	0/206
10	103	69/0	0/69
11	103	103/0	0/103
12	103	206/0	0/206
13	138	103/0	0/103
14	138	138/0	0/138
15	138	276/0	0/276

NOTE: m = stress path slope; σ_y = confining pressure; $\Delta\sigma x$ = vertical cyclic stress; and $\Delta\sigma_z$ = horizontal cyclic stress.

where

$\Delta\sigma_x$ = vertical cyclic stress;

ε_x = recoverable deformation in the x direction;

$\Delta\sigma_z$ = horizontal cyclic stress; and

ε_z = recoverable deformation in the z direction.

Findings

To investigate directional and stress dependency of granular road surface materials, M_R^v and M_R^h results were presented with bulk stress [$\theta = \sigma_1$ + intermediate principal stress (σ_2) + minor principal stress (σ_3)]. Figures 1a and 1b show M_R (M_R^v and M_R^h) variation with bulk stress for dolomite and limestone granular road surface materials, respectively.

  The results showed that the stiffness of tested materials is stress-dependent in both vertical and horizontal directions independent of material type (dolomite, limestone), and it was also observed that stiffness characteristics of the tested materials are direction-dependent, i.e., cross-anisotropic. While it can be seen in Figure 1a that the stiffness of the dolomite granular road surface material increased in both directions with an increase in bulk stress, it was also observed that for the same confining pressures (Sequences 1–3, 4–6, 7–9, 10–12, 13–15) an increase in cyclic stress caused a decrease in M_R^v. This behavior can be explained by considering the effect of the type of cyclic true-triaxial
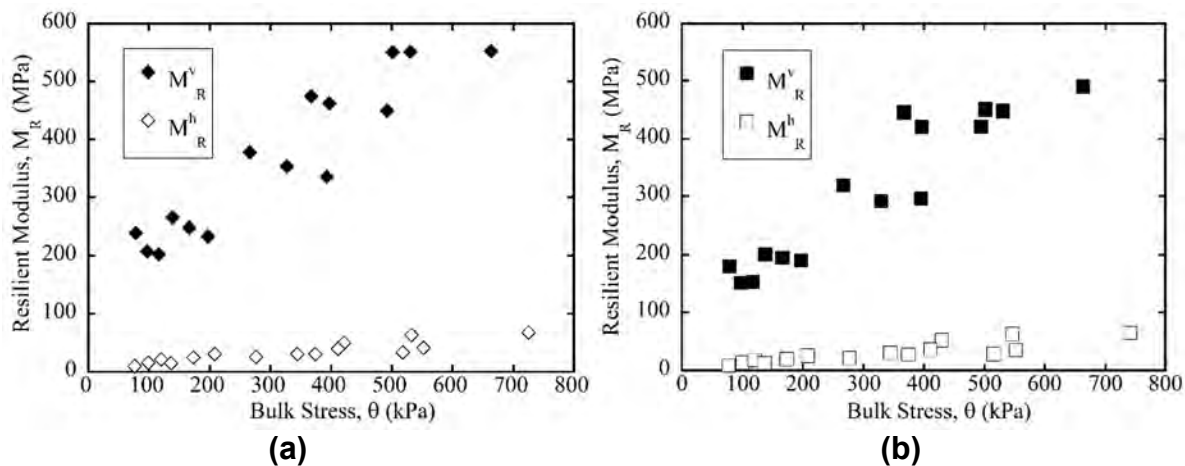
FIGURE 1 Vertical and horizontal resilient modulus versus bulk stress: (a) dolomite base and (b) limestone base.

boundary condition (rigid–rigid–flexible) in which the two compacted-specimen faces in the *y*-direction were confined by air instead of rigid plates, possibly decreasing the contribution of the confining pressure under high cyclic stresses resulting in higher elastic deformations for the same confining pressure. In the horizontal direction, the M_R^h of the dolomite material, unlike M_R^v, exhibited slight stress-hardening behavior. This behavior can be caused by the strengthened structural capacity of granular materials in the horizontal direction as a result of application of cyclic stresses in the vertical direction (*m* = 3). It is believed that the case representing receding moving wheel load [*m* = (–3)] exhibited stress-hardening behavior because the granular material had already become more packed, resulting in less elastic deformation during the *m* = 3 testing program. As can be seen from Figure 1*b,* limestone material showed similar trends in the vertical and horizontal directions; these trends were also observed for the dolomite material.

Conclusions

This study revealed that M_R of granular road surface materials are both direction and stress-dependent, and advanced testing equipment is required to fully characterize the behaviors of these geomaterials and thereby ensure reliable roadway design. The results of the study can be summarized as follows:

- Granular road surface materials exhibit different hardening and softening characteristics in vertical and horizontal directions, and their stiffness is stress-dependent.
- The stiffness of the dolomite and limestone unbound geomaterials increased in both horizontal and vertical directions with an increase in bulk stress, although it was observed that for the same confining pressures (Sequences 1–3, 4–6, 7–9, 10–12, 13–15), an increase in cyclic stress caused stress softening in M_R^v while in the horizontal direction, M_R^h of both dolomite and limestone geomaterials exhibited stress-hardening behavior.
- M_R of dolomite and limestone materials exhibited directional dependency, i.e., the stiffness characteristics of geomaterials were different in the vertical and horizontal directions.
- The highest M_R was observed in the vertical direction independent of material type. It was observed that M_R^v is approximately 7 to 25 times higher than M_R^h for dolomite and 6 to 21 times higher for the unbound limestone geomaterials.

- Anisotropy ratios for coarse aggregate materials were determined to be 0.04–0.18.

It is believed that the results of this study will contribute to the understanding of directional and stress dependency of the stiffness characteristics of geomaterials under both compression and extension tests, which can be used in roadway-design procedures to enable a more realistic approach to simulate the load conditions that occurred as a result of continuous traffic loading.

Acknowledgments

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Bio-Enzyme Stabilizers Comparison with Conventional Granular Layers for Pavement Design of Low-Volume Rural Roads

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Traditional flexible pavement specifications require high-quality aggregates in both base and subbase courses. High-quality aggregates are getting increasingly scarce and expensive in many states in India. In many cases, locally available aggregates are not satisfying the specifications, and aggregates must be hauled from long distances. This significantly increases the cost of construction and subsequent maintenance and rehabilitation. Thus, using locally available marginal materials is a possible sustainable solution. Marginal materials are not in full accordance with the specifications for normal road aggregates but can be used successfully either in particular conditions, made possible because of climatic characteristics, or subject to a specific treatment. Hence, the current study attempted to improve the properties of locally available marginal aggregate (moorum) by adding different bio-enzyme stabilizer combinations of cement. Summarily, the work involves increasing the strength of moorum expressed in California bearing ratio (CBR) and unconfined compressive strength (UCS) value and the durability test to find the resistance to weathering action. It is concluded that there is a 32% reduction in the cost when we design with Terrasil, Zycobond, and cement combination compared to the conventional granular layer. Similarly, it is a 6% reduction in the cost compared with soil stabilized with cement.

Introduction

Low-volume rural roads play a significant role in the infrastructural improvement of the nation as it brings rapid transformation along its path and changes socioeconomic structure, demographics, and environment. In India, these roads are built with little

emphasis on preserving the environment, whereas Western nations have identified the importance of preserving nature. No justification can be offered in the name of progress if it denudes and depletes the environment that allows its sustenance. Green and environment-friendly technologies are available that can create a permanent impact on our environment, as well as end the depletion of good-quality conventional material. Because of the immense construction of roads in India, high-quality aggregates are getting scarce and expensive in many states.

In many cases, locally available materials are not suitable to use. The use of marginal materials in the subbase and pavement base represents a value-added application compared with their frequent waste nature, making these aggregates competitive against conventional materials and reducing the importance of hauling costs over long distances. For this purpose, some stabilization may be necessary to improve their performance. Marginal aggregates do not have the required specifications but can be used by modifying normal pavement design and construction procedures. With this background, the present investigation is aimed to study the effect of the usage of marginal materials for the construction of pavement by replacing it with conventional aggregate. In the present study, blended moorum soil and virgin aggregates (VA) are characterized by stabilizing with Terrasil, Zycobond, cement, and RBI Grade 81 as additives. Each blended material is stabilized with varying proportions of Terrasil, Zycobond, cement, and RBI Grade 81 materials. Basic engineering tests are performed to determine the properties of blended materials.

Literature Review

Patil et al. (*1*, *2*) dealt with stabilizing clayey soil by utilizing fly ash and RBI Grade 81 to enhance the geotechnical properties of soil. They inferred that the fly ash and RBI Grade 81 utilized for soil adjustment indicate a great change in soil properties. Mallikarjun et al. (*3*) concluded that RBI-81 is an effective stabilizer for enhancing the geotechnical properties of lateritic soil and black cotton soil. Rafique et al. (*4*) analyzed the soil classification and earthwork characteristics for two soil types representing pulverized local and transported soil with and without TerraZyme. Most researchers have found that there will be an enormous increase in the strength parameter for the different soils with different stabilizers. A few studies have been done on the effectiveness of the curing period and improving the soil's characteristics. However, very few studies have been done on the utilization of moorum, laterite, gravel, shale, etc., for the construction of subbase bio-enzyme stabilizers for providing waterproofing and calcium-based

additives like cement or RBI grade 81 for improving the strength properties. Therefore, taking this into account, the present study aimed to explore the characterization of blended moorum soil and VA by stabilizing with Terrasil, Zycobond, Cement, and RBI Grade 81 bio-enzyme additives (Figure 1).

Research Approach

The first trial of the stabilizer was the combination of Terrasil, Zycobond, and cement by changing the two different dosages of Terrasil, Zycobond with dosages of 0.3 kg/m^3 and 0.6 kg/m^3 and four different dosages of cement (1%, 2%, 3%, and 4%). The second trial of the study was stabilizing with cement with dosages of 1%, 2%, 3%, and 4%. The third trial of the study was stabilizing with RBI Grade 81 with dosages of 1%, 2%, 3%, and 4%, as shown in Figure 2.
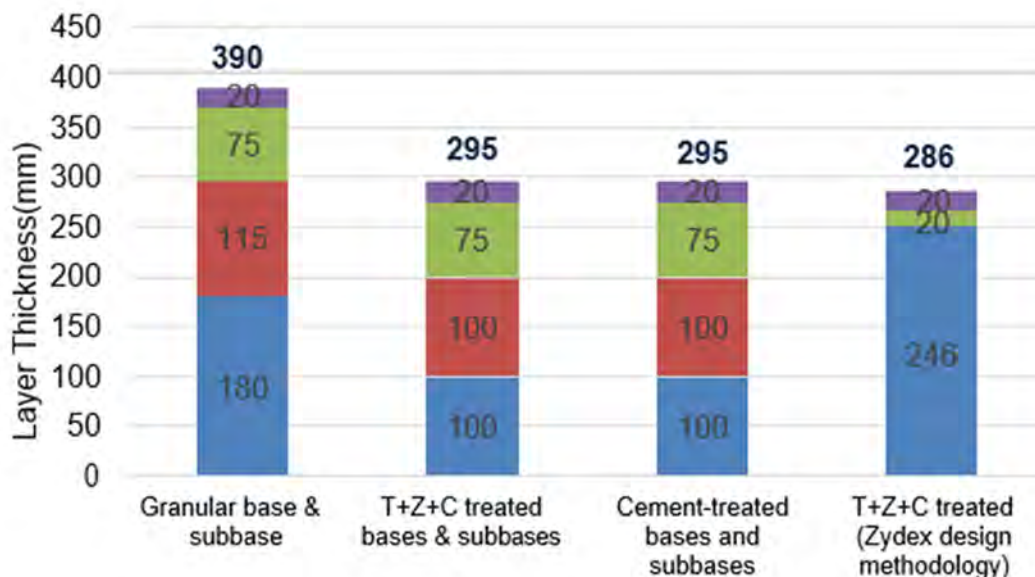
FIGURE 1 Pavement design with different combinations of bio-enzymes.
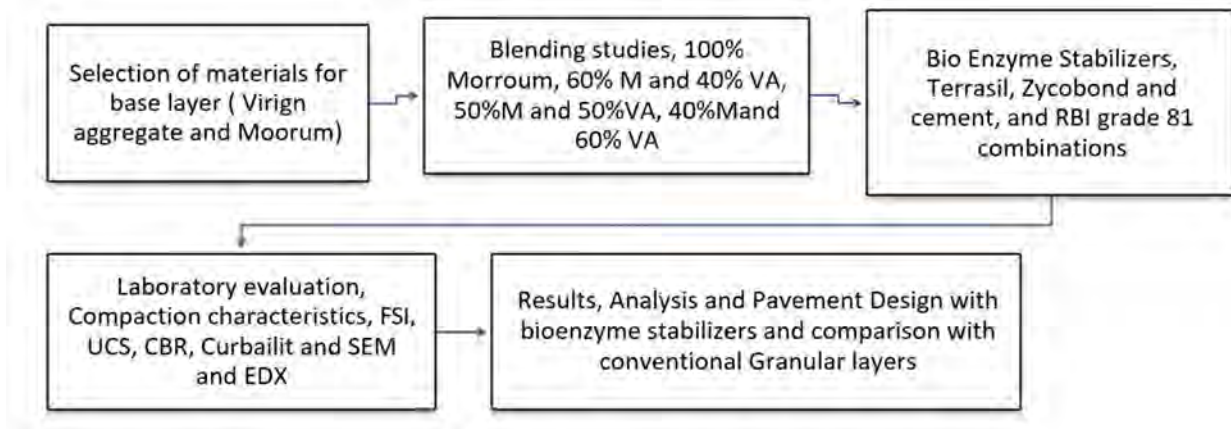
FIGURE 2 Research approach.

Discussion on CBR-UCS and Durability Aspects

Indian Roads Congress (IRC): SP:72-2015 (*5*) recommends that the minimum CBR of the base layer is 80% and the UCS value should be more than 2.76 MPa. Similarly, for the subbase layer CBR value is supposed to be a minimum of 20%, and the UCS value 1.7 MPa as specified by the Ministry of Rural Development (*6*) specifications. This can be achieved by stabilizing the blended moorum and VA combination. Based on the result obtained from the CBR test, it is concluded that out of 16 combinations, soaked CBR reached 80% of the unsoaked CBR for the soil stabilized with 0.6 kg/m^3 Terrasil, 0.6 kg/m^3 Zycobond, and cement. Out of 16 combinations, eight combinations of soaked CBR reached 80% of the unsoaked CBR for the soil stabilized with 0.3 kg/m^3 Terrasil, 0.3kg/m^3 Zycobond, and cement. Also, three combinations of soaked CBR reached 80% of the unsoaked CBR for the soil stabilized with only cement. Out of 16 combinations, eight combinations of soaked CBR got 80% of the unsoaked CBR for the soil stabilized with RBI Grade 81.

Similarly, out of 4 combinations, one combination of soaked CBR reached 80% of the unsoaked CBR for the soil stabilized with 0.6 kg/m^3 Terrasil and 0.6 kg/m^3 Zycobond. Comparing all the results obtained from the CBR that the soaked CBR value of soil stabilized with 0.6 kg/m^3 Terrasil, 0.6 kg/m^3 Zycobond, and cement gave better results among all combinations of the stabilizers. Figure 3 shows the typical CBR, stabilizing with 0.6 kg/m^3 Terrasil, 0.6 kg/m^3 Zycobond, and cement. From the strength perspective, almost all combinations reached the CBR value of more than 100%. Among all, soil stabilized with 0.6 kg/m^3 Terrasil, 0.6 kg/m^3 Zycobond, and cement is gaining more strength than other stabilizers. The CBR and UCS value results replicate why the CBR test is not a comparative strength parameter for stabilized soils. Still, in
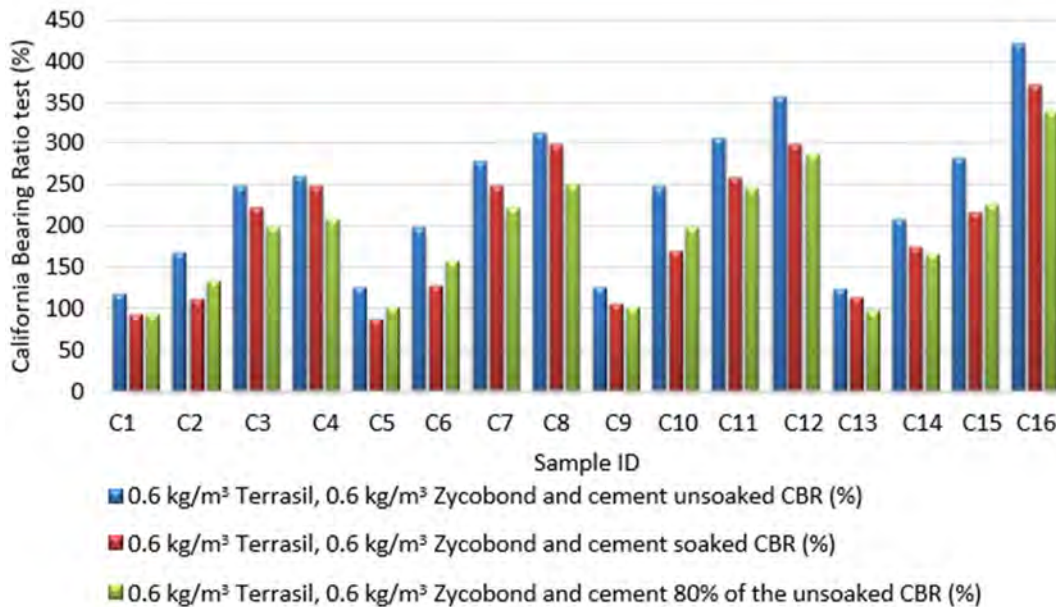
FIGURE 3 CBR-Stabilizing with 0.6 kg/m Terrasil, 0.6 kg/m Zycobond, and cement.

UCS results, very few combinations crossed the minimum requirement of 2.76 MPa for the base layer. UCS test results indicated a need for more percentage of the stabilizer for cement stabilization and RBI Grade 81 stabilization. Based on the observed results of the UCS for the stabilized blended soil materials, the optimum dosage of the stabilizer is the combination of Terrasil, Zycobond, and 4% cement for the blended combinations of 60% moorum and 40% VA, 50% moorum and 50% VA, and 40% moorum and 60% VA. From the economic point of view, 60% moorum and 40% VA stabilized with 0.3 kg/m^3 Terrasil, 0.3 kg/m^3 Zycobond, and cement is the better combination for the construction of the base layer. Based on the durability test results, it is concluded that from the economic point of view, the optimum stabilizer dosage is 0.3 kg/m^3 Terrasil, 0.3 kg/m^3 Zycobond, and cement as a better combination for the construction of base and subbase layer. Figure 4 shows the typical durability test results for 0.6 kg/m^3 Terrasil and 0.6 kg/m^3 Zycobond.
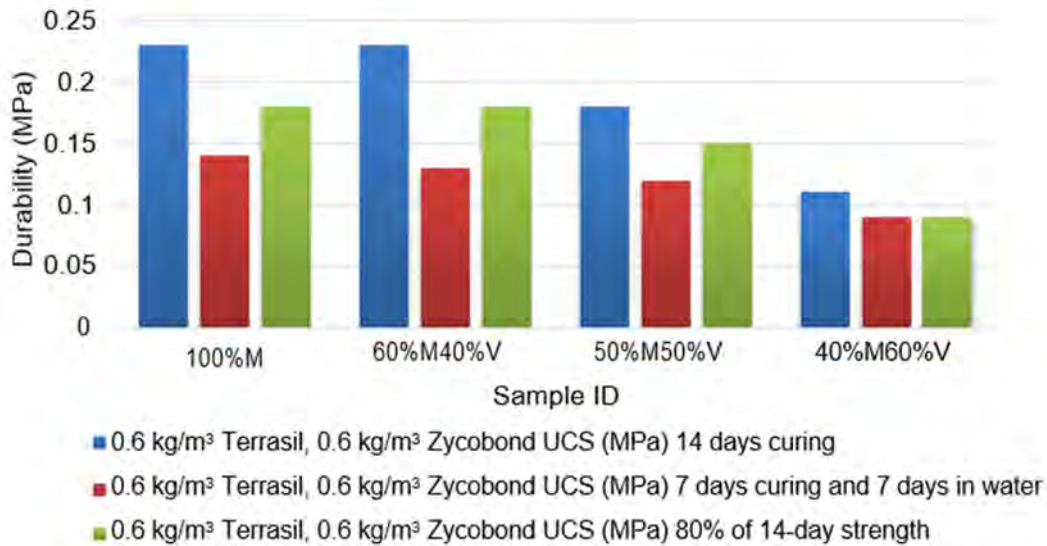
FIGURE 4 Durability test results for 0.6 kg/m³ Terrasil, 0.6 kg/m³ Zycobond.

Pavement Design

The conventional pavement thickness was determined using IRC: SP:72-2015, corresponding to the traffic of 1 million standard axle (MSA) and the subgrade CBR of 7% to 9%. The cross-section details are open-graded pre-mix concrete (OGPC) as a surface layer, water bound madam (WBM) Gr-III as a subbase, and WBM Gr-II base layer with a total thickness of 410 mm (Option 1). The thickness of the pavement for cement-treated base and subbase are OGPC as a surface layer, wet mix macadam (WMM) as a crack relief layer, 60% moorum and 40% VA treated with 0.3 kg/m³ of Terrasil 0.3 kg/m³ of Zycobond with 4% cement as a base layer, 100% moorum with 3% cement and 0.3 kg/m³ Zycobond as a subbase layer is 315 mm (Option 2). Similarly, the thickness of the pavement for cement-treated base and subbase are OGPC as a surface layer, WMM as a crack relief layer, 50% moorum and 50% VA treated with 6% cement as a base layer, 100% moorum with 4% cement and as a subbase layer is 315 mm (Option 3). Similarly, based on the Zydex design methodology according to Australian specifications (Option 4). The pavement design is done by replacing the base and subbase layer with an equivalent layer using the Odomarks equivalent layer of thickness.

According to the Zydex design treated with Terrasil, Zycobond, and cement treated equivalent thickness of the base layer by providing OGPC as a surface layer, a grit layer of size 13.2 mm down as crack relief layer, waterproofing of the top layer of the stabilized soil using Terrasil and Zycobond and water in the ratio of 1:1:200 at 3 ltrs/m²,

50% moorum and 50% VA treated with 0.3 kg/m^3 of Terrasil 0.3 kg/m^3 of Zycobond with 4% cement as an equivalent base layer is 306 mm. Table 1 shows the whole idea of the various design methodologies. For the granular base and subbase, CBR is considered a design parameter. UCS is a strength parameter for the cement-treated base and subbase design. For the base layer, the UCS should be more than 2.76 MPa; similarly, for the subbase layer, the UCS value should be 1.7 MPa. For the Zydex design methodology, both CBR and UCS are considered for pavement design.

Based on the comprehensive study, it is concluded that the effect of the soaking is less for the soil treated with Terrasil, which will make it a waterproofing surface, and the sticky nature of the Zycobond will restrict the propagation of the shrinkage cracks formed due to hydration cement. UCS test results reveal that except for the stabilizer combination of A1 for other combinations, there is a requirement for more dosage of the stabilizer. Based on the durability test, the soil treated with a Terrasil, Zycobond, and cement combination is less affected due to weathering action than other stabilizer combinations. There is a 32% reduction in the cost when we design with a Terrasil, Zycobond, and cement combination compared to a conventional granular layer. Also, it is a 6% reduction in the cost compared with soil stabilized with cement. Considering

TABLE 1 Design Methodologies

For Traffic 1 MSA and Subgrade CBR, 7% to 9% (Conventional Design)	
Pavement Design IRC: SP: 72-2015	CTB and SB IRC: SP: 72-2015
20 mm (OGPC)	20 mm (OGPC)
75 mm (WBM Gr-III)	75 mm (Crack relief aggregate layer)
115 mm (WBM-CBR not<100)	100 mm (Cement-treated base)
180 mm (subbase CBR not < 20)	100 mm (Cement-treated base)

Traffic 1 MSA and Subgrade CBR 7% to 9% (Different Options)			
Conventional Pavement Design	Terrasil, Zycobond, and cement-treated bases	Cement-treated bases	Zydex design methodology
OGPC (20 mm)	OGPC (20 mm)	OGPC (20 mm)	OGPC (20 mm)
WBM Grading-III (Graded Metal)	Crack relief layer (WMM)	Crack relief layer WMM	Grit layer of size 13.2 mm down
WBM Gr-II Graded Metal Base	60% M + 40% VA; 0.3 kg/m^3 TS; 0.3 kg/m^3 ZB; and 4% C (base)	50% M+ 50% VA; 6% C (base)	Waterproofing of subgrade top layer
Granular Subbase with Well-Graded Material	100% M+ 0.3 kg/m^3 ZB + 3% cement (subbase)	100% M+ 4% cement (subbase layer)	60% M + 40% VA + 0.3 kg/m^3 TS + 0.3 kg/m^3 ZB and 4% C (equivalent of base and subbase)

Zydex's design methodology, the construction cost is more than the other designs, but the advantage of this methodology is that it can sustain for longer. As per IRC design, Option 2 is the better among all designs (Figure 5).

Acknowledgments

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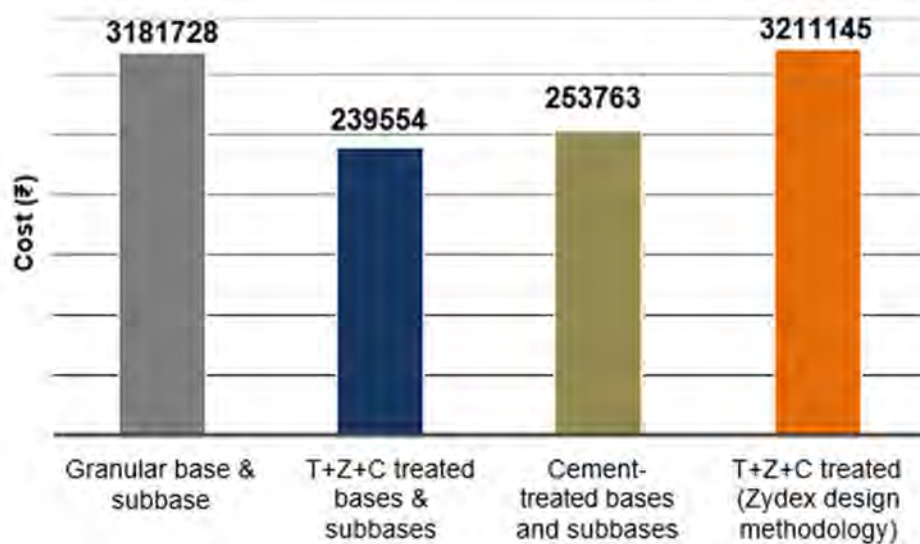
FIGURE 5 The construction cost of different design methodologies.

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Trafficability Study of Full-Scale Geosynthetic Portable Road-Building System

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This study describes the full-scale traffic evaluation of a prototype submersible matting system (SUBMAT) at a test site on the US Army Engineer Research and Development Center's Waterways Experiment Station campus in Vicksburg, Mississippi. The SUBMAT prototype was designed to bridge the gap between high tide and low tide at a beach interface to enable 24-h vehicle offloading operations at an expeditionary watercraft landing site. This unique system is made from common geotextile materials, is filled with indigenous sand using simple commercially available pumps, and creates a robust driving surface. The results of the study showed that the SUBMAT system was able to sustain an accumulation of 1,000 Medium Tactical Vehicle Replacement, 350 Heavy Expanded Mobility Tactical Truck, and over 150 M1A1 main battle tank passes without experiencing any significant damage. The ease of deployment, relatively low cost, and trafficability results could make the SUBMAT a suitable candidate for expedient low-volume roads in austere environments such as stream beds, low-water crossings, recently flooded or flood prone areas, and areas with weak soil.

SAFETY PRACTICES AND APPLICATIONS

Local Road Managers' Safety Practices and Perceptions in North Dakota

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Local agency ownership accounts for almost 75% of the U.S. roadway miles. About 50% of U.S. roadway deaths occurred on the local system in 2016, based on Strategic Highway Safety Plans (SHSPs) and Highway Safety Improvement Program (HSIP) information (*1*, *2*). The local road definition is based on the legacy functional classification system, so it does not represent HSIP eligibility. It does, however, offer a proxy in the road group safety trends. The prominence of local road traffic varies by state considering the all-public-road-safety-system context governing federal road investment planning. States with substantial agricultural and other natural-resource-based industries often rely heavily on local roads with more than 80% of all miles traveled on this network segment (*3*).

Rural local roads are usually identified as low-traffic-volume roads. Hall (*4*) noted that low-volume road classification can vary significantly among jurisdictions. For example, rural local roads typically have traffic of 400 to 2,000 vehicles per day, depending on factors such as proximity to major urban centers or rural economic activity hubs. Despite having fewer total crashes than their rural state road counterparts, rural local roads are susceptible to higher crash risk rates when travel is adjusted for exposure in vehicle miles traveled (VMT) (*5*). In North Dakota, a sharp increase in traffic in state's oil producing region resulted in an unprecedented spike in traffic crashes between 2010 and 2015, with these disproportionately occurring on local roads.

Continued work to heighten local rural road safety awareness is coupled with a need to identify resources for investment, operation, and outreach on this network. The primary federal road infrastructure safety program is the HSIP. Under the Fixing America's Surface Transportation Act, lump-sum apportionments flowed to states with a directive to reduce severe and fatal traffic injuries on all public roads. This directive came with a requirement for strategic and data-driven improvements. The state SHSP

guides allocations of HSIP funds to most effectively reduce roadway traffic injuries. Effectively coupling local road safety program (LRSP) activities with the state SHSP is key in a systemwide approach to severe crash prevention, in terms of priorities and resource allocation. Research here was aimed at understanding county road managers' safety awareness, practices, and investments as leaders in these decisions.

Local county road managers in North Dakota were surveyed to learn more about local traffic safety asset characteristics and management practices, with special emphasis given to the state's oil impact region. Central in this effort was understanding how the LRSP strategy has been adopted as a proven crash reduction measure. The strategy relies on state engagement with local stakeholders to collectively reduce crash injuries in their communities. The LRSP presents a framework for stakeholders to identify, analyze, and prioritize traffic safety improvements for their roadways. Benefits of federal and state support in instituting local road safety practices were evident in funding sources and planning activities reported by counties. Several opportunities to carry the LRSP approach forward in proactive safety opportunities for individual counties with specific crash-type prevention strategies, and for systemwide site-based safety countermeasures, were evident.

Objectives

The goal of this project was to support ongoing local traffic safety efforts by state and local champions by improving their understanding of (1) LRSP implementation and investments, (2) county road manager practices and awareness, and (3) data gaps and inconsistencies. The oil impact region was highlighted because it has recently become a relatively high-risk area for local road crashes.

Methodology

A mail survey, descriptive statistics, and visualization methods were used in the research. The questionnaire was mailed to 53 North Dakota county road managers, with 33 (62%) of the managers responding. Crashes on the local rural road network, with the exception of those within towns, were analyzed in the state's early work with county and tribal nations' LRSPs (*6*). The work here was to explore progress related to that immense local road safety effort. A first step in this process was to revisit the data used to compile county LRSPs.

Robust crash data were essential information in this work. Crash records include details such as location, contributing factors, and environmental conditions that were used to identify problems and safety project investment priorities. That analysis was conducted within a limited use agreement that was determined exempt by North Dakota State University's institutional review board.

Findings

The local road severe crash trend analysis over the past 15 years showed a peak in severe crash events in 2012, following a steep incline after 2009. Severe crashes were defined as events with fatal and disabling injury outcomes. Crashes on local rural and other rural roads in the state generally trended downward after 2012. The severe crash trend on other roads fell below the 2005 benchmark count in 2019, while the local road crash trend remained slightly above that baseline after a slight spike in 2017 (Figure 1). While traffic characteristics impact crash trends, LRSP safety improvements in HSIP investments, along with proactive low-cost roadway safety measures, public education, and high-visibility enforcement, are also likely influences in the downward trend.
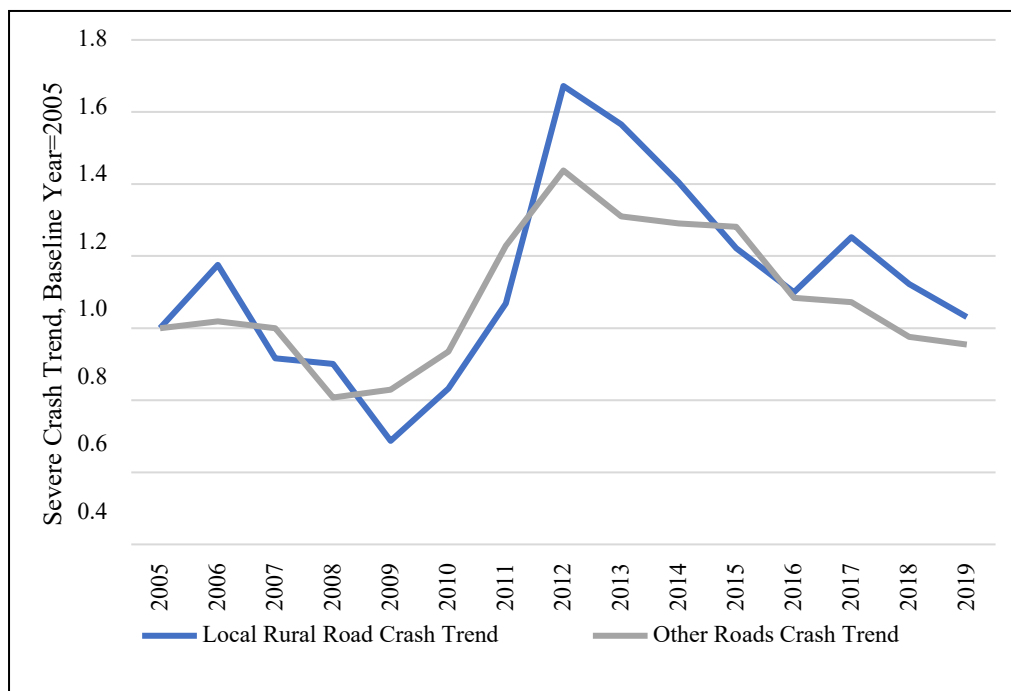
FIGURE 1 North Dakota crash incidence, fatal and disabling crashes by road group.

Local road managers' safety awareness has grown compared to a survey of this same position group about a decade ago. Nearly half the counties had applied for HSIP projects since the LRSPs were completed, based on the 2020 survey responses. Only 25% of counties reported that they had applied for road safety funds in 2010. Comparison to a previous survey shows a notable expansion in counties adopting low-cost safety strategies. For example, 50% of the 2020 responses indicate a high likelihood for chevron use on curves compared to just 30% in 2010.

Counties' likely use of curve delineators, rumble strips, and guardrail safety endcap strategies also increased substantially.

In the oil region, lane width and shoulder width of local roads play an important role in improving road safety. While lane width and shoulder recovery surface are important, local road budgets often prohibit reconstruction with these improvements. Enhanced edge lines have been an effective safety countermeasure when lane width is less than 12 ft. The importance of delineators was highlighted in the studied regions where 50% of the oil region counties reported their application. However, it would be helpful for the counties to consider the effect of delineators when investigating the frequency and severity of crashes. More specifically, monitoring and maintaining bridge delineator signs and bridge load posting signs are of high importance.

Gravel road dust was reported as a prominent safety issue. Dust can limit visibility and lead to raveling and washboarding of roads. In addition, dust is a health issue for people, animals, and crops living or growing next to dusty roads. One option may be to identify hazardous gravel road intersections in LRSPs and apply dust control products as spot treatments using HSIP funds. The challenge remains that a product may last a few months to a year depending on weather and traffic. In addition, gravel surfacing specifications are crucial for counties, tribes, and townships. Encouraging counties to focus on the plasticity index, a measure of how sticky the fines or dust particles are when wet, may be beneficial for improved surfacing with gravel specifications. A proper choice of the index for gravel can prevent washboards and unraveling issues on gravel roads.

Road signage is also vital, providing drivers with advisement needed for navigation and vehicle control (7). A significant dispersion was observed in the number of signs on local roads across North Dakota counties. Twenty-four of 32 respondents reported their sign count.

According to the survey results, the average number of signs reported by a county from the oil impact region was 2,806, while the average from all surveyed North Dakota counties was 2,754, which shows no considerable difference in the use of signs in the regions. Figure 2 shows the variation in road sign density, in sign counts per 1,000
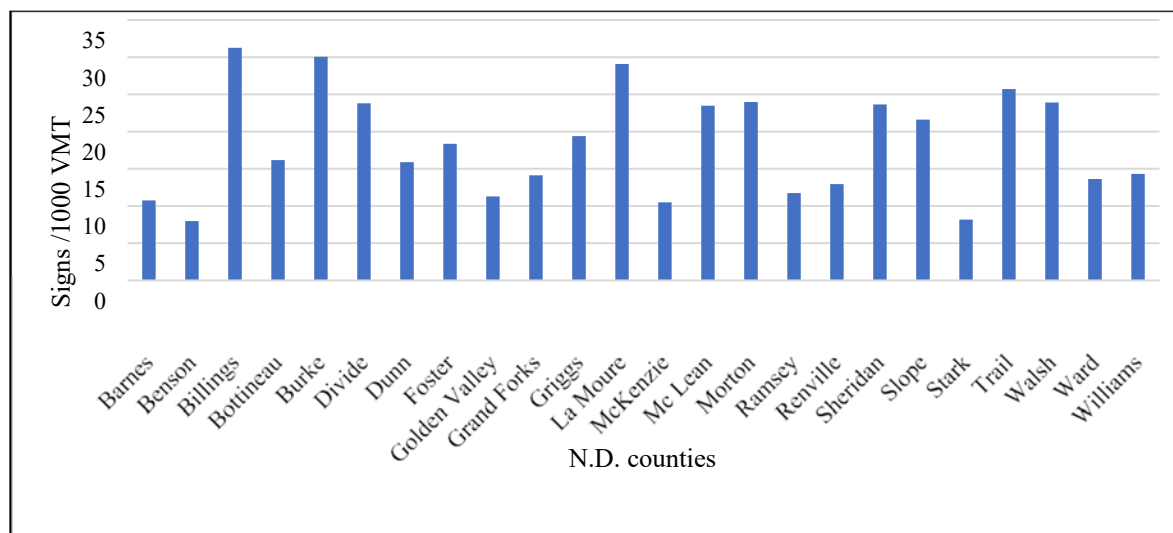
FIGURE 2 Road sign density, by county.

vehicle miles traveled, across the respondent counties. Road sign use can be a function of topology, road geometry, traffic density, life-cycle cost, and engineers' discretion.

Conclusion

A more widespread awareness of safety issues and local road crash prevention strategies was evident in the knowledge and practices of local road managers in the state compared to a similar survey in 2010. The data-driven crash knowledge foundation was established with the state-supported LRSP plans developed for each county in the state beginning in 2012. While data gaps and inconsistencies were identified, these plans presented a new opportunity for county road managers to pursuesystematic and site-specific safety investments based on data-driven priorities and strategies. Benefits from this investment were anticipated in increased local knowledge regarding traffic safety strategies and increased safety investments in local road management plans. Not surprisingly, subsequent action in terms of investments and proactive traffic safety decisions has been mixed. Since the LRSPs were created more than 5 years ago, anecdotal evidence in conversations with experts and reviews of road managers have been the basis for understanding progress in incorporating safety into local road planning and knowledge.

While survey findings were limited by the voluntary responses from a single state's

county road managers, important information was gathered about common practices and leading peer traffic safety strategies. Valuable insight regarding the need for state leadership and support with traffic safety initiatives, especially with counties that may have severe resource constraints and/or inexperience with regard to safety issues and improvement strategies was gained. In addition, the value of baseline and period survey with this group provides a means to continue to assess progress quantitatively to support anecdotal qualitative insights. Sharing this work on ongoing improvement in the LRSP with other rural states may be especially helpful in identifying and sharing best practices to support local road safety efforts. Future work is planned to for safety decision tools to enhance and promote best practices and ongoing emphasis in local road department planning.

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ASSET MANAGEMENT

Predicting Iowa Concrete Overlay Performance and Remaining Service Life Using Statistics and Deep Learning Techniques

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Since 1901, portland cement concrete (PCC) has been used to overlay existing pavements, and before 2017 a total of 1,289 concrete overlay sections had been built in at least 46 states in the United States, 32% of which are located in Iowa (*1*, *2*). Considering that PCC overlays are heavily used in Iowa counties, these roads play a critical role for county engineers. Such complex county pavement systems with multilayers resulting from pavement construction and renewal give Iowa county engineers difficulty in estimating the current and future pavement performance and remaining service life, and this challenge creates a need to establish reliable and accessible methods or tools for use by Iowa county engineers in their routine pavement analysis/design/asset management practices, the objective of this study.

This study, a part of IHRB TR-740 Project (*3*), focusing on developing a pavement performance analysis tool for Iowa county roads, evaluates statistical and deep learning techniques using an artificial neural network (ANN) for predicting pavement performance expressed in terms of the international roughness index (IRI) and resulting in forecasting pavement remaining service lives (RSL).

Utilizing statistics- and ANN-based approaches, pavement performance models were developed using a historical and condition database of PCC overlays in Iowa. RSL values for the pavement sections were calculated using threshold limits as the

performance indicator once the pavement performance model had been developed. IRI was used as a critical performance indicator of pavement for RSL calculations, with RSL determined as the time between the current pavement age and the age at which future performance prediction has reached its threshold limit. A macro-enabled automation tool Iowa Pavement Analysis Techniques (IPAT) based on Microsoft Excel and Visual Basic for Applications (VBA) was also developed to analyze project-level pavement performance, to make future pavement performance predictions, and to estimate RSL developments for any given road section. This tool can be incorporated into pavement management processes and help engineers make better infrastructure planning decisions using real pavement performance data to create realistic future condition predictions.

Methodology

Preprocessing Data

A historical database was provided by the Iowa Concrete Paving Association, and a condition database was provided by the Iowa Pavement Management Program. Traffic data were obtained from the Iowa DOT Roadway Asset Management System/open data online. A unique project identifier (Road ID) was attributed to every single project from the different spatially integrated databases by a geographic coordinate system using latitude and longitude data and a linear referencing system using the beginning and ending mileposts of each project location.

Where condition data (i.e., IRI) were missing in a road section, the interpolation substitution method was used to replace the missing values. The interpolation was performed using the last existing condition data prior to the missing one and the first existing data after the missing one.

Developing Statistics-Based Models

A statistically defined sigmoid pavement deterioration curve-based approach was utilized for IRI calculations for county PCC overlaid pavement sections in Iowa. The nonlinear sigmoidal model can be computed using Equation 1. The model parameters, e.g., C1, C2, C3, and C4 for IRI, representing contributions of different input parameters, are solved through an iterative process that begins with good initial estimates of these parameters. During convergence, new estimates are obtained by

minimizing the error sum of squares, i.e., the squares of differences between the target and predicted IRI values.

$$IRI = C_1 + \frac{C_2}{1 + e^{(C_3 + C_4 \times age)}}$$

(1)

Developing ANN-Based Models

An artificial intelligence–based pavement performance model was developed for evaluating county PCC overlay pavement performance by IRI prediction. The developed database was utilized for model development and independent testing of developed models. About 87% of composite pavement data points in the county database were used in model development, and 13% of them, corresponding to 20 road sections (194 data points), were used for independent testing of the developed model. In detail, the study used 148 PCC overlay pavement sections with 1,478 data points in model development and independent testing. It used 900, 128, and 256 data points as training, testing, and validation data sets, respectively, corresponding to approximately 70%, 10%, and 20% of the model development database (1,284 data points).

The input parameters, i.e., overlay thickness (in.), traffic [accumulated annual average daily traffic (AADT)], pavement age (year), joint spacing (ft), and previous consecutive 2 years of IRI measurements [IRI(i-2) year and IRI(i-1) year] (in./mi) are used to develop the ANN model, and the output parameter was the current year IRI (IRI (i) year) (in./mi).

The success of the pavement performance prediction models in mimicking measured pavement performance indicators was quantified using coefficient of determination (R^2), absolute average error (AAE), and standard error of the estimates (SEE). Higher R^2 and lower AAE and SEE values are indications of accurate model prediction.

Figure 1 compares IRI values measured in the field to those predicted by the ANN-based model. The model produced high accuracy in model development, with high R^2 and low AAE and SEE values obtained for all training, validation, testing, and independent testing data sets. Table 1 presents the minimum and maximum data ranges of data sets used to train and independently test the ANN-based IRI model. Since the range of the independent testing data set lies within the range of data used to develop ANN model, the independent testing accuracy shown in Figure 1 was high, meaning that the predicted IRI values were almost overlapped by the measured IRI values.
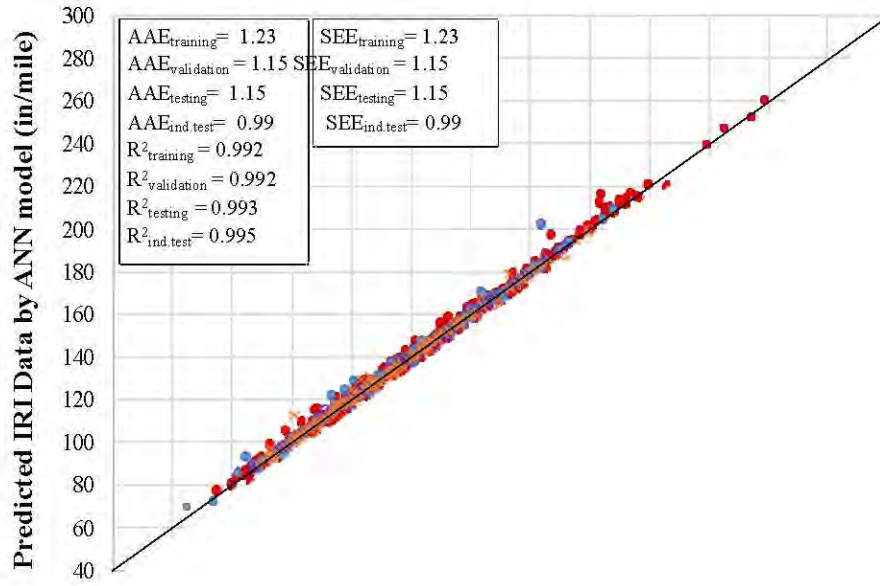
FIGURE 1 Measured versus predictions by ANN-based IRI model.

**TABLE 1 Input Ranges Used in Developing and Testing
ANN Models for PCC Overlay Pavement Sections**

ANN-Based IRI Model	Input Ranges Used for Developing ANN Model		Input Ranges Used for Independently Testing ANN Model	
	Min.	**Max.**	**Min.**	**Max.**
Overlay thickness (in.)	2	10	5	8
Traffic (accumulated AADT)	120	90,600	240	38,750
Pavement age (yr.)	4	52	4	38
Joint spacing (ft)	0	40	6	20
IRI (i-2) year (in./mi)	60.5	249.7	82.4	190.9
IRI (i-1) year (in./mi)	62.8	254.5	87.5	195.3

Developing Automation Tool

As stated earlier, as part of this study, a macro-enabled Microsoft Excel and VBA-based IPAT automation tool was developed for analysis of project-level pavement performance, for prediction of future pavement performance, and for estimation of RSL developments for any given road section. The tool is capable of processing PCC overlay sections, predicting their future pavement performance predictions, and based

on these predictions and user-input threshold values, estimating their RSLs. The statistics- and ANN-based models were integrated into the IPAT tool for PCC overlays, while the tool also provides a series of options for four pavement types representing Iowa county pavement systems. The tool is user-friendly and available for downloading free of charge. Screenshots and the latest version of the tool can be found in another study (*3*).

This tool can be incorporated into pavement management processes and help engineers make better infrastructure-planning decisions using real pavement performance data to create realistic future-condition predictions.

Consequence Analysis

As part of this study, a consequence analysis was conducted to evaluate the developed ANN-based models based on two different what-if scenarios to find out how pavement performance and RSL predictions change using (1) different traffic levels and (2) different overlay thicknesses for a PCC section. For this purpose, several PCC overlay sections were tested for various traffic levels (50% reduced, 25% reduced, actual, 25% increased, and 50% increased) and PCC-overlay thicknesses (2-in. thinner, 1-in. thinner, actual, 1-in. thicker, and 2-in. thicker), and IRI predictions were obtained from developed ANN-based models. It was observed that as traffic levels increase and PCC-overlay thicknesses decrease, IRI predicted values increase. It was also observed that the impacts of traffic and thickness on the IRI are not the same for all road sections, with the magnitude of changes in IRI predictions after changing traffic and thickness depending on age, traffic level, structural properties, and condition of each individual road section.

Findings

For the sake of validating the developed statistical- and ANN-based models, individual road sections were tested to predict current IRI values that were then compared with the measured IRI and future IRI values for estimating their RSLs, as shown in Figure 2. The developed nonlinear sigmoidal models corresponding to each road section can be written as follows:
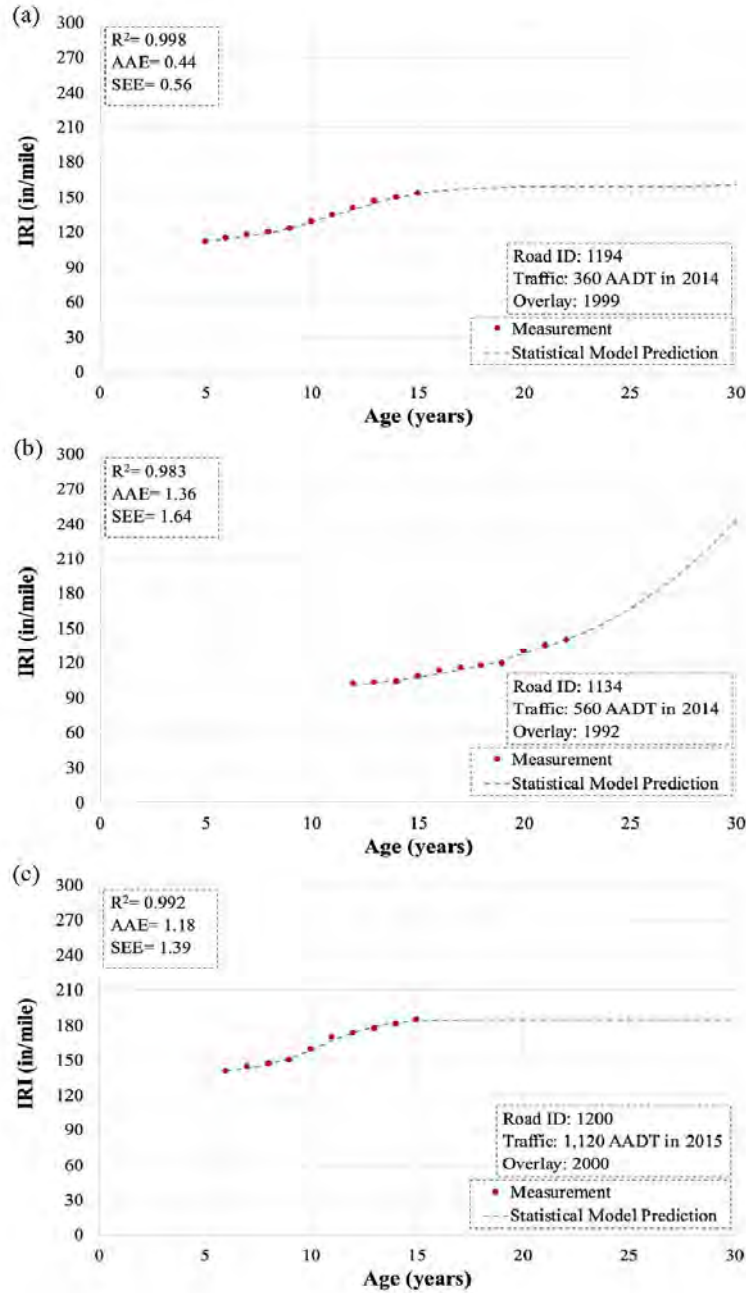
FIGURE 2 Measured IRI versus Predicted IRI (*a*) Road ID 1194, (*b*) Road ID 1134, (*c*) Road ID 1200 by Statistical Model and ANN Model, respectively (*continued on next page*).
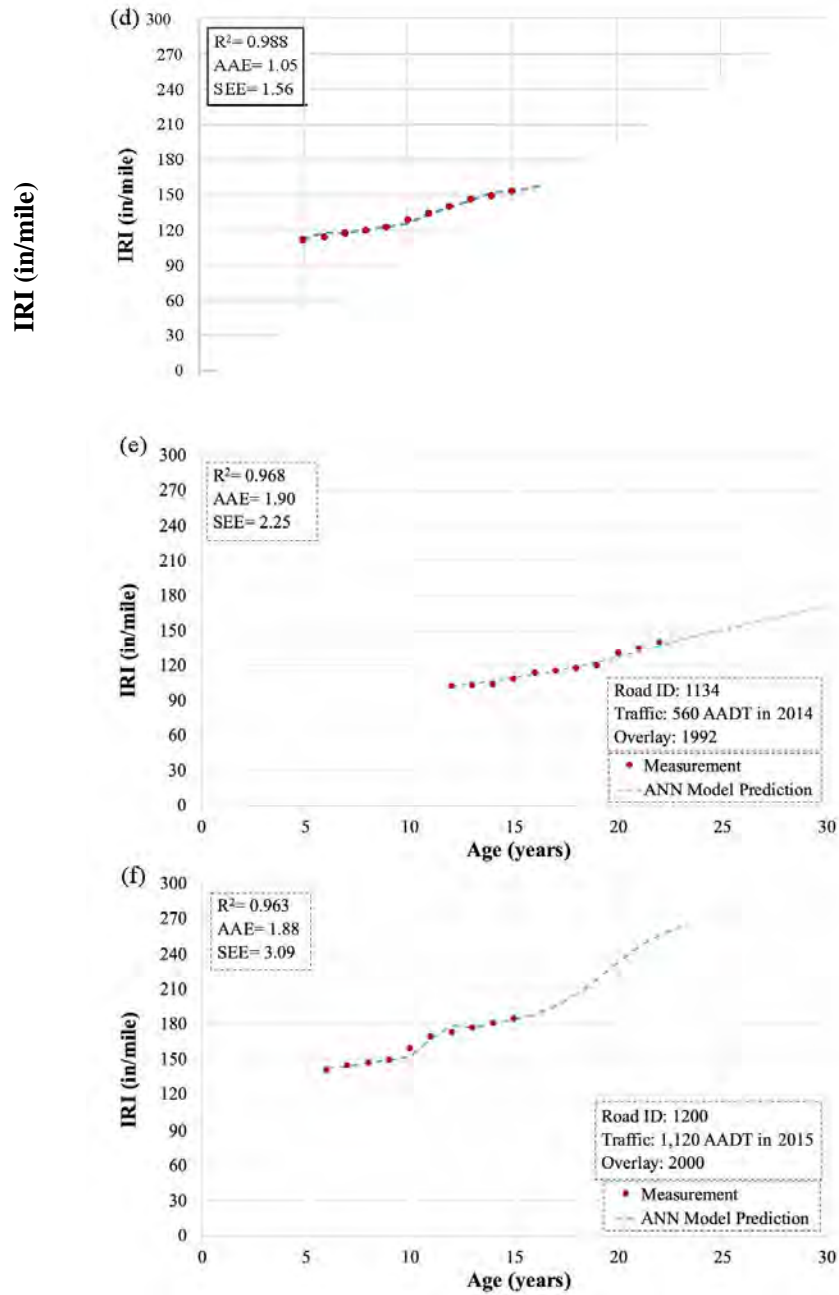
FIGURE 2 (*continued*) Measured IRI versus Predicted IRI (*d*) Road ID 1194, (*e*) Road ID 1134, and (*f*) Road ID 1200 by Statistical Model and ANN Model, respectively.

Road ID 1194: $IRI = 110.07 + \dfrac{49.64}{1+e^{(5.31-0.48 \times age)}}$

Road ID 1134: $IRI = 87.98 + \dfrac{1734.59}{1+e^{(6.62-0.14 \times age)}}$

Road ID 1120: $IRI = 139.26 + \dfrac{45.45}{1+e^{(7.36-0.71 \times age)}}$

After future pavement performance of county roads had been predicted, the RSLs of these roads could be calculated considering the threshold limit of pavement performance indicators,170 in./mi, recommended by the FHWA (*5*). The RSL of each county road section can be calculated by subtracting the current year from the year in which the threshold was exceeded. Under these conditions, the average RSL of 18 county PCC overlay sections, as an independent test data set, was found to be about 15.3 years by the statistics-based model and about 7.4 years by the ANN-based model.

Conclusions

Accurate statistical- and ANN-based IRI performance models were developed for PCC overlays. Development of the ANN-based models using the Iowa county database reflected the importance of data availability and data limitations used in models. The calculated average RSL results are based on only a limited number of pavement sections, and since the pavement sections analyzed also had low IRI values throughout the years of data collection, IRI could not reach the threshold limit within the pavement's design life, so the RSL was calculated based on the design life, resulting in higher RSL values when using a statistics-based approach. However, it should be noted that the availability of more measured data for models could provide better patterns for predicting future data.

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Data-Driven Approach to Identify Maintained Pavement Segments and Estimate Maintenance Type for Local Roads

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Missing maintenance records is one of the challenges that face the pavement management process for local transportation agencies. Some studies investigated possible solutions to overcome this issue. Part of them suggested using the rate of deterioration concept to generate performance models when the age of pavements is unknown. However, this approach did not give the ability to detect maintenance activities on the road network. Other studies suggested using either probabilistic or deep learning techniques for maintenance detection. In this study, a data-driven approach was proposed and utilized to detect probable maintenance activities on the network. Pavement condition data for municipal roads in Iowa was used to generate performance models for flexible, rigid, and composite pavements, which were used to predict the condition for the year 2021. Predictions were compared to actual data, and the difference between the actual and predicted values was calculated for each road segment. The data clusters were obtained using the mini-batch k-means clustering algorithm. This process was done separately for flexible, rigid, and composite pavement at low-volume and high-volume traffic levels. It was found that the resulting clusters could be used to roughly determine the maintenance type. On the other hand, clusters might be better used to indicate the probability of a segment being maintained based on the value of observed condition improvement for that segment.

Introduction

Successful pavement management practices require pavement condition data, performance models, and suitable optimization algorithms (*1-2*). However, some challenges might impact the pavement management process, such as the lack of data, data quality issues, uncertainty associated with the process of pavement deterioration, and missing maintenance records (*1-4*).

Nowadays, pavement condition data is more affordable and easier to acquire for transportation agencies since the data collection process is now fully automated after employing various advanced technological methods for this purpose (*5*). Quality assurance and quality control procedures are applied to ensure that pavement condition data is as accurate and complete as possible (*6*). This is essential for most transportation agencies that have their data collected by contractors.

Still, not all types of data can be collected during pavement condition data collection. For instance, maintenance information is not a part of the collected data during condition data acquisition. Maintenance data can be obtained from maintenance records which are not always available, especially for local roads (*4*). This issue might impede the performance modeling in case age was one of the dependent variables in the employed performance models.

One approach to overcome modeling issues due to the missing maintenance records was proposed by Kargah-Ostadi et al. (*4*), which utilized the concept of rate of deterioration (ROD) for generating deterioration curves of local roads. This approach only requires having the initial condition in terms of one of the performance indices and the ROD, which can be calculated from the historical condition data by finding the difference in condition between 2 consecutive years for a certain road section and then normalizing the difference by the time gap between the 2 years of data collection to obtain the ROD for a single section. Afterward, RODs for the entire network must be grouped based on the initial condition into predefined groups. Later, the average ROD for each group is calculated and then used to obtain the deterioration curves. The mentioned approach is beneficial when condition data for several consecutive years is available. Thus, calculating the mean ROD can be done, and the estimated means would be more accurate, yielding more reliable predictions.

Nevertheless, this approach is useful for performance modeling when age or maintenance records are missing, but it cannot be used to predict or detect pavement maintenance. For example, if a section was maintained between the last year of data collection and the year of condition forecasting. In that case, the approach proposed by Kargah-Ostadi et al. (*4*) will face the same issue as ordinary performance models; the

prediction will not be accurate because of the instantaneous improvement due to the maintenance action.

To overcome the missing maintenance records issue, there was a need for a method to detect pavement maintenance actions. Therefore, many studies have proposed different methods to achieve this goal. In particular, Gao et al. (*7*) used three different deep learning (DL) models to detect segments with probable maintenance actions by employing 21 surface condition indicators, including the International Roughness Index (IRI), rutting with different severity levels, and various types of crackings. The accuracy of these models was quantified for the three models using a testing dataset, and the best model accuracy was 87.5%. Another approach that accounts for condition restoration caused by maintenance was proposed by Gao et al. (*8*). This approach assigns probabilities for pavement segments that reflect the likelihood of a maintenance action took place by comparing the previous condition (prior distribution) to the current condition (posterior distribution) based on the Bayesian approach in a way which gives sections that are having larger differences between prior and posterior conditions a higher probability of being maintained.

The maintenance and rehabilitation (M&R) actions may also be identified and confirmed by referring to the video logs of pavements that were captured during the data collection phase. Which sometimes would be necessary to support the conclusions drawn from the numerical analysis.

Some studies discussed the effectiveness of different treatments on the pavement's overall performance and how to account for it in the performance models. These studies are important for estimating the value of the condition improvement for different types of treatments and establishing a suitable inclusion way of these values in different performance models (*9–12*). This helps in moving from subjective decision trees to data-driven decision trees of maintenance selection and time of implementation supported by what was learned from the historical data (*13*).

Machine learning (ML) techniques have been widely used for pavement performance modeling especially supervised learning methods, including regression and classification algorithms (*14*). However, unsupervised learning was less commonly used since it gives an understanding of the data's patterns, structure, or clustering but not the relationship between a dependent variable and some other independent variables.

To sum up, missing maintenance records is causing inconveniences in pavement performance modeling and condition forecasting. Some approaches proposed methods to allow condition forecasting or to obtain performance curves when maintenance records are missing. In contrast, other approaches were focused on detecting the

maintenance action and properly incorporating that into the performance models. In this research, a set of previously mentioned methods will be combined into one approach to identifying segments with maintenance actions and the type of applied maintenance strategy (i.e., preventive, minor rehabilitation, major rehabilitation, or reconstruction).

Objectives and Methods

This study aims to set the framework for detecting pavement sections with probable maintenance action. Furthermore, it aims at determining the maintenance type. For this research, the approach was applied separately using condition data for municipal roads with low and high traffic volumes.

Data Description

The data for this analysis was obtained from the Iowa Pavement Management Program's (IPMP) pavement condition database, which is managed by the Center for Transportation Research and Education at Iowa State University on behalf of the Iowa DOT. The data was collected by a semi-automated data collection technique using the Automated Road Analyzer (ARAN) vehicle, which gathers and maintains the pavement surface condition data as video recordings. Pattern recognition software is then used to manually detect, examine, and classify the pictures obtained from the video recordings. The raw data is delivered in 16-m segmentation, resulting in approximately 1.8 million segments per collection cycle, covering approximately 17,860 mi of roadway. The collected data includes roughness, rutting, different types of cracking, and faulting. Since 2013, the Iowa DOT has collected statewide data on all local roads in the state using a data collection vendor. Each road segment is collected every other year, resulting in a 2-year analysis period for this study. Iowa DOT has utilized the same vendor for the data collection from 2013 to 2015, and another vendor collected the data from 2016 to 2021. However, the standards for data collection are rigorous and documented in the agreement and the Data Quality Management Plan as required by the FHWA.

The pavement condition data used in this study to estimate the ROD is for the years 2013–2019 and only contains municipal roads' condition information with no information about maintenance activities. The data included raw condition data aggregated into pavement management sections and the Pavement Condition Index (PCI) calculated for each section. Condition data for the year 2021 was also used to compare predictions

obtained from the performance model with the actual condition, highlighting the segments with improved performance. Hence, the approach was applied to the condition data for the municipal road networks in the southern half of Iowa, which data is collected in odd years (i.e., 2019 and 2021). In the geodatabase, each pavement management section record contains information about the functional class of that section, the pavement type, and the jurisdiction in which the pavement section is located. Table 1 shows the count of pavement sections initially used in this study by pavement type and traffic level (high, low). The total number of pavement segments is 65,752 segments, which is the number of sections in the year 2019.

Methodology

Figure 1 shows the procedure applied in this study. The first step is to obtain the condition data for several years, and in this study, the data for the years 2013 to 2019 was used to obtain the performance model using the ROD approach, as discussed previously. The concept of ROD was used because the age data was not available. A similar approach to the one proposed by Kargah-Ostadi et al. (*4*) was adopted, except that more factors were considered besides the initial PCI, including road functional classification, geographical region, and pavement type. RODs were stored in tables instead of using them to generate deterioration curves, and a tool in the ArcGIS Pro environment was developed to automate the prediction process and to store the predictions in the network shapefile as attribute data since hundreds of data groups were formed to accurately calculate the mean ROD for each group and accurately predict the future PCI when age data is absent. The reason behind using predictions instead of initial conditions (initial PCI) was to compare in the same year instead of comparing the condition in two different years. One thing to consider is to have the time gap between the 2 years of obtained data as small as possible to estimate the time of treatment application more accurately.

TABLE 1 Number of Initially Selected Sections to Run the Study

	Flexible	Composite	Rigid	Total
High Traffic Volume	3,758	4,161	7,286	15,205
Low Traffic Volume	16,595	6,829	27,123	50,547
Total	20,353	10,990	34,409	65,752
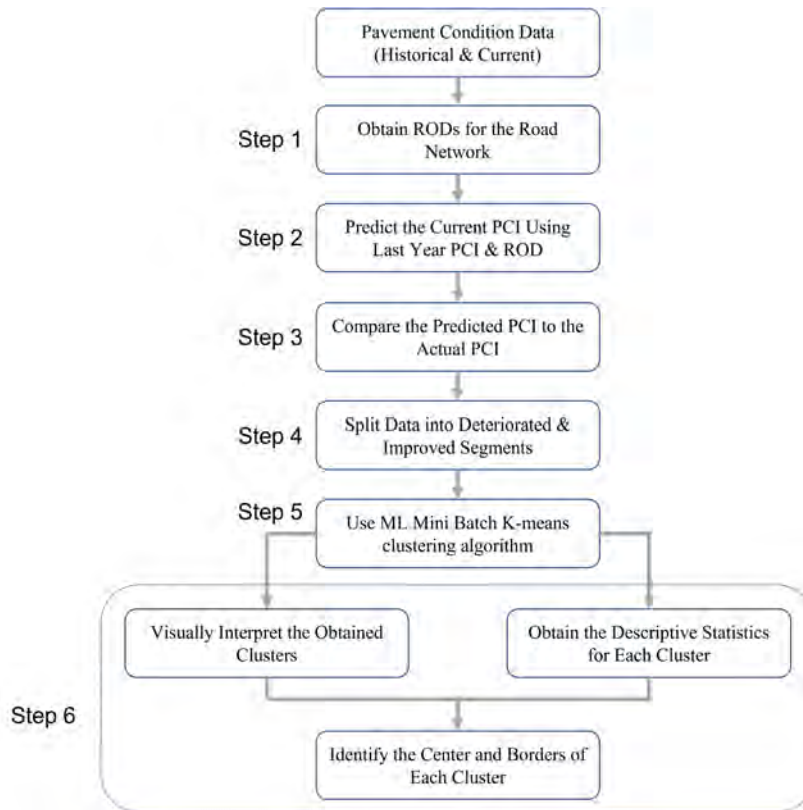
FIGURE 1 Proposed methodology for maintenance detection.

Data for the year 2019 was used as the initial condition, which was used to forecast the condition in the year 2021, and data for the year 2021 was utilized as the actual condition or the current condition. After predictions were calculated, they were compared to the actual PCI (i.e., actual PCI in 2021 vs. predected PCI in 2021). The difference between the actual and predicted PCI was measured and then used with the actual PCI value as inputs for the ML clustering algorithm.

Clustering is an unsupervised form of machine learning. Clustering techniques are useful when data labels are missing because the labeling process is time-consuming and requires a well-trained person to be performed (*15*). Therefore, it was the best choice out of the ML techniques to achieve this study's goal since data labels (i.e., maintenance records) were missing. Before running the clustering algorithm, the data was split into six datasets based on the type of pavement (i.e., flexible pavements, rigid pavement, or composite pavements) and the volume level (i.e., high or low). The roads were classified into high- or low-volume ones based on their functional classification. Hence, residential or local roads were assumed to be low-volume roads, while collectors and arterials were considered high-volume roads).

Different clustering algorithms were investigated and applied to the datasets, and mini batch K-means (MBK) algorithm (*16*) was selected due to the procedure it follows in data grouping and because it works well with large datasets, which was the case in this study. MBK, which is an improved form of the K-means clustering algorithm, is working similarly to k-means but with an improved performance in terms of running time and accuracy (*15*). Like K-means, MBK divides the data into a predefined number of groups (k). It starts by randomly initiating centroids for the clusters and then assigning each point to its closest centroid. The centroid is iteratively updated every time the cluster is updated by adding a new point. The algorithm will maintain the output clusters to achieve the minimum squared residuals within each cluster. The major difference between the two algorithms is that MBK uses mini batches of the data instead of including the entire data simultaneously, which remarkably reduces the running time (*15, 17*).

The specified number of clusters varied for the six datasets, but it was never less than seven clusters. The number of clusters was selected to reflect the number and the type of treatments that would be applied throughout the pavement's lifespan, considering three to five preventive or routine maintenance procedures, two or three minor rehabilitations, one major rehabilitation, and a reconstruction.

Afterward, the output of the MBK algorithm can be visually interpreted, and the cluster numbers can be stored in the data file to draw more useful information about each cluster, such as the descriptive statistics to numerically address the characteristics of each cluster. Finally, the centroid and borders of each cluster can be useful information to obtain, which could help calculate the expected improvement associated with types of maintenance.

Results and Discussion

Predicted PCI values were obtained and validated by comparing the actual and the predicted PCI values, then accuracy was reported as the percent of predictions having an error of less than or equal to 5 PCI points. The average accuracy was found to be around 70%. However, the accuracy of predictions was variable based on the pavement type, which was 78%, 68%, and 65% for flexible, rigid, and composite pavements, respectively. Segments that showed an improving condition (i.e., actual PCI greater than predicted PCI) were kept and used in the following steps except for segments with improvement values between 1 and 5 PCI points which were excluded because segments in this region are highly likely to have prediction error since the acceptable

prediction error for the ROD method as indicated by Kargah-Ostadi (*4*) was equal to 5 PCI points. At the same time, sections that showed an indication of deterioration were excluded and assumed to be not maintained segments in the period starting from the year of the initial condition (2019) until the last year of data collection (2021). Table 2 shows the number of sections used as input for the clustering algorithm after removing segments that show condition deterioration and minor improvement.

An important thing to be aware of when applying this approach is to keep the analysis period as short as possible to estimate the year of applying the treatment reasonably. If the analysis period is long, then the time and maintenance effectiveness will not be determined appropriately. Thus, an analysis period of one or maybe two years would yield the optimum results since it will be easy to determine the time of treatment implementation and the treatment effectiveness before the segment deterioration negates the treatment's effect over time. Figure 2 is a scatterplot showing the actual PCI against the difference between the actual and predicted PCI for pavement sections that showed improved performance for all pavement types and traffic levels.

TABLE 2 Number of Sections Used as Input for the Clustering Algorithm

	ACC	COM	PCC	Total
High	1,305	1,700	2,271	5,276
Low	8,208	3,009	10,713	21,930
Total	9,513	4,709	12,984	27,206
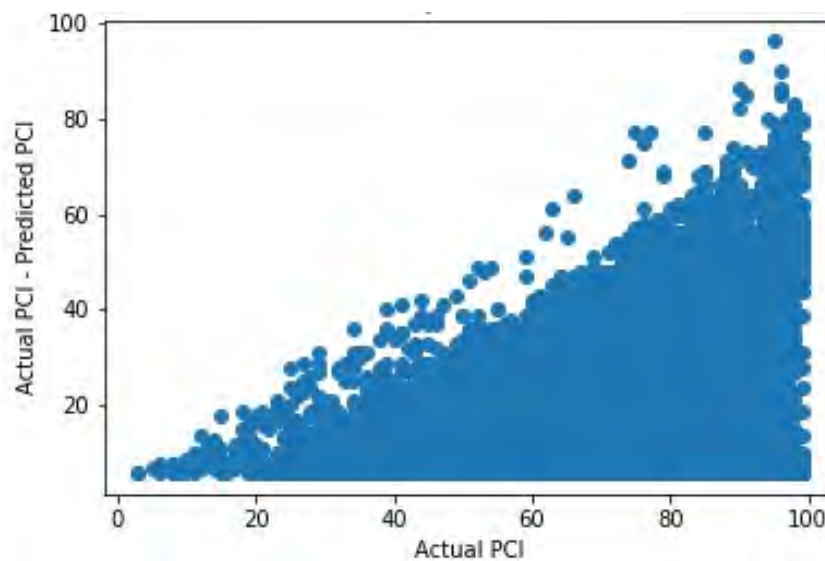
FIGURE 2 Actual PCI against improvement in the condition.

Next, the dataset of sections that showed a condition improvement was split into six subsets based on the pavement type (i.e., flexible, rigid, and composite) and the traffic volume level (i.e., high and low). Then the resulting subsets were used as input for the MBK with actual PCI and condition improvement as x and y, respectively.

After testing various clustering algorithms, it was found that MBK achieved the best results in terms of the obtained clusters' shape (i.e., rectangular or trapezoidal shapes) and vertical and horizontal dimensions of the clusters. The shape and dimensions of the clusters are approximately comparable to the theoretical values associated with each type of maintenance, as seen in Figure 3, which shows the MBK algorithm output clusters for the six subsets. Each subset represents sections of one pavement type at either low or high traffic volume levels. For all of the six subsets, it can be noticed that the improvement is divided into four horizontal bands consisting of either one or more clusters. Each one of the four bands may represent one type of maintenance. For instance, the lower band with low improvement values might indicate a preventive maintenance region. Yet, it might also indicate a region of measurement or prediction errors. Still, the probability of maintenance existence is the lowest for this band compared to other ones. The band above the lower band contains sections with a higher probability of being maintained, and it could be considered a minor rehabilitation region. The third band with condition improvement ranges from 30 to 60 for the different subsets designates the region of major rehabilitation. The last band, with the highest improvement values ranging from 50 to 100, would be considered the reconstruction region. Generally, it can be noted that a significant number of points fall in the lower band, while fewer points are in the upper bands, which is expected since there are fewer reconstruction and major rehabilitation projects compared to the preservation and minor rehabilitation projects per analysis period from a cost perspective. Additionally, sections in the lower band are more likely to experience small increases due to preservation treatments which can make it difficult to differentiate from prediction errors than those in other bands.

The lowest two bands always consist of two or more clusters for all subsets of data. Each cluster groups the segments that have a certain type of maintenance at a certain stage of life. For example, in Figure 3a, the pink cluster in the second lowest band groups segments had a minor rehabilitation in an earlier stage of the sections' life span compared to the sections in the grey cluster in the same band. These conclusions could be generalized for the six data subsets. The differences between subsets' clusters could be in the number of clusters, clusters' dimensions, and clusters' centroids. One
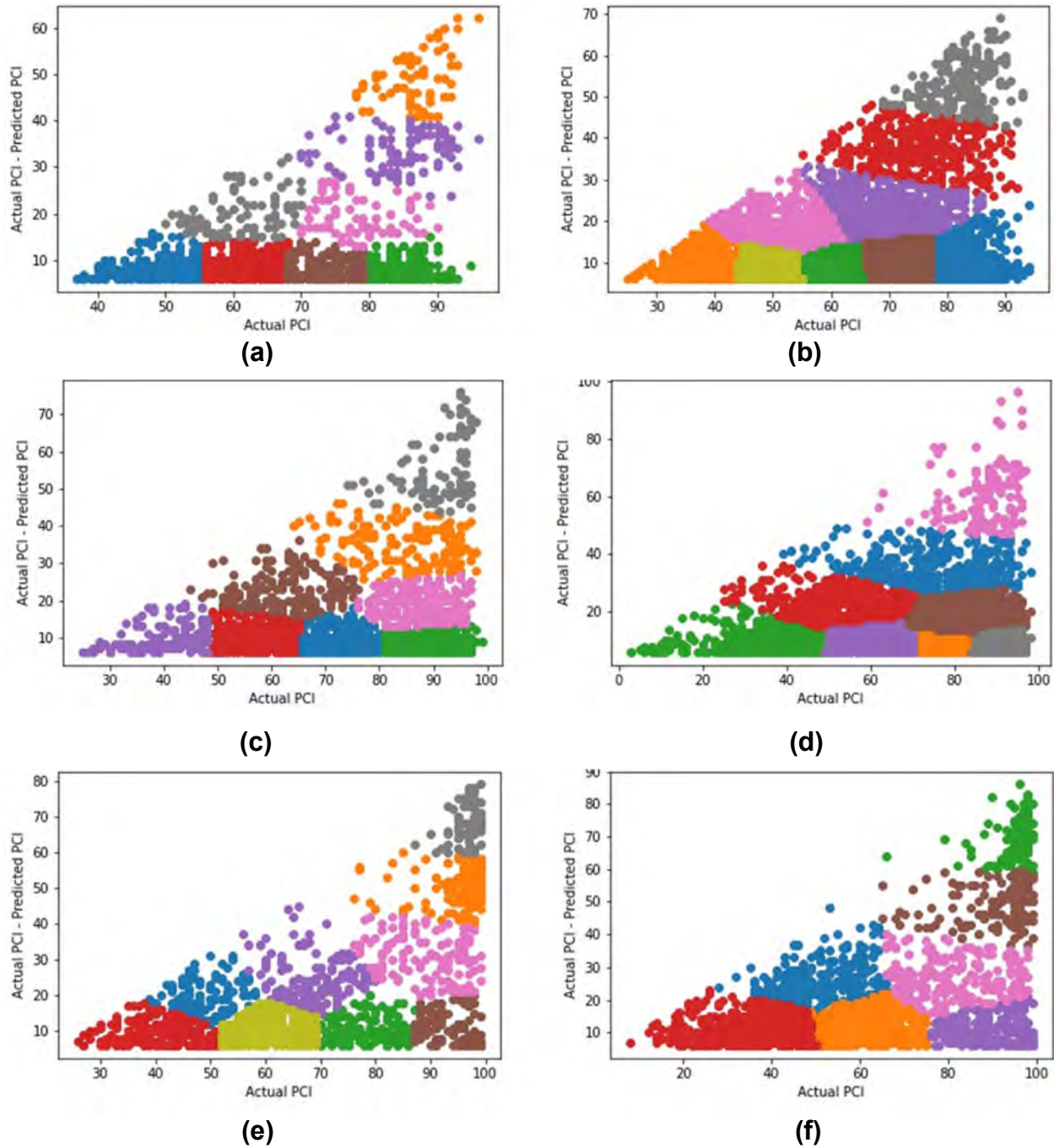
FIGURE 3 MBK clustering results: (*a*) ACC high-volume roads; (*b*) ACC low-volume roads; (*c*) PCC high-volume roads; (*d*) PCC low-volume roads; (*e*) COM high-volume roads; and (*f*) COM low-volume roads.

disadvantage of clustering or unsupervised learning, in general, is that there is no way to assess the accuracy of the results unless data labels are available. But in that case, using supervised learning will be a better choice to be considered rather than unsupervised learning. Still, there are some solutions to getting better results. One solution could be to apply this approach at the agency level instead of the state level because the composition of the road network might vary between agencies in terms of the construction materials used. For example, some agencies' road networks might be mainly composed of rigid pavements, while others might have a network composed of flexible or mixed types of pavements. Therefore, the treatments used by the agencies might differ, and thus the improvement in condition might vary accordingly. For that reason, applying this approach at the agency level could guarantee some level of homogeneity in the condition improvement associated with the same type of maintenance strategies. A second strategy to improve clustering results would be to specify the initial centroids using approximate condition improvement associated with each maintenance type and the implementation trigger (i.e., at what condition it would be applied) as the y and x coordinates, respectively. To better understand the outputs, the descriptive statistics for each cluster in the six data subsets were tabulated in Table 1, Table 2, and Table 3.

As shown in Table 3, for flexible pavements, the mean improvement for most of the clusters for low-volume road segments was slightly higher than it for high-volume road segments. Also, the maximum value for improvement for low-volume flexible road segments was 69 PCI points increment. In contrast, the maximum improvement for high-volume roads was 62 PCI points increment.

Maximum and minimum values of improvement and actual PCI are beneficial for calculating the linear boundaries between two adjacent clusters when mean improvement could estimate the effectiveness of a certain type of maintenance at a certain stage of pavement life. The range of improvement values for clusters indicates major rehabilitation or reconstruction is wider than that for the other two clusters indicating preventive maintenance or minor rehabilitation, which, again, makes sense since the condition restoration associated with major rehabilitation or reconstruction is much larger than that for minor rehabilitation and preventive maintenance. Clusters with a higher mean improvement have higher standard deviation because these clusters are more spread than clusters having a lower mean improvement value. Minimum actual PCI and maximum actual PCI represent the cluster range, which can also be used to estimate the condition range in which a certain type of maintenance could be implemented.

TABLE 3 Clusters' Descriptive Statistics for Flexible Pavements

Traffic Volume	Mean ActualPCI	Mean Improvement	Min. Improvement	Max. Improvement	Improvement Std	Min. ActualPCI	Max. ActualPCI
High	48.960	8.385	6	16	2.396	37	55
	61.775	8.412	6	14	2.280	56	68
	73.457	7.885	6	14	2.006	68	79
	84.971	7.811	6	15	2.024	80	95
	61.149	20.103	15	32	4.300	50	70
	78.167	19.423	13	27	3.822	70	90
	84.151	33.398	24	41	4.030	70	96
	86.886	48.871	41	62	5.710	78	96
Low	38.626	9.193	6	19	2.960	25	44
	49.638	8.554	6	14	2.117	44	55
	60.258	9.270	6	15	2.495	55	66
	71.423	9.969	6	16	2.859	66	77
	82.808	9.853	6	24	3.442	78	94
	52.838	17.881	13	32	3.670	39	62
	68.736	21.449	16	33	4.078	55	86
	75.295	37.266	26	48	4.863	55	92
	82.319	52.812	43	69	5.446	69	93

Table 4 presents similar information as Table 3, except that it describes the clusters of rigid pavement subsets. Mean improvement was approximately the same for both traffic volume levels for most of the clusters, except for the clusters indicating the highest improvement values, the mean improvement for low-volume roads was slightly higher. Moreover, the maximum improvement for that cluster was significantly higher for low-volume roads than for high-volume roads. Compared to flexible pavements, the range of improvement and actual PCI is wider, while other characteristics seem to be the same.

Similarly, Table 5 summarizes the clusters' descriptive statistics for composite pavements with minor or no differences in improvement value between the low-volume and high-volume roads. The number of clusters used for the low-volume roads was seven, with three clusters representing the preventive maintenance whose ranges' were considerably wider than the clusters for other pavement types for the preventive maintenance region. However, the lower number of clusters did not affect the estimate of the mean improvement value, as well as the band shapes, were the same with no major changes.

As discussed, the proposed approach can be employed to detect maintenance activities for low- and high-volume roads. The estimated mean improvement or type of maintenance using this approach is approximate and not very precise since it uses unsupervised ML techniques. In cases where maintenance records are missing, thus; it would be hard to label the data to have the ability to use one of the supervised ML techniques such as classification to estimate the type of maintenance more accurately by using training data.

A remarkable difference in the mean improvement values for different pavement types for the same maintenance type (band) which can be noted by comparing values from the three tables. e.g., the mean improvement for the band with the highest improvement was 53, 60, and 69 for flexible, rigid, and composite pavements, respectively.

Hence, the proposed approach is very useful when maintenance records are missing or if a suitable training data set does not exist to train ML classification models to detect maintenance activities and the type of maintenance. Also, it would be very useful for assigning probabilities of how likely a segment is being maintained based on the band that the segment falls in. In addition, it can help to roughly estimate the type of maintenance. However, some sections are expected to be assigned to the adjacent band. For example, a section with major rehabilitation might have a high improvement value and then be grouped with the reconstruction cluster or vice versa.

TABLE 4 Clusters' Descriptive Statistics for Rigid Pavements

Traffic Volume	Mean ActualPCI	Mean Improvement	Min. Improvement	Max. Improvement	Improvement Std	Min. ActualPCI	Max. ActualPCI
High	39.657	9.571	6	19	3.520	25	48
	57.906	9.869	6	18	3.284	49	65
	72.012	8.788	6	18	2.676	65	80
	89.460	8.113	6	13	1.999	81	99
	63.109	22.657	16	36	4.443	45	76
	87.371	18.560	13	27	3.634	76	97
	84.392	34.873	25	46	5.226	64	98
	91.054	55.849	44	76	8.616	74	98
Low	36.754	9.671	6	23	3.239	3	49
	61.188	9.013	6	16	2.668	49	70
	78.475	8.991	6	14	2.267	71	85
	90.040	8.123	6	14	2.034	84	98
	55.270	20.894	14	36	4.505	25	71
	83.460	17.772	13	28	3.550	69	98
	74.821	34.691	26	49	5.487	39	98
	86.679	60.073	47	96	9.992	59	97

TABLE 5 Clusters' Descriptive Statistics for Composite Pavements

Traffic Volume	Mean ActualPCI	Mean Improvement	Min. Improvement	Max. Improvement	Improvement Std	Min. ActualPCI	Max. ActualPCI
High	42.461	9.302	6	18	2.842	26	51
	60.358	10.533	6	18	3.151	52	69
	78.441	10.270	6	20	3.519	70	87
	94.218	9.686	6	20	3.972	87	99
	49.031	19.492	12	31	4.353	39	58
	68.144	23.569	15	45	6.122	56	80
	89.129	30.403	20	42	6.079	76	99
	95.587	51.674	40	60	4.857	76	99
	96.532	67.696	60	79	5.170	87	99
Low	39.042	10.547	6	23	3.685	8	50
	59.413	11.644	6	22	3.893	49	75
	91.842	9.523	6	21	3.862	76	99
	49.892	25.282	17	48	5.455	28	66
	81.655	26.685	16	40	5.714	66	98
	91.746	49.849	37	59	5.666	65	99
	95.409	68.898	59	86	6.284	66	99

Practically, this approach will save the time and effort required to obtain a training dataset (labeled data) to be used by classification algorithms to detect maintenance. This approach will help transportation agencies in having a way to account for unrecorded maintenance actions, especially for municipal roads. Moreover, this approach incorporates using the ROD approach for condition forecasting when age data is missing. Thus, by using the ROD approach with the approach proposed in this research, agencies will be able firstly to detect maintenance activities on their network and then include segments with detected maintenance action in performance modeling by assigning an appropriate condition rest value to the maintained segments, then merging them into the modeling and forecasting process. So, it will be a comprehensive approach to dealing with missing age and maintenance records at once.

To further improve this approach, it would be useful to use ML classification techniques, allowing for a more accurate determination of the maintenance type. Also, it will allow accuracy reporting, unlike clustering, since there are no reference measurements to refer to for validation purposes. Besides, applying this approach at the agency level is expected to yield a better grouping for the data and, therefore, a better condition improvement estimation because each agency has a set of maintenance practices that may differ from the practices of other agencies due to differences between the two networks. Also, suppose it is required to boost the accuracy of clustering in this approach. In that case, it is encouraged to use initial centroid values representing the condition at which a treatment is implemented as the x coordinated of the centroid and the estimated condition improvement of a treatment as the y value, which will enhance the overall output. Another strategy is using more advanced clustering algorithms that might be considered semi-supervised learning algorithms.

Conclusions

For local agencies, missing maintenance records is a common problem that impacts performance modeling, forecasting future conditions, and pavement management practices in general. Some research articles discussed different methods to tackle this issue. Some studies have dealt with this issue from a performance modeling perspective and others from a maintenance detection perspective. This research proposed a data-driven approach that uses a combination of concepts, including the ML clustering technique, to detect road segments with probable maintenance action and to roughly estimate the maintenance type at those segments. The approach was applied

using data for flexible, rigid, and composite pavements at high-volume and low-volume traffic levels.

It was found that the mean improvement varies significantly for different pavement types, especially for the upper bands that contain segments with a higher probability of being maintained segments. Variations between the mean improvement values for roads with different traffic levels were not as high as variations based on the pavement type. Still, for some bands, the contrasts were notable. It was recommended to use ML classification techniques to improve the outcome of the proposed approach. Also, applying this approach at the agency level was encouraged to reduce the variability observed when applying it at the state level.

Author Contributions

The authors confirm contribution to the paper as follows: study conception and design: Al-Hamdan, Nlenanya; data collection: Al-Hamdan; analysis and interpretation of results: Al-Hamdan, Nlenanya, Smadi; draft manuscript preparation: Al-Hamdan, Nlenanya. All authors reviewed the results and approved the final version of the manuscript.

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Sign Life Expectancy on Low-Volume Roads

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Traffic signs provide warning and guidance information to drivers 24 h a day. They also represent a significant maintenance and replacement concern and cost for agencies with the advent of retroreflectivity requirements. In some cases, agencies choose to replace their signs in conjunction with the end of the manufacturer warranty period or other time intervals to ensure that signs maintain their retroreflectivity. However, this could result in signs being replaced while they still exceed their minimum retroreflectivity requirements, with labor and material costs being incurred years before necessary. This research evaluated retroreflectivity data from in-service signs in Iowa to determine expected sign life values. It evaluated 10,799 retroreflectivity data points across three different sheeting materials for a variety of sign types. A total of 65 linear regression models were developed to evaluate signs by sheeting type, age, and sign category (regulatory, warning, and guide), as well as directional orientation. The results indicated that all sheeting materials, sign types, and sign directions were predicted to have lives of at least 10 years before falling below *Manual of Uniform Traffic Control Device*s (MUTCD) minimums. Plots of retroreflectivity versus age indicated that many signs remained well above the MUTCD minimums at the predicted age where failure was expected. In general, the predicted lives for a material–sign type–direction combination were greater than 5 years longer than manufacturer warranty periods. For conservative purposes, agencies could consider a sign to be approaching minimum retroreflectivity levels at approximately 5 years past the manufacturer warranty based on the research results.

Survival Analysis of Concrete Overlays on Low-Volume Roads in Iowa

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Concrete overlays are an important component of Iowa's rural roadway network. More than 2,000 centerline miles of concrete overlays have been paved in Iowa over the last few decades, including many overlays on low-volume roads serving 400 vehicles per day or less. In this study, a survival analysis was performed using automated pavement condition data collected on Iowa's concrete overlays to obtain a probabilistic assessment of concrete overlay service life, with particular attention to the performance of overlays on low-volume roads. Concrete overlays were found to perform very well, with a 30-year survival probability of 85.3% for all projects to rehabilitation or reconstruction, and 76.6% for overlays only on low-volume roads. Survival life was reduced when the survival condition was changed to performance thresholds based on pavement condition index (PCI) and International Roughness Index (IRI), but still good overall. Overlays tended to reach the failure condition for PCI before IRI, indicating that cracking is a more common distress than surface roughness. Another important finding was that concrete on asphalt overlays had a longer median survival life and 30-year survival probability than concrete on concrete-unbonded overlays. Overall, concrete overlays have performed well in Iowa and are well-suited for low-volume roads and rural county highways.

PAVEMENT DESIGN AND CONSTRUCTION

Validation of a Compact Man-Portable Automatic Dynamic Cone Penetrometer

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The U.S. military requires a rapid means of measuring subsurface soil strength for construction and repair of expeditionary pavement surfaces. Traditionally, a dynamic cone penetrometer (DCP) has served this purpose, providing strength with depth profiles in natural and prepared pavement surfaces. To improve on this device, the U.S. Army Engineer Research and Development Center validated a new battery-powered automatic dynamic cone penetrometer (A-DCP) apparatus that automates the driving process by using a motor-driven hammering cap placed on top of a traditional DCP rod. The device expedites the DCP process by applying three to four blows per second while digitally recording depth, blow count, and California bearing ratio (CBR). An integrated global positioning sensor and Bluetooth connection allow for real-time data capture and stationing. Similarities were illustrated between the DCP and the A-DCP by generation of a new A-DCP calibration curve. This curve relates penetration rate to field CBR that nearly follows the DCP calibration with the exception of an offset. This offset was hypothesized to result from a change in energy response between devices and was demonstrated through an impulse study. Field testing of the A-DCP showed less variability and more consistent strength measurement with depth at a speed five times greater than that of the DCP with minimal physical exertion by the operator.

Mechanistic–Empirical Pavement Design for Low-Volume Roads Impacted by Freeze–Thaw

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Low-volume roads (LVRs), most of which are not designed, make up more than half the centerline mileage in the United States. The Cornell University Local Roads Program worked with local highway agencies in New York State to develop a mechanistic–empirical (M-E) pavement design tool that overcomes the limitations of expertise and time of most LVR highway officials but takes advantage of the knowledge of their own LVRs. The goal was to use existing information and apply them for the limited design related to LVRs. The tool developed, RoadPE: LHI, uses two common pavement fatigue criteria—surface tensile strain and subgrade vertical strain—with simplified inputs and built-in trend analysis to determine the thickness of the asphalt layers for overlaid, mill and filled, rehabilitated, and reconstructed LVRs. An M-E-based tool will allow the highway manager to prepare better designs to account for real-world variations.

Investigation of In-Situ Strength of Various Construction and Widening Methods Utilized on Local Roads

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The project goal was to develop and verify a low-cost, repeatable, nondestructive methodology to characterize the load carrying capacity of materials used in road widening and construction when established values are not available and establish a range of structural coefficients and moduli for these materials. A total of 99 test sites were selected from 68 projects in seven counties across Ohio, grouped into five clusters. These sites included 19 different widening treatments.

Each site was visited, tests were conducted, and specimens were gathered using the following techniques: falling weight deflectometer (FWD), portable seismic pavement analyzer, light weight deflectometer, dynamic cone penetrometer (DCP), and coring. The data and specimens collected were used to measure layer thicknesses, moduli, effective structural numbers, and layer coefficients applicable to each treatment. At least seven analysis methods were used to obtain the numbers from the data collected.

The results were plotted in box plot and cumulative frequency format for each material and each analysis method. For each material, there is a wide variability of values both within a section and between different sections. There are many sources for this variability. However, a range of numbers for moduli and layer coefficients can be identified for most treatments that can be utilized by local engineering personnel to design future projects.

The procedure based on Section 2.3.5 of the 1993 AASHTO *Pavement Design Guide* using FWD data provided the best estimate of published layer coefficients. However, the accuracy when using these values in other areas cannot be guaranteed; truly accurate layer coefficients come from careful monitoring of test sections under controlled loads.

Utilization of Recycled Concrete Aggregate Stabilized with Lime Fly Ash for Low-Volume Road Flexible Pavement

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India has 4.40 million kilometers of rural roads contributing to 73% of the total road network (*1*). Further, the Government of India planned to construct 7.5 lakh km to connect every corner of the country. The extension of this ambitious goal has been influenced by the shortage of quality materials near project sites. Simultaneously, India generates huge amounts of construction and demolition waste (CDW) but recycles only 1% of CDW. Using CDW as recycled materials in place of conventional aggregates results in economic and environmental benefits. Concrete waste, the main component of CDW, should be recycled, and its use as a replacement for natural aggregate (NA) should be encouraged (*2*).

The crushing of concrete waste into the required size, called recycled concrete aggregate (RCA), has become a substitute for NA in concrete. Still, due to its heterogeneous nature, high water absorption, low density, and high adhered mortar content prevent using RCA as a concrete material and therefore used as aggregate in pavement base or subbase courses in low-volume flexible pavement. The chemical stabilization technique is commonly used to enhance the strength and stiffness of pavement materials (*3*). Therefore, several studies blended RCA with cement and other cementitious materials such as fly ash (FA), ground granulated blast furnace slag, lime kiln dust, and cement kiln dust to improve the performance of RCA (*4–7*). FA is another byproduct material obtained from thermal power plants. In India, 200 million tons of FA are produced annually and 60% of generated FA is used to create portland pozzolana cement and embankment fill. The remaining unused FA is disposed to landfills, and the leachates generated from FA result in environmental pollution (*8*). Due to the slow hydration of FA in combination with cement or lime, it has been successfully

used as an alternative to cement (*9–10*). Hence, the use of FA should be supported wherever it is quickly and inexpensively obtainable.

The studies on using RCA and FA for pavement base course applications are limited. Therefore, the present study was carried out to explore the possible use of FA and RCA in the base course of low-volume roads (LVRs). RCA stabilization with lime FA (LFA) was studied and compared with cement. FA possesses no cementing characteristics; therefore, lime is used as an activator. The main objectives were:

- To investigate the mechanical and durability characteristics of RCA stabilized with LFA.
- To design low-volume flexible pavement as per IRC SP: 72-2015, with LFA and stabilized RCA as a base.

Materials

In the present study, the combination of lime and FA was used as a stabilizing agent, and RCA was used as an aggregate. The LFA of 10%, 15%, and 20% with a lime to FA ratio of 1:2 was considered to stabilize RCA. Cement content of 5% and 7% was used as a standard binder to stabilize RCA. LFA's effect on RCA's stabilization compared with RCA's cement stabilization.

The concrete waste was collected from a nearby site and processed by a compressive jaw crusher to produce RCA of the required size. The Ministry of Rural Development (MORD) (*11*) considered the particle size distribution recommended for soil-aggregate mixtures. The characteristics of RCA are presented in Table 1. Class F FA is used in the present work and collected from the thermal power plant. The chemical composition of FA was tested as per IS 1727, 1967 (*12*). The major constituents are silica 63.17%, alumina 21.35%, and calcium oxide (CaO) content of 5.52%. The specific gravity of FA was 2.30. Moderately hydrated lime, approximately 70% CaO by weight, was used, and the lime's specific gravity was 2.34. Pozzolanic portland cement was used in the present study.

TABLE 1 General Properties and Compaction Characteristics of RCA

Property	Result	Method	MoRD Specifications (2014)
Plasticity Index	Non-plastic	IS: 2720 (Part-5)	LL<25; PI<6
Specific Gravity	2.65	IS: 2386 (Part-3)	—
Water Absorption [%]	5.60	IS: 2386 (Part-3)	<2
Wet Aggregate Impact [%]	37.50	IS: 5640-1970	<25
Flakiness Index [%]	6.15	IS: 2386 (Part-1)	≤25
Compaction Characteristics of RCA, RCA Stabilization with LFA and Cement			
Material	**MDD[g/cc]**	**OMC [%]**	**Method**
RCA	2.0	10.1	
RCA+10LFA	2.062	9.5	
RCA+15LFA	2.056	10.1	IS: 2720(Part-8)
RCA+20LFA	2.025	10.3	
RCA+5C	2.15	6.52	
RCA+7C	2.13	8.51	

Mix Design and Methodology

Initially, a modified proctor compaction test was performed on RCA, RCA blends with LFA content of 10%, 15%, and 20%, and cement content of 5% and 7% to determine the maximum dry density (MDD) and optimum moisture content (OMC) using IS-2720 (Part 8) (*13*). Then, 7 days of cured unconfined compressive strength (UCS) in the case of cement, whereas 28 days of cured UCS for LFA is determined to assess the suitability of stabilized material about strength criterion of minimum 3 MPa for LVRs (*15*). The mixes which satisfy the required strength were subjected to 12 aggressive wet-dry cycles as per ASTM D 559 (*14*). The accumulated percent weight loss of stabilized mixes after the required number of cycles less than 14% can be considered the optimum mix design (*17*).

The particle size distribution for RCA is considered according to MoRD (*11*), shown in Figure 1. The compaction test outcome was used to calculate the weight of material required for cylindrical samples of (100 mm diameter and 200 mm length) for the UCS test, whereas cylindrical samples of (100 mm diameter, 60 mm height) for indirect diametrical tensile strength (ITS) and durability tests. All samples were prepared at OMC and compacted to MDD; as soon as extraction, the samples were wrapped in airtight polythene bags and kept curing for 7 and 28 days at room temperature. For the UCS test, cylindrical samples were subjected to axial loading of 1.25 mm/min till the sample get failed; maximum stress at
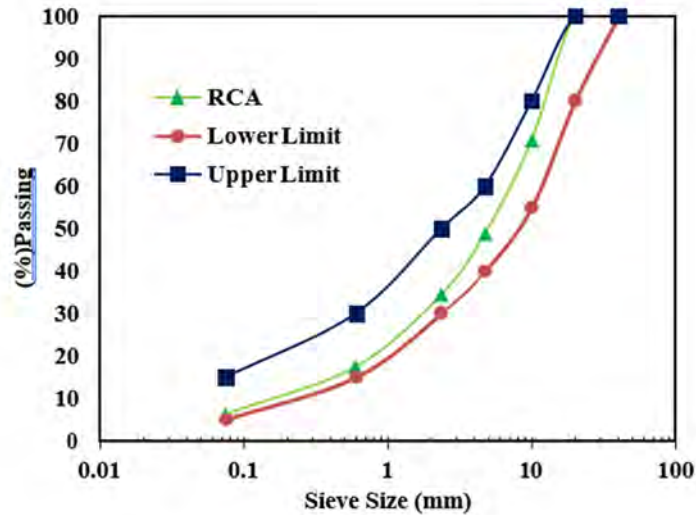
FIGURE 1 Particle size distribution of RCA.

failure was reported as UCS of the sample, whereas for the ITS test, the specimens were loaded at the rate of 1.25 mm/min along its vertical diametrical plane until failure, ITS of the specimen was calculated using failure load and geometry of the specimen. The durability test was conducted on 28-day cured samples subjected to 12 wet-dry cycles. Each wet- dry process consists of submerging the specimen inside water for 5 h and then drying the sample in the oven at a temperature of 72°C for 42 h (*14*). Three specimens were tested for each experimental condition.

Discussion

It can be observed from Table 1 that the MDD of RCA is increased with the addition of a stabilizer compared to that of RCA. This is expected because finer particles fill the pores of adhered mortar present on RCA and increase the density. The OMC for RCA is higher than RCA stabilized with cement and 10% LFA. This can be due to inadequate cohesion. OMC variation in the case of RCA stabilized with LFA was found to be indefinite due to increased finer particles and packing of finer particles in voids of RCA. Table 2 shows that all mixtures met the required UCS of 3MPa specified by IRC: SP 72-2015 (*15*) for cement-stabilized material cured for 7 days, whereas LFA-stabilized material cured for 28 days. The ITS of the cement-stabilized material shall be 0.1 to 0.125 times UCS (*16*).

**TABLE 2 Mechanical Strength Parameters of RCA
Stabilized with LFA and Cement**

Material	Average UCS (MPa)		Average ITS (MPa)
	7-Day	28-Day	28-Day
RCA+10LFA	1.30	4.80	0.70
RCA+15LFA	1.33	5.0	0.81
RCA+20LFA	1.40	5.20	0.85
RCA+5C	4.10	7.0	0.98
RCA+7C	5	8.40	1.25

Therefore, the ITS of all the mixtures was more significant than the recommended value. Figure 2 shows that all the combinations satisfied the durability criteria of accumulated percent weight loss of less than 14% after 12 wet-dry cycles (*17*). Accumulated percent loss decreases as the LFA and cement content increase. Therefore, from the above results, RCA stabilization with 15% LFA and 5% C are the optimum mixtures.
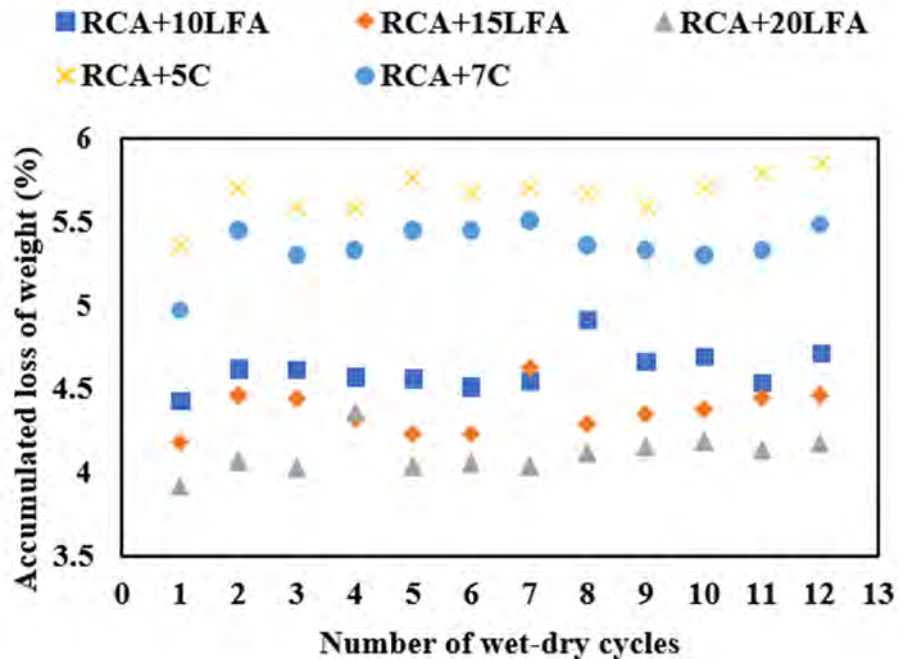
**FIGURE 2 Percent weight loss against a number of
wet–dry cycles during durability test.**

Pavement Design

In India, the LVRs' design is carried out per the code IRC: SP: 72-2015 and is based on the American Association of State Highway and Transportation Officials (AASHTO,1993). A pavement design using RCA stabilized with LFA and cement as a base course layer for different traffic, and subgrade conditions were carried out. The pavement cross-section consists of open graded premix carpet (OGPC) with seal coat, aggregate interlayer (AIL), RCA stabilized with LFA (LFRCA) as a base or RCA stabilized with cement (CRCA), granular subbase, and a subgrade. Except for RCA stabilized base, the remaining layer properties were kept constant. The pavement structure was designed for the following conditions: Design life of 10 years; a traffic growth rate of 5%; standard wheel load of 80 kN; tire pressure of 0.80 MPa. A structural layer coefficient of 0.20 was considered for RCA stabilized with lime FA, as recommended by Bartis and Metcalf (*19*), and the layer coefficient chart mentioned in AASHTO 1993 was used for RCA stabilized with cement. The results of the pavement design are summarized in Table 3.

Conclusions

The following are the specific conclusions of the present study:

1. The compact ability of RCA was found to be improved with cement or lime fly ash stabilization. The maximum dry density of RCA stabilized with LFA and stabilized with cement improved compared to RCA alone.
2. All the stabilized mixes met the unconfined compressive strength criteria of 3 MPa to use stabilized materials for base applications of LVRs according to IRC: SP: 72-2015. The durability studies in aggressive wet–dry cycles indicated that the RCA stabilized with 10% LFA performed satisfactorily and a percentage weight loss of less than 14% as per the requirements of IRC: SP 89 (2010). Therefore, RCA stabilized with 10% LFA is sufficient, but 15% LFA is recommended from a strength and durability point of view. Hence, the combination of LFA can be used as an alternative to cement to stabilize RCA.

**TABLE 3 Pavement Design Comparison Between Conventional Pavement and
Pavement with LFRCA and CRCA as a Base Course Layer for LVRs**

Traffic	Subgrade Condition												
	CBR (%) = 5					CBR (%) = 7				CBR (%) = 10			
	OGPC	BM	AIL	LFRCA	GSB	OGPC	AIL	LFRCA	GSB	OGPC	AIL	LFRCA	GSB
T7	25	—	75	270	100	25	75	230	125	25	75	200	125
T8	25	—	75	260	165	25	75	250	125	25	75	220	125
T9	25	50	75	265	165	25	75	250	150	25	75	235	125
Traffic	OGPC	BM	AIL	CRCA	GSB	OGPC	AIL	CRCA	GSB	OGPC	AIL	CRCA	GSB
T7	25	—	75	280	100	25	75	240	125	25	75	210	125
T8	25	—	75	275	150	25	75	250	150	25	75	230	140
T9	25	50	75	260	170	25	75	250	170	25	75	240	140
Traffic	OGPC	BM	WBM (G-3)	WBM (G-2)	GSB	OGPC	WBM (G-3)	WBM (G-2)	GSB	OGPC	WBM (G-3)	WBM (G-2)	GSB
T7	25	—	75	150	100	25	75	150	150	25	75	150	125
T8	25	—	75	150	200	25	75	150	200	25	75	150	175
T9	25	50	—	225	200	25	—	225	150	25	—	225	125

T7, T8, and T9 are traffic categories; T7> 0.6 – 1.0 MSA; T8>1.0 – 1.5 MSA; T9> 1.5 – 2.0 MSA; MSA = million standard axles; OGPC = open-graded pre-mix carpet; BM = bituminous macadam; LFRCA = RCA stabilized with LFA; CRCA = RCA stabilized with cement; AIL = aggregate interlayer; GSB = granular subbase; WBM = water bound macadam.

3. The pavement design demonstrated that RCA stabilized with LFA can replace conventional dense-graded aggregate base material for LVRs.

4. The current research results provide insight into recycling industrial by-products like RCA and fly ash and will be helpful for economic and sustainable pavement construction.

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Layered Viscoelastic Analysis with Moving Loads of Low-Volume Roadway Pavement Response

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Low-volume roads (LVRs) are typically not expected to withstand large amounts of traffic, consequently they have relatively thin pavement structures. However situations can occur when traffic loading rapidly increases, resulting in rapid structural deterioration of the pavement. Examples include resource exploration and production, new industrial facilities, and military operations. Improved evaluation procedures, including computation of pavement structural responses, are needed to accommodate these scenarios. This paper reports recent advances in methods for computing the structural response of LVRs that incorporate viscoelastic asphalt material behavior and accommodate moving loads. Application of layered elastic analysis, using a Hankel transform with numerical integration methods, is popular in several mechanistic-empirical pavement design procedures. This paper describes an application of the collocation method for approximating the creep compliance and viscoelastic solution, which removes the necessity for approximating the Laplace transform inversion. This was coupled with Boltzmann's superposition principle for moving loads. Comparisons showed that the proposed approximate method matched well with the viscoelastic solution, and with the full-scale instrumented test data of highway pavements. The measured near surface response of a thin LVR test pavement subjected to military truck loads was then modeled to determine the suitability of the approach for LVR. The modeled results for asphalt strain and vertical pressure in the granular layers showed generally good agreement with the measured instrumentation data. The developed structural response model provided a more realistic simulation of flexible pavement viscoelastic material behavior and accommodation of moving loads with similar computational speed to a conventional layered elastic approach.

Environmental and Structural Factors Influencing a Section with Different Interlayer Thicknesses of a Low-Volume Road Unbonded Overlay

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This paper examines 10 years performance of two contiguous test sections made up of a thin interlayer and a regular geofabric interlayer thickness respectively in a thin unbonded overlay pavement in a Minnesota Department of Transportation (DOT) research initiative. It comparatively analyzes environmental versus load effects in the constructed a 3-in. thick concrete overlay on a thick (conventional 3/16-in. thick, 16 oz/yd^2 unit weight) nonwoven geofabric interlayer and a thin (3/32-in. thick 8 oz/yd^2 unit weight) nonwoven geofabric interlayer each over a pre-existiing 6 ½-in. thick concrete substrate as contiguous subcells at the Minnesota Road Research Facility (MnROAD) facility in 2013. The study measured vertical displacement, International Roughness Index (IRI), and visual distress on the contiguous wheel paths in each of the two lanes where the thick standard weight interlayer (subcell 240) and the thinner interlayer material (subcell 140) were used. The different interlayer subcells exhibited statistically significant differences in deflection, IRI and tire-pavement interaction noise. Higher IRI values were observed in the thinner interlayer when wheelpaths of each traffic lane of the two subcells were compared. However, longitudinal, and transverse strain within the overlay and distresses appearing on the surface were more extensive in the thin interlayer subcell. This research thus justifies the prevalent preference of the conventional interlayer to the thinner interlayer. However, after 9 years in service, the traffic effect [inherent in comparing load transfer efficiency (LTE) in the environmental lane to that of the traffic lane LTE] was preponderant over interlayer thickness.

Introduction

Background

Increased standards and requirements for sustainable development have intensified the choice of unbonded overlays in concrete pavements in lieu of reconstruction. Unbonded overlays deploy the existing asset that has some structural value as substrate (*1–3*) for an overlay. They minimize construction costs that are otherwise expended in pavement replacement. Many state DOTs, including Minnesota DOT, still prefer unbonded overlays despite initially high investment cost and reduction of bridge clearance. Remarkable progress has been made in the transition from permeable asphalt stabilized stress relief course to nonwoven geotextiles. Nonwoven geofabric has been studied for general interlayer performance (*3*) and for lateral transmissivity (*4*, *5*). However, a 4-in. thick unbonded concrete overlay constructed in Michigan in 2011 reportedly experienced a minor acoustical issue. In the project, a relatively thick nonwoven geotextile (14.7 oz/yd^2) that was used as a stress relief layer resulted in noise from the concrete panels moving relative to each other at the joints under traffic (*4*). Researchers also observed objectionable vertical movements of the 6-by-6-ft slabs (*4*). Another research, showed that conventional interlayers provided sufficient drainage under load, using the Minne-ALF accelerated load frame (*5*). It is hypothesized that thick nonwoven geofabric interlayers are susceptible to detectable vertical motions due to overcompensation associated with Helmholtz resonance from moving traffic (*6*). Helmholtz resonance phenomenon in pavements has been explained by various authors including Izevbekhai (*6*). It is also hypothesized that thin interlays may exhibit poor stress relief and premature failure. However, the optimal fabric layer thickness with respect to overlay thickness design is not yet absolutely known. This research addresses that gap with the following clearly stated objectives.

Objectives

Noting the relative effect of loading and environmental factors, this research examines characteristics of two nonwoven geofabric interlayer dimensions, the effects of interlayer thickness on overlay performance and seeks to determine if the current 3/16-in. thick conventional interlayer is the optimum. More importantly, the research examines the 10-year performance of the test cell, pursuant to which ride and LTE over time are examined. Additionally, this research seeks to examine if thinner interlayers provide sufficient stress relief. Based on the above goals, research results will hopefully provide

some insight into the optimal fabric thickness for each condition and ascertain if the vibrations of the pavement due to interlayer are significant. Results of 5-year distress observation and results of IRI measurements as well as falling weight deflectometer (FWD) LTE provide guidance towards an optimum thickness of interlayer. The following experiment was designed to accomplish these objectives.

Experimental Design

Test Facility and Location

MnROAD is a full-scale outdoor pavement testing facility located in Monticello (about 40 mi northwest of downtown St. Paul) and is owned and operated by Minnesota DOT. The test track was constructed in 1993 to validate prevailing pavement designs and to improve new and efficient designs and materials. The original test sections were created between 1991 and 1994, and the tracks were open to traffic loading on July 15, 1994. MnROAD consists of three test tracks: the Mainline, a 3.5-mi two-lane westbound section that receives live traffic diverted off the adjacent I-94; the Old West Bound (OWB) lanes, which carry traffic a few days each month when traffic is diverted from the Mainline; and the Low-Volume Road (LVR), a 2.5-mi loop that is loaded with a five-axle 80-kilo pound semi-trailer that makes 80 laps per day for 5 days a week on the inside lane. The outside lane of the LVR is subjected to environmental loading only, which acts as a control in comparison to the inside lane. The second phase of MnROAD, from 2008–2016, focused on building new test cells and performing research in materials, design, and pavement surface characteristics studies. The facility is currently beginning its fourth phase of construction and research, which will focus on sustainability and resiliency. MnROAD continues to provide data and research findings to the public at no charge. Currently, MnROAD consists of over 80 test sections. The LVR consists of around 40 test sections, and the OWB consists of more than 40 test sections. Full details on the history and current test sections of MnROAD can be found on the MnROAD website (*1*).

This research will focus on cells 140 and 240 located in the LVR of MnROAD. These are part of MnROAD's phase two construction events, built in 2013.

Test Cells Design and Instrumentation

Prior to 2013, a test cell (Cell 40) was made up of a 6.5-in. thick concrete pavement built of regular aggregate base above a 2-ft thick layer of suitable grading material over on a sand silt subgrade. As one of the original test cells from the original 1993 construction it was a skew-jointed doweled concrete pavement with 20-ft joint spacing. In 2013 this cell was split into Subcells 140 and 240 comprising of the different interlayer thicknesses under a 3-in. thin concrete overlay. The thin and thick interlayers were placed in concrete overlay cells 140 and 240 respectively. The schematic layout of the substrate, interlayer and overlay are shown in Figure 1.

Apart from the interlayer thickness, there was otherwise no distinguishable difference in construction between the two subcells. Structural fibers were included in the concrete mix to possibly increase aggregate-interlock load transfer in this
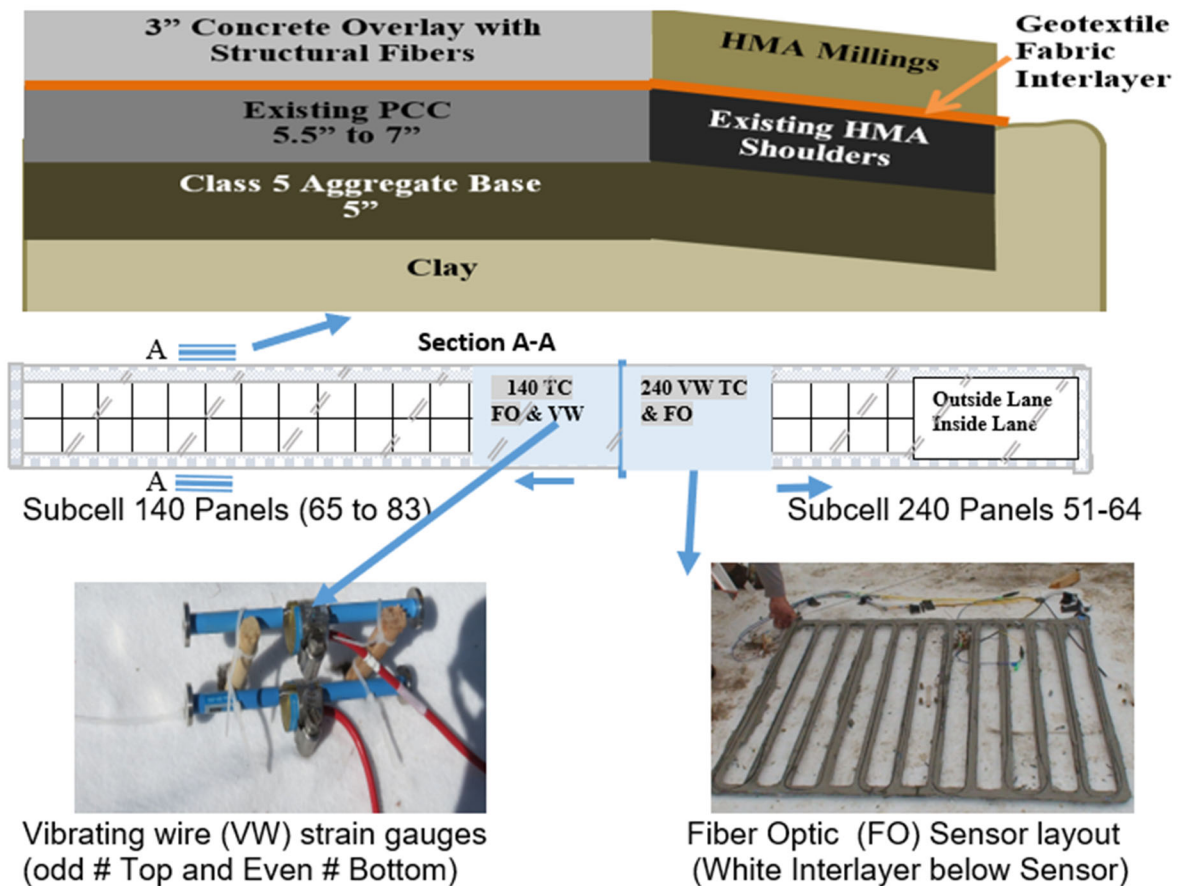
FIGURE 1 Panel and sensor layout MnROAD Test Cells 140, 240: vertical section (*top*); schematic plan (*middle*); and instrumentation (*bottom*).

undowelled overlay and the saw-cut joints were sealed with bituminous hot pour. The interlayer fabric was placed full width, above and glued at a 6-ft grid pattern to the concrete substrate and above shoulder-base before 3 in. of fiber-reinforced concrete was slip-form placed. Subsequently, 6-x 6-ft panels were established by joint sawing. The subcells were moderately instrumented for data collection as the experimental design leaned more to a study of fiber-enhanced LTE and durability of a thin unbonded overlay. Instrumentation was placed at the panels near subcell 140/subcell 240 transition such that the vibrating wire sensors in both subcells were connected to the same cabinet. Some fiber optic (polymer packaged fiber bragg grating) sensors were also placed in the same panel locations proximate to the vibrating wire sensors (*3*). It has been opined that fiber optic sensor types are less susceptible to electromagnetic interference than traditional vibrating wires (*3*). This was the first use of fiber optic sensors at the MnROAD facility (*1*). Figure 1 shows the layout of the test cell at the MnROAD facility.

Strain Analysis.

Fiber optic sensors had been installed in different directions and locations in the substrate and overlay to detect vertical, longitudinal, and transverse strain in the substrate and in the overlay. To compare strain in the thin interlayer subcell to that of the thick interlayer subcell, strain results observed at three different ages from construction were analyzed.

Vertical Amplitude Measurements.

The Ames Vertical Displacement Laser Unit was connected to the truck body with a system of rod-bracket assembly. The assembly was anchored to the tire hub by connecting directly to the hub bolts following manufacturers' safety instructions. The vertical amplitude measurement is an automated test roller that evaluates the actual displacement with respect to the undisturbed surface.

Four sets of runs (two in each wheelpath) were made in each lane of traffic after setting up the data collection unit and articulating an automated vertical displacement unit to a Minnesota DOT Truck weighing 31,000 lbs (Figure 2). The Ames Vertical Displacement Laser apparatus attached to each of the front wheels detected the displacement of the pavement surface with respect to a specified datum at very small increments of longitudinal distance along the pavement surface. The truck was driven in the same direction for each repetition in each lane. (The actual direction for the inside

(a)   (b)   (c)

FIGURE 2 Automated test roller designed for vertical displacement measurement: (*a*) mounting disk; (*b*) displacement laser device; and (*c*) details

lane being counterclockwise, implying that the test was conducted opposite the direction of traffic in the inside lane).

International Roughness Index Monitoring (Ride Quality)

The IRI was measured using two different accelerometers mounted on the Lightweight Inertial Surface Analyzer (LISA). The LISA is a profile device used to measure the amount of vertical rise over a horizontal distance based on the vertical response of the quarter car suspension algorithm (*7*). In the spectral domain the IRI is the average rectified value of the slope power spectrum density. This definition conveys a perspective of IRI in the waveform and frequency domain analysis as the quarter car is not a "rod and level" or "total station" but a mechanism with sprung and unsprung masses as well as spring and dashpot constants. Measurements were conducted with separate laser sources on the side of the vehicle: the ROLINE (1 kilo hertz) laser that takes continuous profile measurements over a 4-in. path and the TriODS (triple point) laser which measures three discrete profiles across the 4-in. path. The raw data collected from these lasers represented the two different IRI values.

Falling Weight Deflectometer Load Transfer Efficiency

Since the test cells were built in 2013, MnROAD Operations have measured FWD LTE. Since 1994, MnROAD has been using the Dynatest Model 8000 FWD to measure the response of pavement layers to different dynamic loads. The FWD, seen in Figure 3*a*, is composed of a loading plate, a weight package, geophone sensors, and data acquisition equipment. In Figure 3*b*, the schematics for the calculation of LTE are accentuated. The weight package is lifted hydraulically and dropped, thus imposing a dynamic load to the pavement. Geophone sensors capture the resulting deflection basin, which are used to evaluate the modulus of underlying layers and the structural capacity of the system.

Routine testing has been performed on cells 140 and 240 within the overall testing of the entire low-volume loop since their construction in 2013. Tests were performed in late spring and fall. FWD is performed at various locations in each test slab at the center, edge, corner, before the joint, and after the joint. This analysis focuses on the deflections before and after the joint, which help to evaluate the LTE of the dowels in the joints. To test LTE, the FWD trailer is placed so that the joint is between Sensors 1 and 3 or 1 and 10 as shown in the FWD trailer setup in Figure 3*a*. The load is applied at Sensor 1, and the deflection is measured on the leave slab and the approach slab. The LTE can then be calculated through a ratio of these deflections, as Figure 3*b*.

Visual Condition Survey

Periodic condition surveys were conducted on the LVR to observe, distress locations for initiation and progression of cracks faulting, spalling, patching, material related distresses such as d-cracking and ASR as well as construction-related defects such as crazing. These distresses include patching, cracking, faulting, and delamination.

Results

Evaluation of Strain

Fiber optic strain data was observed within the first year of testing (*3*). In a fair comparison, longitudinal, transverse, and embedded strain were individually compared to their corresponding values across subcells. The observed data shows that higher longitudinal strain was obtained in the test cell with thinner interlayer (Table 1 and

Figure 4). This difference was attributed to the difference in stress relief explained [2] who idealized mechanism of interlayer stress reduction as an attenuation of stress across the joint.

FWD Trailer Setup for LTE and Layer Moduli Analysis

Approach Pavement, Joint Before

$$LTE = \frac{D3}{D1} * 100\%$$

Leave Pavement, Joint After

$$LTE = \frac{D10}{D1} * 100\%$$
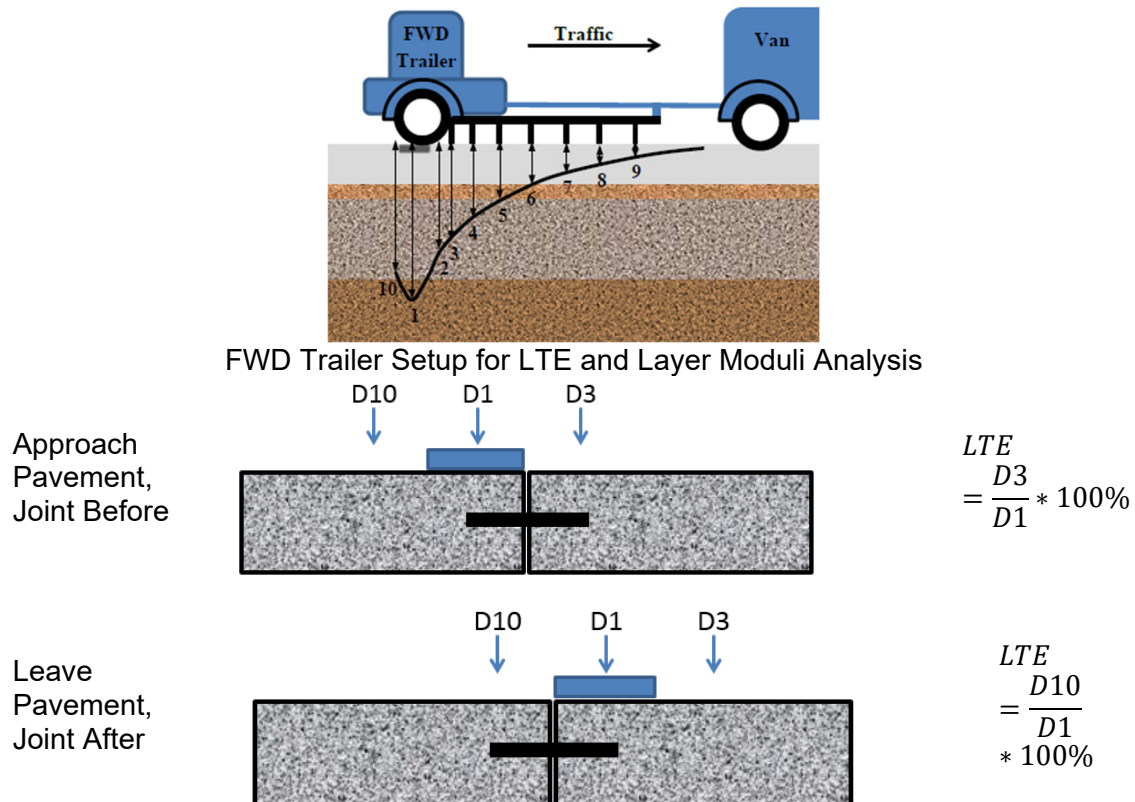
FIGURE 3 (*a*) **Dynatest Model 8000 FWD and** (*b*) **FWD sensor schematics setup and equations.**

Table 1 and Figure 4 show that in all cases (embedment sensor, longitudinal strain sensor and transverse strain sensor abbreviated in the table) higher strain levels were observed in the thinner interlayer subcell. Stress relief is usually facilitated by the compressibility of a functioning interlayer. This is indicated by the longitudinal embedded and transverse sensors indicated as circular, triangular, and rectangular marker points respectively, where the values for the thinner cells are shown as solid markers and the values for the thicker fabric as shown as hollow markers. The results suggested that the thinner interlayer fabric may not have provided as much stress relief as the thicker fabric. It is noteworthy that the strain is measured by vibrating wire sensors or by fiber optic sensors within the pavement but deflections (measured by the Ames Vertical Displacement Laser) are topical measurements of the surface as would be detected in a test rolling operation. The deflection which is that of the overlay on the

TABLE 1 Strain With Respect to Time in Thick and Thin Interlayer Subcell

Days	Cell 140 Thin Interlayer μ-strain			Cell 240 Thick Interlayer μ-strain		
	140 Long. 3D2T2	140 Embed 1D5	140 Trans 3D2T1	240 Long. 1D3	240 Embed 1D1	240 Trans 3D1T2
24	348.00	−1251.00	388.00	106.00	−197.00	327.00
55	420.00	−956.00	423.00	73.00	−211.00	291.00
89	447.00	−927.00	388.00	29.00	−179.00	209.00
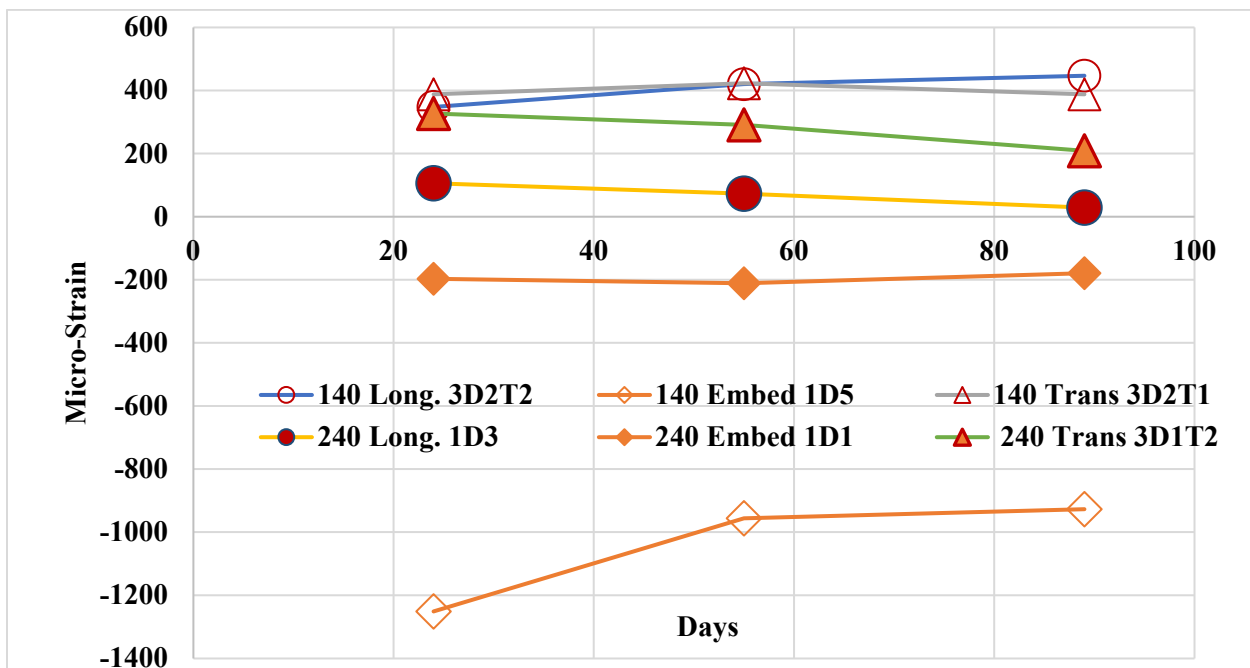
FIGURE 4 Results of comparative fiber optic strain observation.

substrate through the interlayer is measured topically whereupon the substrate is the reference and compression is mainly in the interlayer. In some situations, compression could result in overcompensation (Helmholtz resonance) that could produce higher amplitudes than what is structurally expected. In any case while displacement of the overlay, interlayer, substrate, and base may not be absolutely ignored, these are considered inconsequential in comparison to the interlayer induced displacements that are being measured. Therefore, comparative analyses of performance of thick interlayer section to thin interlayer section are done. Similarly, the performance of the loaded lane is compared to that of the unloaded lane in each of the interlayer sections.

Vertical Amplitude Results

Author tabulated the vertical amplitude profiles of each wheel in each lane across the two subcells with their corresponding station along the pavement. The vertical amplitudes were examined in subcell 140 and subcell 240. The wheel path which are left wheel path (LWP) and right wheelpath amplitudes were also compared between subcells and the LWP values to each other.

The maximum and the overall range for each data set from each trial were obtained by taking the difference between consecutive amplitudes (Figures 5 and 6). Maximum positive amplitudes were generally higher in Cell 240 (thicker interlayer) than Cell 140 (thinner interlayer). Clearly, the maximum negative amplitude and the maximum positive amplitudes are higher in the thicker interlayer than the thinner interlayer. This shows that the thicker inlerlayer experiences more vertical displacement for compression and rarefaction. In addition to the maximum negative and maximum positive vibration comparison, higher amplitude ranges were also computed in the thicker interlayer (Figure 5). In Figure 6 there is again evidence that the thicker interlayer is associated with more deflection or vibration in the interlayer. The dynamic influence of the interlayer can be analyzed using the principles of beams on elastic foundations (10) where the modulus of subgrade reaction can be assigned to the interlayer, but the analysis must be extended to simulate the influence of a traveling load on a discontinuous slab.
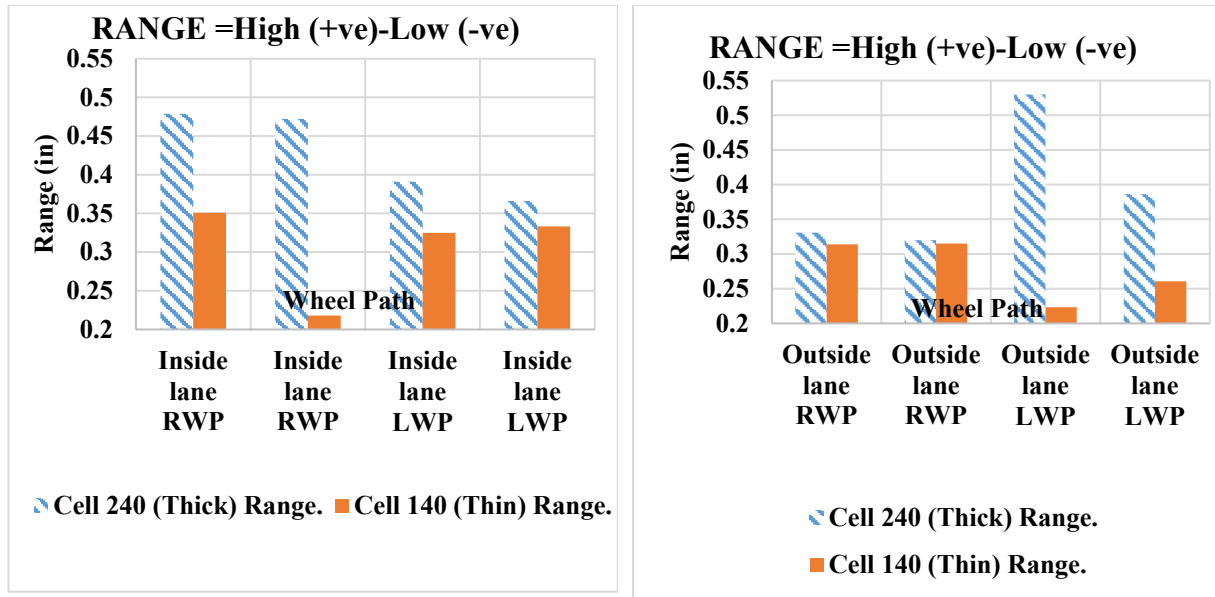
FIGURE 5 Comparison of range of vibration/deflection between conventional and thinner interlayer subcells (two data sets) (*a*) outside lane and (*b*) inside lane.
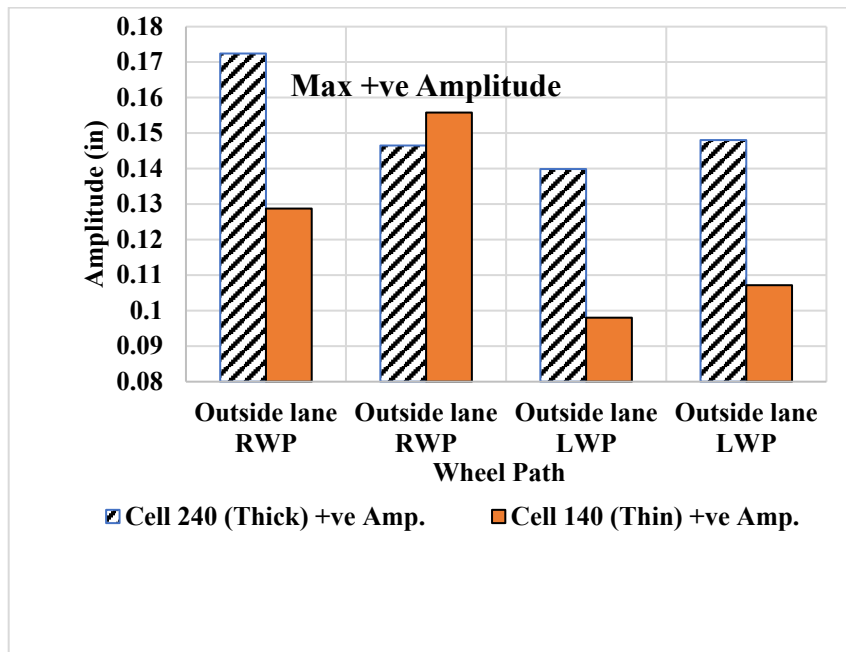
FIGURE 6 Maximum positive and negative amplitude between subcells in outside lanes.

Data shown in Table 2 is an excerpt of the results of vertical displacement measurements conducted in Cell 140 inside lane with about 12 rows (out of 7000 rows). Generally, it depicts a relative drop in the tire suspension with respect to the bodyFigure 2*b* shows a laser device at the wheel hub and a mounting structure connected to the vehicle body. The laser follows a displacement feature that captures and measures the movement of the pavement surface. Essentially, this technology was adopted from the original idea of the device for measuring ruts or pumping in test rolling. An analytic method is to use range, maximum negative and maximum positive displacement as descriptors or use comparative analysis of the same measurements taken in two cells as a descriptor.

International Roughness Index Results.

The outside lane is the environmental lane, and the inside lane is the traffic lane in the regular application of repetitions of the 80 kip 5-axle semi-trailer. Cumulative damage effects are typically more evident in the loaded lane in contradistinction to the unloaded lane. Results of IRI versus time shows higher IRI values in the thick interlayer subcell than in the thin interlayer subcell left wheelpath (Figure 7) where the data spread is higher with the thinner interlayer subcell, the lower bound values of the IRI of the thicker interlayer subcell are at the 50-percentile mark of the IRI distribution of the thin

TABLE 2 Result of Laser Displacement Measurement Ames Automated Test Roller

Highway = MnROAD LVR Cells 140 and 240		Date: June 17 2017					
Lane	Inside						
Lane Location = LWP							
Pass	10						
GPS Report (ft)	Latitude (ddmm.mm)	HDOP	Satellites	Geoid (ft)	ODS1 Profile	ODS2 Profile
Distance	Station						
0.015	1223+00.01				−0.03344	0.020526
0.03	1223+00.03				−0.03341	0.016815
0.044	1223+00.04				−0.03342	0.014584
0.059	1223+00.06				−0.03316	0.013812
0.074	1223+00.07				−0.03191	0.014889
0.089	1223+00.09				−0.02922	0.013346
0.104	1223+00.10				−0.02533	0.010961
0.118	1223+00.12				−0.02321	0.009007
0.133	1223+00.13				−0.02187	0.004402
0.148	1223+00.15				−0.02078	0.003461
0.163	1223+00.16				−0.01906	0.002633
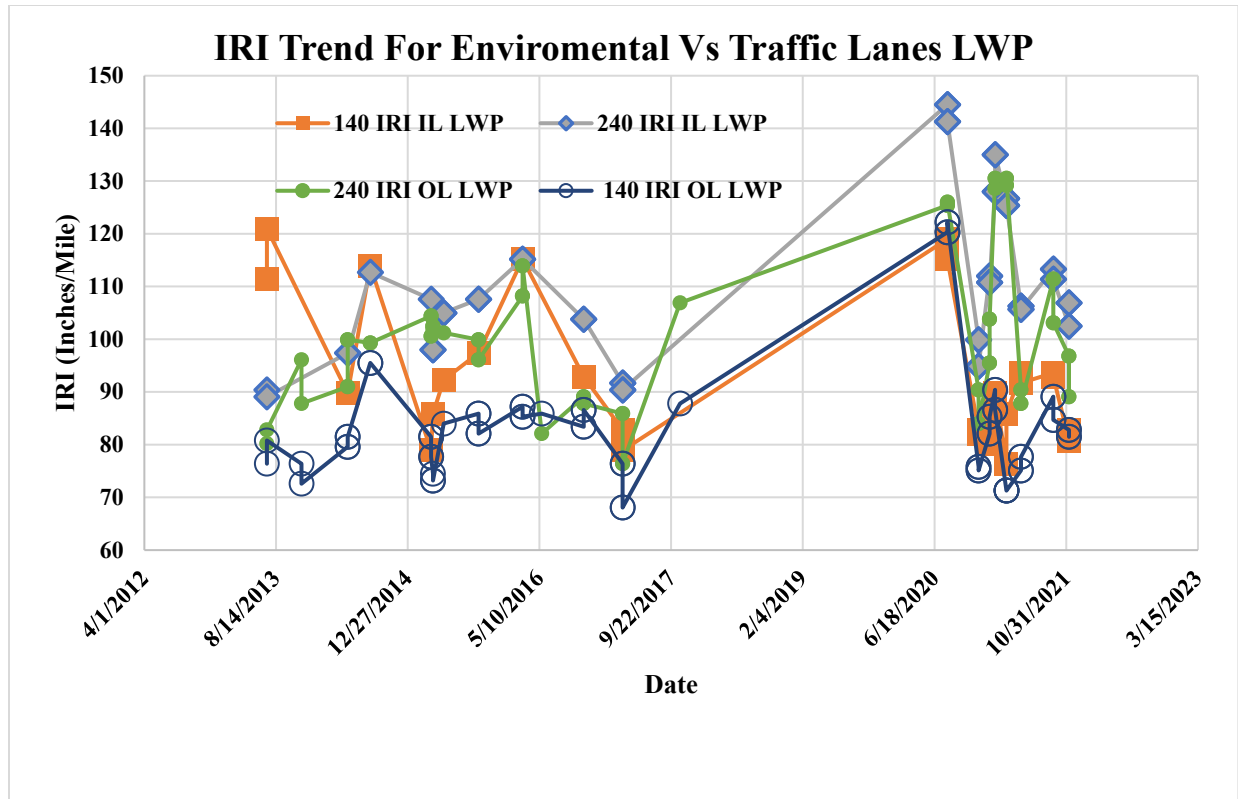
FIGURE 7 IRI results in the left wheelpath.

interlayer subcells. In the traffic lane, the IRI was higher in the thinner interlayer than the thicker interlayer subcell. This can be explained by the higher strains observed in the thin interlayer by Huang et al. (*3*). Observation of high IRI in the thinner interlayer is attributable to initiation of degradation in this subcell due to insufficient stress relief. This observation led to a review of the distress survey records on the test cells to ascertain if load related degradation was visually evident particularly in subcell 140.

Generally, the IRI trends show some degree of seasonality that may require detrending. However, averaging data from measurements that were conducted on the same day in each lane minimize the likelihood of seasonal variability in a comparative analysis. In Figure 6, IRI in the inner lane of Cell 240 appears to be consistently higher than that of the outer lane of Cell 240. Similarly, the outer lane of Cell 140 appears to be higher than outer lane of Cell 140. These are explained by loading which occurs only in the inner lane. However, in the inner lane cells 140 and 240 appear to maintain similar IRI trend especially long term whereas in the outer unloaded lane, Cell 240 now shows higher IRI than Cell 140 a thus accentuating the preponderance of traffic effects over interlayer type in the past 9 years.

In Figure 8 the inner lane right wheelpath IRI of Cell 240 appears to be consistently higher than the outer lane of Cell 240. Moreover, the inside lane of Cell 140 appears to be higher than the outside lane of Cell 140. These are explained by traffic loading which occurs only in the inner lane. In the inner lane Cell 240 exhibits higher IRI than Cell 140 and in the Outer Lane Cell 240 exhibits higher IRI than Cell 140. This again validates the preponderance of traffic over interlayer thickness in ride performance as observed in these cells.

Load Transfer Efficiency

An original presentation of every FWD test result resulted in a busy unwieldy plot where the average LTE was obtained and plotted in Figure 9. For ease of presentation after all the test results had been obtained, the LTE daily averages of measurement of each cell and lane were plotted (Figure 9). From Figure 9 the LTE for Cell 240 inside lane is clearly lower than the LTE of Cell 140 in the inside lane. This is explained by the higher amplitude of vibration due to the interlayer thickness compounded by the lightness of the 3-in. overlay. Such excessive vibration compromises LTE especially in the loaded lane. In the unloaded (environmental lane) Cell 240 exhibits generally similar LTE to Cell 140 implying that with the thinner interlayer where vibration amplitudes are not excessive, there is a lesser influence on actual LTE, it also implies that the influence of the interlayer thickness on LTE is not dominant over the relative effect of traffic load (traffic lane versus environmental lane).
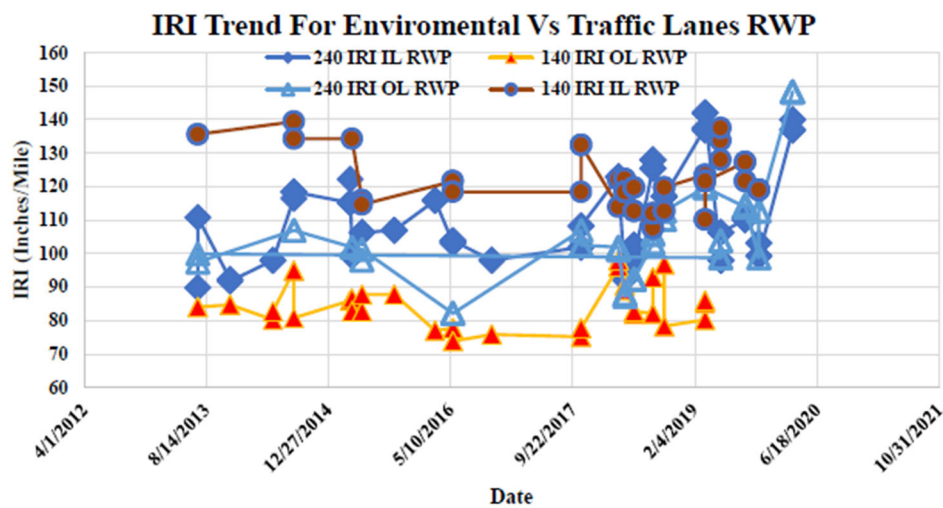
FIGURE 8 IRI observed in the right wheelpath.
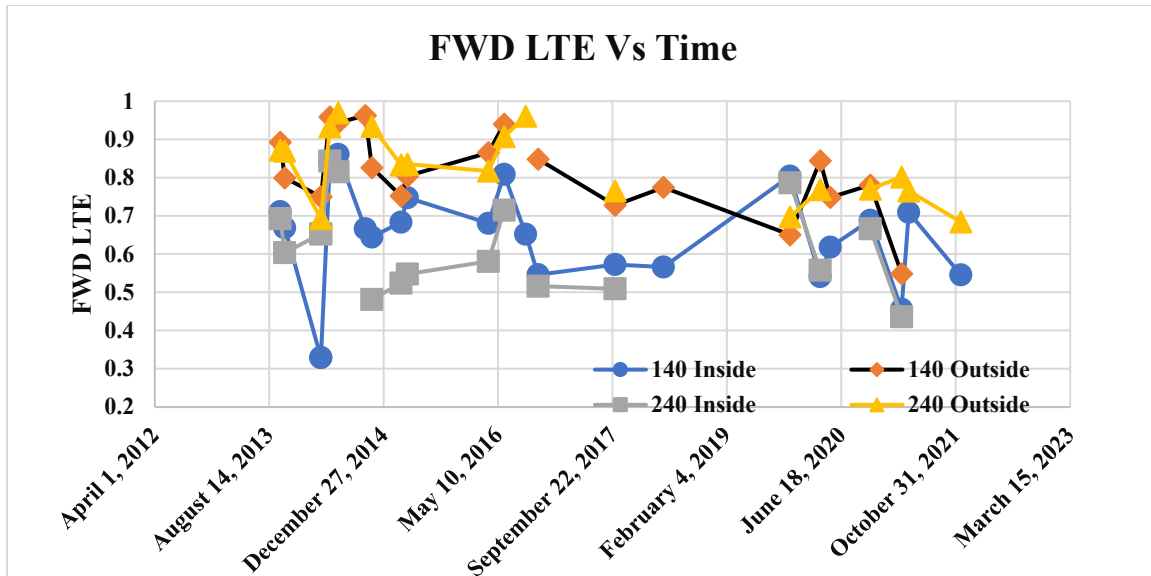
FIGURE 9 LTE versus time.

The equality plot in Figure 10 accentuates the differences and similarities between the various lane LTE values. The equality line as expected to coincide with the Cell 140 outside lane which is the thin interlayer in the unloaded lane. Any data set below preponderantly below the equality line indicates lower LTE than this reference cell and lane. As in Figure 10 the thick interlayer outside lane appears to coincide with the equality line of course with a different spread of data. Apparently, the similarity between the outside lane and the inside lane indicates that traffic loads may be influencing LTEs more than the interlayer thickness. This is further accentuated by the similarity between the distributions of the two cells of different interlayer thickness in the traffic lane. Relative to the line of equality they are both lower than this line which is an indication of the fact that traffic in these cells may have reduced the LTE over time. Here the effect of traffic is preponderant over the thickness of interlayer.

There is a distinct difference between the environmental lane LTE (higher LTE) than the traffic LTE. However, in the thick interlayer Figure 10 shows that the environmental lane also exhibits higher LTE than the traffic lane. Here the effect of traffic is preponderant over the thickness of interlayer.
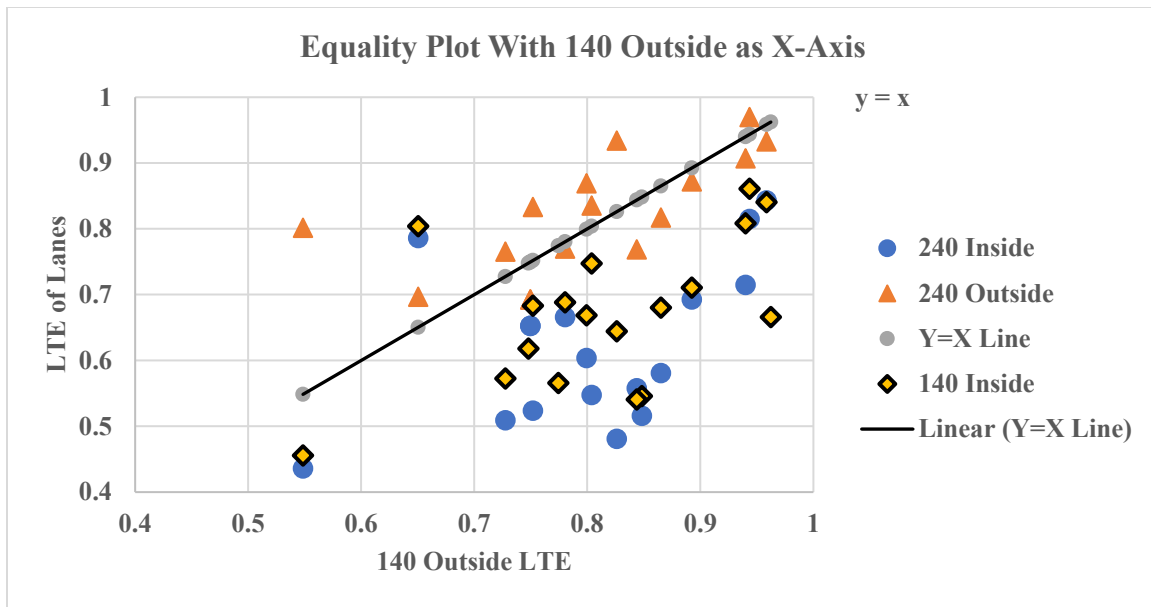
FIGURE 10 Comparison of LTE across lanes (inside and outside lanes) and across interlayers thickness (Cell 140 and Cell 240, thin and thick interlayer, respectively).

Visual Distress Survey Results

Test cells in the MnROAD research facility are visually examined periodically for cracked slabs, joint sealer failure, spalling or raveling. Although these visible distress signs do not necessarily reflect ride quality, they are usually transcribed into a separate index known as surface rating (SR) based on the type of distress, frequency of occurrence and severity. Absence of SR quantification in Subcells 140 and 240 necessitated a count of the visually evident distress modes. Joint sealant degradation was prevalent in the inside (loaded) lane of the two subcells which indicated fair to good sealant condition in the environmental lane and fair to poor in the traffic lane. Distress forms thus appeared to be more traffic volume (and not interlayer) related. There were three transversely cracked panels in Cell 240 and six transversely and three longitudinally cracked panels in Cell 140. However, a pre-existing utility crack in the pre-existing cell reflected through the overlay in Cell 240 immediately after construction. That transverse crack across both lanes was not counted since its presence is extraneous to the influence of the interlayers.

Conclusions

This research examined two contiguous concrete unbonded overlay subcells with different interlayer thickness: the conventional 3/16-in. thick weighing 16 oz/yd^2 and the 3/32-in. thick interlayer weighing 8 oz/yd^2 respectively. Results obtained conclude that compressive and tensile strain were higher in the thinner interlayer test cell. The amplitude of vibration of the panels due to interlayer compression was higher in the thicker interlayer. Additionally, the surface distresses were more in the thinner interlayer subcell than the thicker interlayer subcell. In the acoustic domain, the thinner interlayer subcell was about 4 dB quieter than the conventional interlayer subcell. This observation was attributed to Helmholtz resonance that tends to amplify spectral and acoustic amplitudes resulting in objectionable low frequency noise and panel rocking effects. Whereas huge test cell construction and instrumentation cost limited this study to only two interlayer thicknesses, there is a preponderance of evidence of vibration due to interlayer thickness. However, the higher strain and prevalent surface distresses in the thinner interlayer suggest that the thinner interlayer may not be providing sufficient stress relief. Nevertheless, amplitudes observed in the 3-in. concrete overlay test cells are likely to be reduced when overlays are thicker. This study thus justifies the use of the current thick conventional interlayer until other thicknesses are examined and they are proven to exhibit better properties. This research however does not dispute that interlayer thicker than the conventional 16 oz/yd^2 may result in uncontrollable amplitudes that can be counterproductive. LTE for Cell 240 inside lane is clearly lower than the LTE of Cell 140 in the inside lane. This is explained by the higher amplitude of vibration due to the interlayer thickness compounded by the lightness of the 3 in. overlay such excessive vibration compromises LTE especially in the loaded lane. In the unloaded environmental lane Cell 240 exhibits LTE generally similar to Cell 140, implying that with the thinner interlayer where vibration amplitudes are not excessive there is a lesser influence on actual LTE. The equality plot in Figure 8 accentuates the differences and similarities between the various lane LTE values. The equality line as expected to coincide with the Cell 140 outside lane which is the thin interlayer in the unloaded lane. Any data set below preponderantly below the equality line indicates lower LTE than this reference cell and lane. The thick interlayer outside lane appears to coincide with the equality line of course with a different spread of data. The similarity between the outside lane and the inside lane indicates that traffic loads may be influencing LTEs more than the interlayer thickness. This is further accentuated by the similarity between the distributions of the two cells of different interlayer thickness in the traffic lane. Relative to the line of equality they are both lower than this line which is an

indication of the fact that traffic in these cells may have reduced the LTE over time. Here the effect of traffic is preponderant over the thickness of interlayer.

Author Contributions

The authors confirm 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. All authors reviewed the results and approved the final version of the manuscript.

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Bridges and Structures

Field Demonstration of an Innovative Box Beam Connection

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Brent M. Phares
Justin Dahlberg
Iowa State University

Adjacent concrete box beam bridges constitute more than 15% of the bridges built or replaced each year. This type of bridge is generally constructed by placing box beams next to one another, grouting adjoining shear keys, applying a transverse post-tensioning force, and then, perhaps, placing either a thin wearing surface or a thick (6 in.) structural deck. Historically, these adjacent precast elements have suffered from differential displacements, which cause cracking in the joint material. Cracking of the shear key between adjacent box beams appears to be a service-related problem. Even with a cracked joint, a bridge can continue to distribute loads effectively throughout the primary load-carrying members (Huckelbridge et al. 1995). Cracking does not seem to be first initiated by the application of live loads. There are, however, differing opinions on the relative contribution to cracking from shrinkage and temperature. Nevertheless, once cracking is initiated by either shrinkage and/or temperature, cracks can continue to grow with subsequent live load application. (Miller et al. 1999; Sharpe 2007; Attanayake and Aktan 2008; Grace et al. 2012).

Liu and Phares (2019, 2020) conducted multiple levels of material tests and analytical studies to select the most suitable crack-resistant material associated with various joint configurations. The 6½-in. wide joint filled with shrinkage-compensating concrete was found to perform superior to all the other choices. Based on that, an innovative 6½-in. wide joint was designed without a shear key, incorporating other concepts, such as shrinkage compensating concrete achieved by using Type K cement, form retarder used to create a rough surface on the sides of the box girder to increase shear and bond capacity, and reinforcing steel bars that crossed the interface between the joint and box girder. The design was first evaluated with a series of small-scale tests and analytical models to investigate its effectiveness at an early age (first week after placement) and subject to the ultimate load. Following that, a series of laboratory tests on a full-scale innovative joint subject to temperature loading, vertical cyclic loading, and

horizontal loading was calculated. (Liu et al. 2020; Shi et al. 2019a). Although the previous laboratory test and analytical simulation results indicated that the joint shows superior performance in resisting the joint early-age cracking and keeping the integrity of the bridge superstructure, its performance on a real on-site bridge were not evaluated.

This paper aims to evaluate the on-site performance of the innovative joint design proposed by Liu and Phares (2019) on a box beam bridge in Washington County, Iowa. To achieve this objective, the bridge was instrumented with a combination of embedded and surface-mounted sensors during the construction. To evaluate the joint performance, the bridge was monitored during the first seven days after joint material placement. During the monitoring, the strain in the box girder and longitudinal joint induced by the early-age material self-volume change and temperature effect was captured. In addition, visual inspections were conducted with a focus on the condition and performance of the joint, with a specific interest in the development of any cracking.

Methodology

Bridge Design and Joint Construction

The subject bridge is a single-span integral abutment bridge 32 ft in width and 70 ft in length. The bridge superstructure consists of 8 box beams, each approximately 4 ft wide. Figure 1 shows a cross-section view of the bridge superstructure with labels for each box girder and joint. The bridge has zero skews. The innovative joint on this bridge was designed consistent with the design by Liu and Phares (2019; 2020), including the following features: wide joint (6½ in.) without a shear key, shrinkage compensating
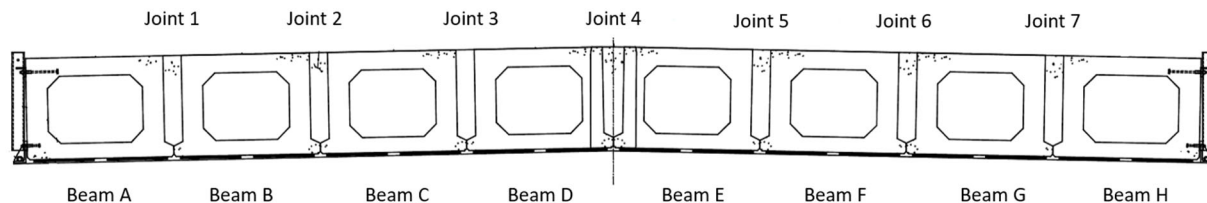
FIGURE 1 Bridge cross-section view.

concrete mixed with Type K cement, form retarder used to create a rough surface on the sides of the box girder to increase the shear capacity, and reinforcing steel that crosses the interface between the joint and box girder. The box girders were cast and post-tensioned in a precast concrete plant. Before the beams were placed into the position, the 90° hook bars were screwed into the dowel bars that were embedded into the precast box girder. After the beams were placed in position, the joint reinforcement, including stirrups and the longitudinal straight rebar, was installed, as shown in Figure 2.

Joint Material Property

In order to have a qualitative evaluation of the field used joint material, multiple material properties tests were conducted with a focus on capturing the time-dependent compressive strength, splitting tensile strength, and self-volume change (expansion). Shrinkage testing was conducted following the provisions outlined in ASTM C157 (ASTM 2017a). The average shrinkage of each truck concrete is shown in Figure 3a. Both trucks of concrete expanded about 200 to 310 microstrains during the first 24 hr. The compressive strength was tested following ASTM C39 (ASTM 2018), and the splitting tensile strength was conducted following ASTM C496 (ASTM 2017b). Figures 3b and 3c show the material test results for the compressive strength and tensile strength, respectively. The results indicated that both trucks of the material showed similar compressive strength on the 28th day of 5.2 to 5.8 ksi. This is higher than the bridge drawing required compressive strength (4 ksi) for the joint material. For the

(a)   (b)
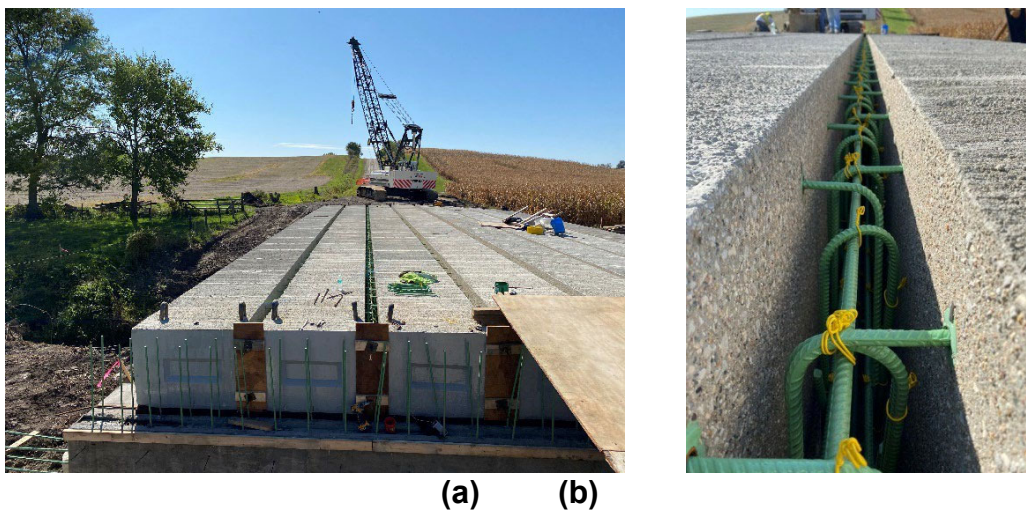
FIGURE 2 Longitudinal joint before placement of joint material.
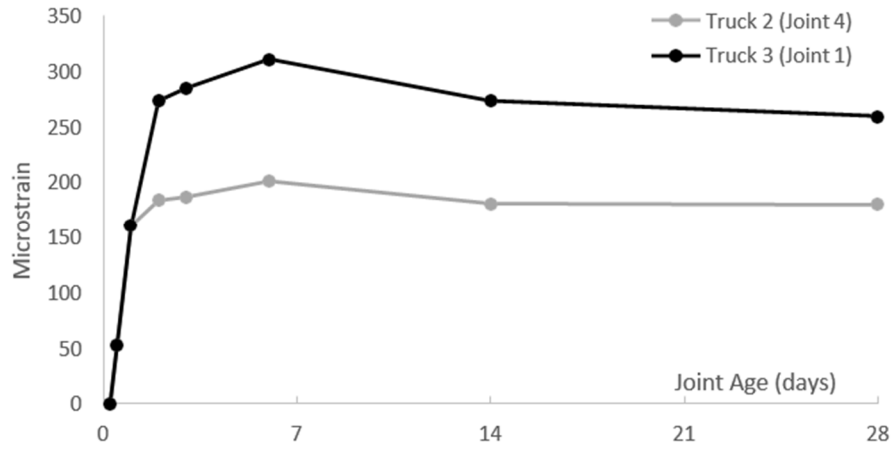
(a)
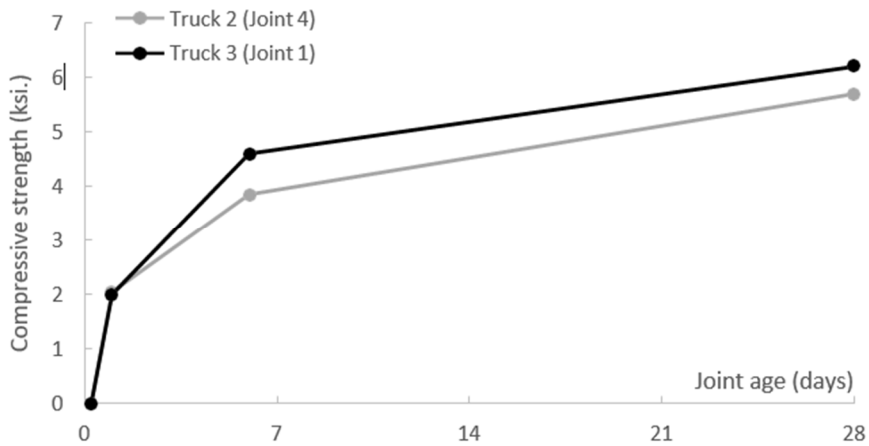
(b)
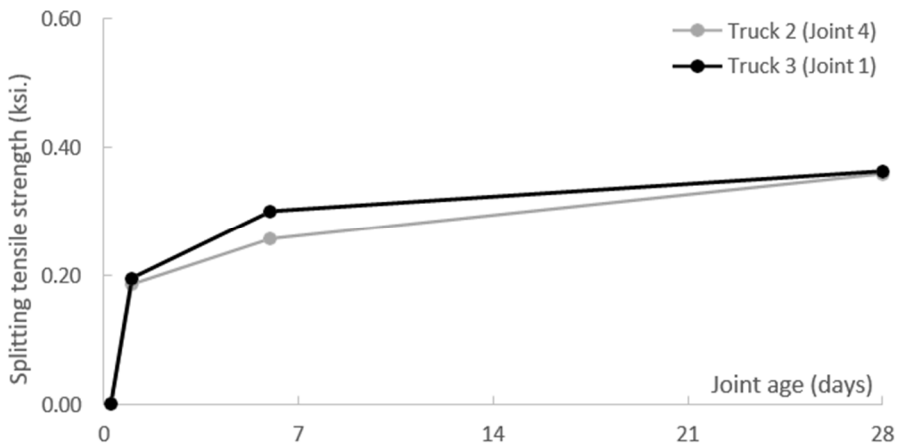
(c)

FIGURE 3 Material test results: (*a*) shrinkage test results; (*b*) compressive strength, and (*c*) splitting tensile strength.

tensile strength, both truck materials also resulted in a similar strength of 0.4 ksi on the 28th day. The time-dependent behavior of the material indicated that Type K cement concrete obtained a significant strength increase during the first 24 h.

Early-Age Performance Monitoring

Since the results from previous research indicated that cracks tend to be initiated during the early age of the joint material, the early-age joint behavior was monitored for the first seven days. The joint early-age behavior was monitored with a focus on the bridge transverse behavior since the past study indicated that most of the cracks in the joint or the debonding at the interface formulate in the longitudinal direction. Figure 4 shows the instrumentation plan for early-age monitoring. Two joints were selected for the instrumentation: Joint 1 and Joint 4. In total, 24 vibrating wire strain gauges were utilized, with 16 embedded in the box girders and 8 embedded in the joints. Each

(a)

(b)
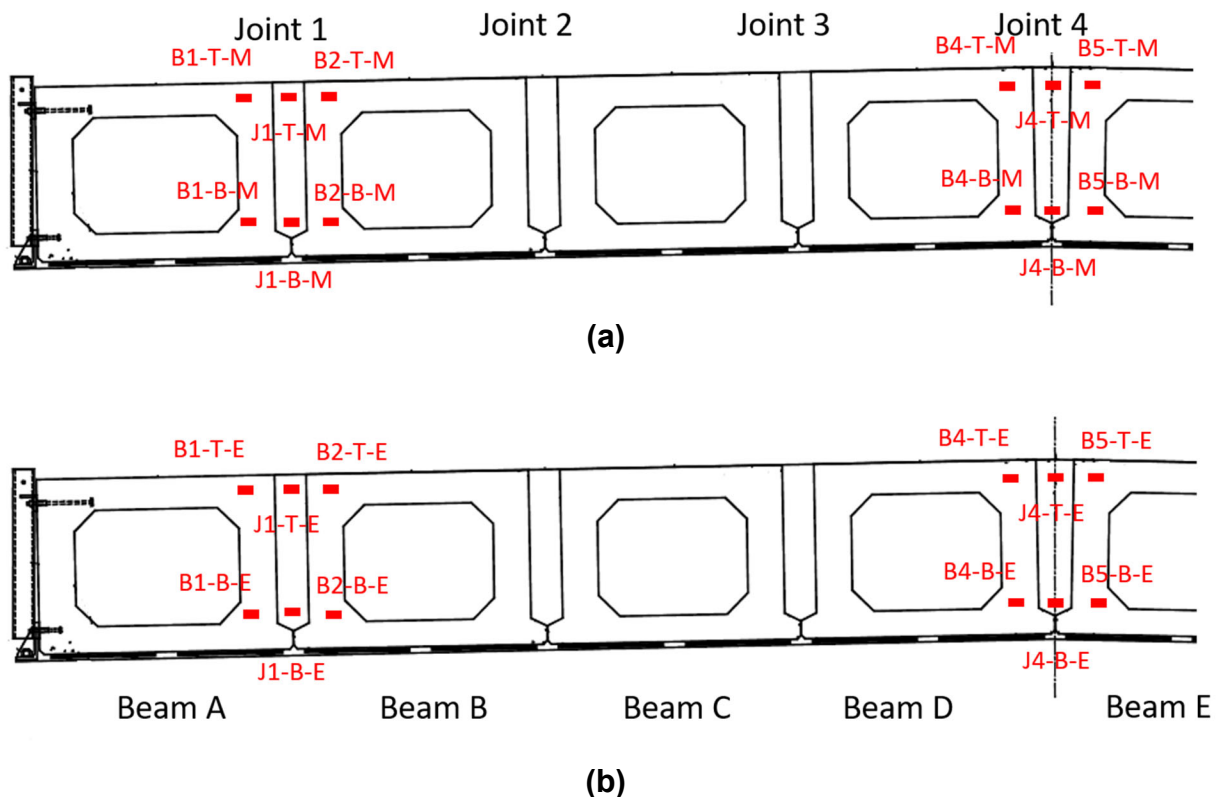
FIGURE 4 Early-age monitoring instrumentation plan:
(*a*) mid-span section and (*b*) end-span section

vibrating wire strain gauge can collect both strain and temperature data. The data collection started immediately after the joint concrete placement at 12:20 p.m. on October 07, 2020, and ended when the joint material was 7 days old. A frequency of 10 min was used. In addition, multiple visual inspections were conducted in October 2020, May 2021, October 2021, and August 2022.

Findings

Field-Collected Temperature Results

Figure 5 shows the temperature measured from selected vibrating wire strain gauges embedded in the box girder (Beam A and B and Joint 1). The data from the other gauges showed similar results and was not presented in this paper. Figure 5*a* shows the temperature measured from the mid-span. It was found that the gauges near both top and bottom of the girder follow a typical daily temperature cyclic during the first seven days of the joint age: higher temperature in the daytime and lower temperature at night.

Figure 5*b* shows the temperature data collected from the end span. The results indicated that the temperature near the end of the span followed the daily temperature cycle during the first two days, and a temperature increase of about 10°F occurred on the third day (October 9, 2020). Figure 5*c* shows the temperature data collected from the gauges embedded in the longitudinal joint. The data from the gauges at the mid-span shows similar results as those embedded in the box girder where seven complete daily temperature cycles were measured. However, the vertical temperature gradient during the first 2 days is minimal. This is because the heat of hydration of the joint concrete increased the temperature in the joint. The data from the gauges at the end span also measured a temperature increase on the third day after the abutment concrete was placed.

Field-Collected Strain Results

Since the bridge transverse behavior is critical, all the strain gauges were placed to measure the strain in the bridge transverse direction. Figure 6 shows the strain development in the box girder during the first 7 days after joint material placement. In general, all the strains captured from the girders are small in tension (<20 microstrain). This indicated that the daily temperature change did not induce a significant effect on the box girders. The strain collected from the midspan of the box girder (Figure 6*a*)

(a)   (b)   (c)
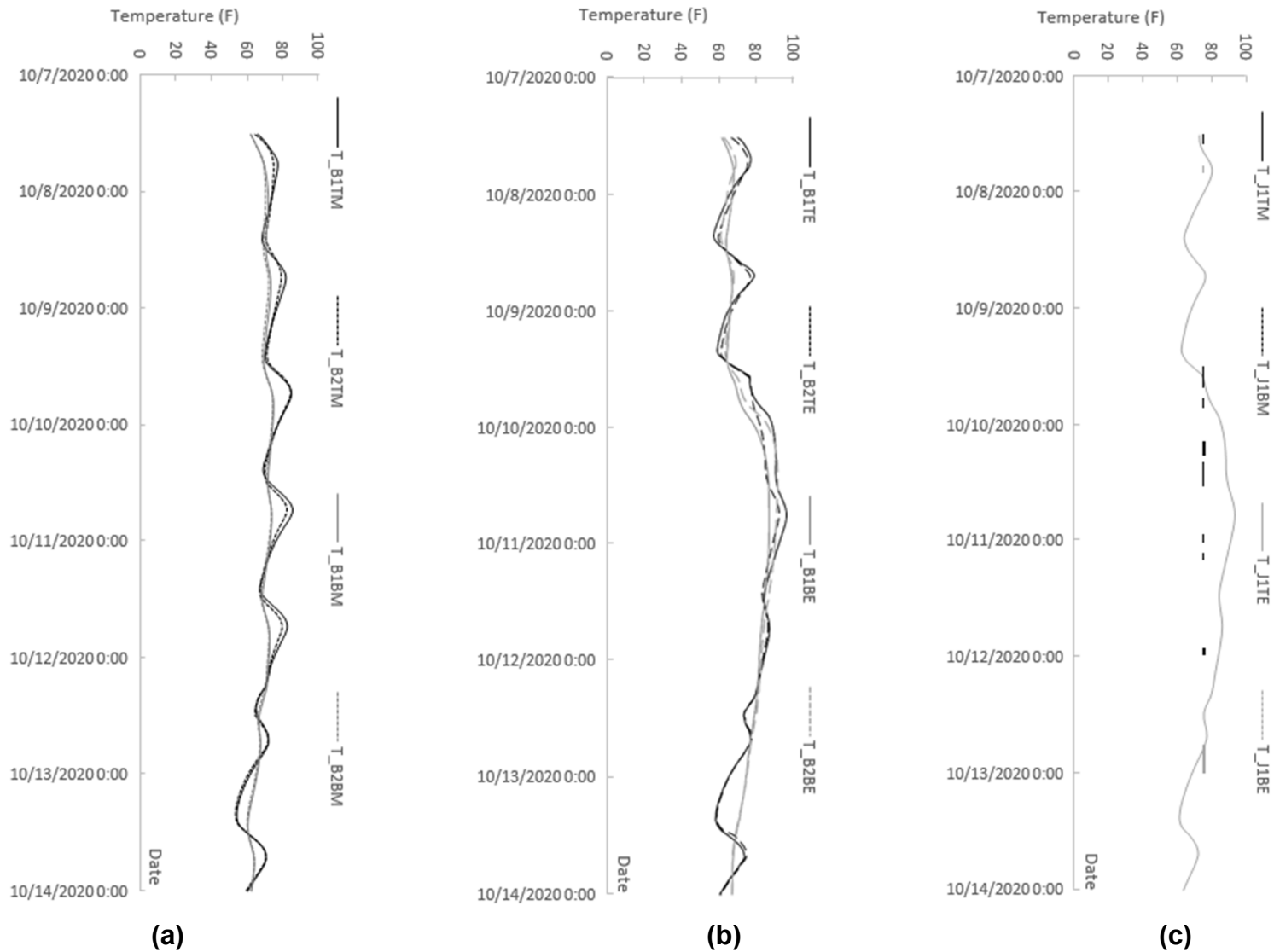
FIGURE 5 Early-age temperature: (*a*) temperature data from gauge B1TM, B2TM, B1BM, and B2BM; (*b*) temperature data from gauge B1TE, B2TE, B1BE, and B2BE; and (*c*) temperature data from gauge J1TM, J1BM, J1TE, and J1BE.
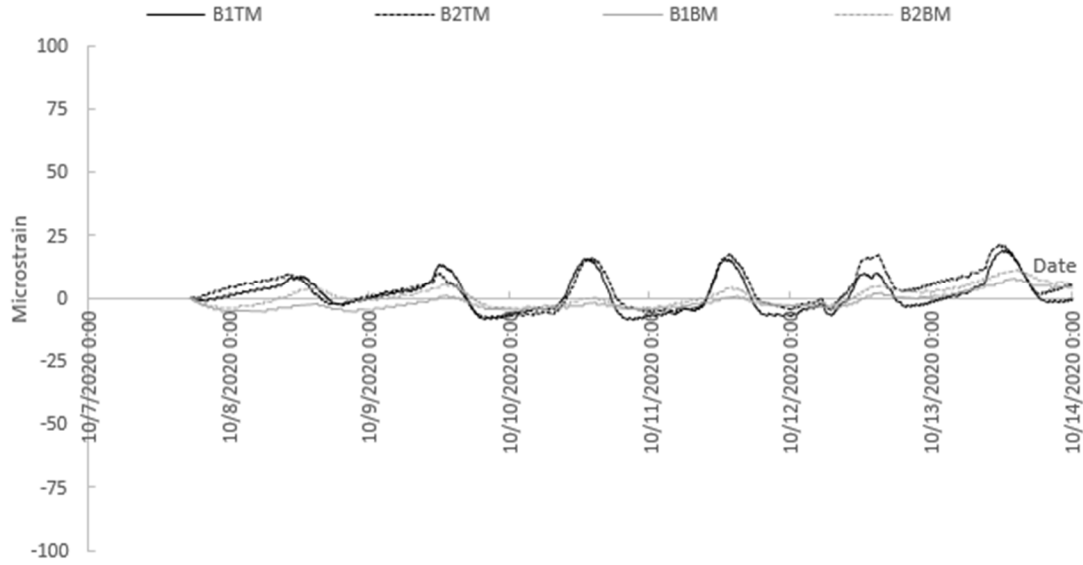
(a)
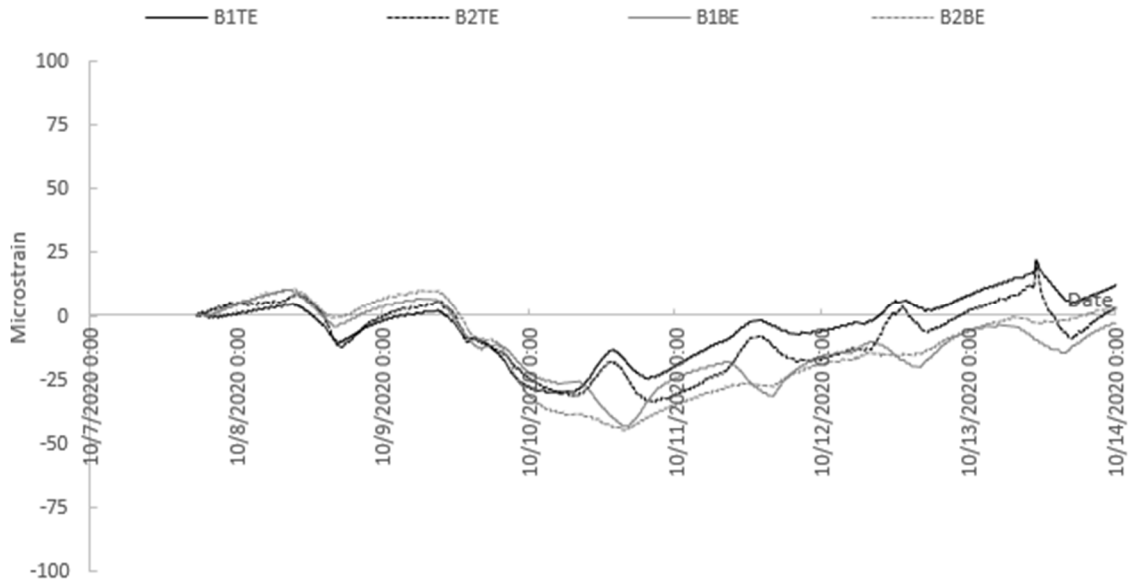
(b)

FIGURE 6 Early-age strain development in the box girder: (*a*) strain data from gauge B1TM, B2TM, B1BM and B2BM and (*b*) strain data from gauge B1TE, B2TE, B1BE and B2BE.

indicated that the girder expanded during the daytime when the temperature on the top surface increased. However, the strain change at the bottom of the girder is minimum. The strain collected from the end span of the girder is presented in Figure 6*b*. It was found that the construction of the abutment shows a positive effect on the beam end behavior and reduces the transverse strains. This is probably because of the shrinkage of the integral abutment during the first few days after the placement of abutment concrete, which provides a transverse restraint to the beam ends.

Figure 7 shows the strain collected from the longitudinal joint. Like the findings from Phares et al. (2017), an expansion of 50 to 300 microstrain was captured during the first 24 h after joint material placement. This expansion was induced by the Type K cement in the joint material and is desired to create a compression-domain joint (Liu and Phares 2019; Liu and Phares 2020; Liu et al. 2020). Like the strains collected from the box girder, the strain measured from the end of the joint was also affected by the construction of the abutment. This effect is also positive and reduces the transverse strain at the end of the joint. Comparing the expansion from Joint 1 and 4, the results indicated that higher expansion occurred in Joint 1. The field monitoring results match the findings from the material property test that most of the expansion of the joint occurs during the first 24 h after placement. In addition, the magnitude of the expansion measured from the material property test (310 microstrain from Joint 1 and 200 microstrain from Joint 4) shows an agreement with the field joint expansion that more expansion occurred in Joint 1.

Inspection Results

The visual inspections were conducted with a focus on the condition and performance of the joint, with a specific interest in the development of any cracking. Figure 8 shows the bridge top surface by October 2020. To date, no cracks have been found in the joint.

Conclusions

The conclusion from this field demonstration were as follows:

- The innovative joint is sufficient to resist the early-age joint longitudinal cracking. Specifically, joint cracks commonly found in other box girder bridges with traditional narrow joints were not seen on the new joint.
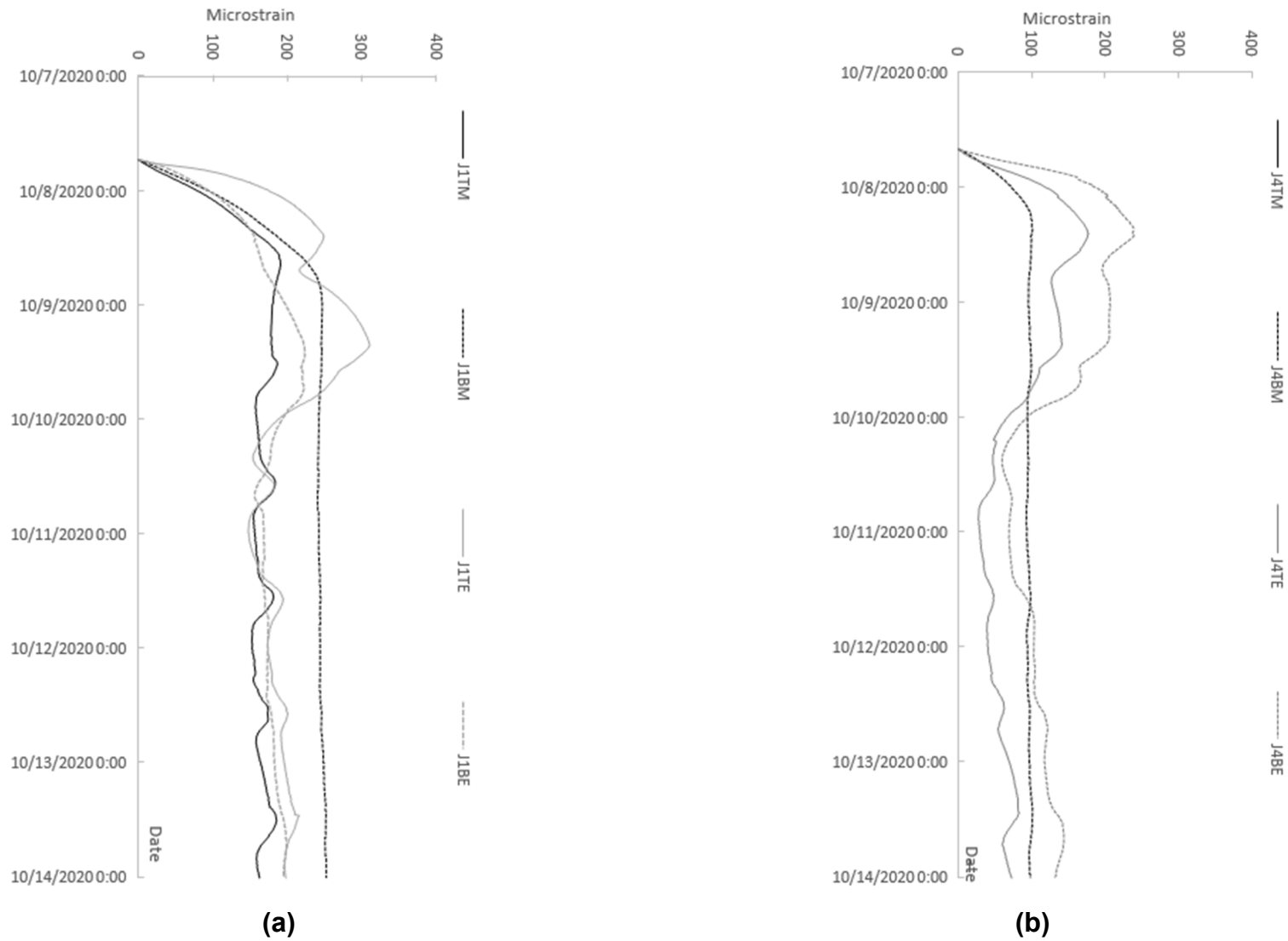
FIGURE 7 Early-age strain development in the joints: (*a*) strain data from gauge J1TM, J1BM, J1TE and J1BE and (*b*) strain data from gauge J4TM, J4BM, J4TE and J4BE.

FIGURE 8 Bridge inspection results.

- With an integral abutment, the construction of the abutment affects the strain distribution in the joint near the joint ends. Fortunately, this tends to provide more restraint in the transverse direction and reduce the chance of joint cracking.

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Local Bridge Construction and Repairs

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Like many counties and communities around the world bridges are a huge issue. Buchanan County Iowa is a rural community of 21,000 people and 372,000 pigs. It has three major rivers that flow through the county and numerous smaller creeks. It has 963 mi of roads and 260 bridges in the secondary roads system. Traditional funding would not even replace one average bridge per year. With 260 bridges this is not realistic to rely on so nontraditional solutions were needed. This presentation covers some of the project solutions that were utilized and can be replicated in other rural areas. It also includes methods to extend the life of both the older bridges and new bridges.

Methodology

The presentation is broken into three basic sections. A key portion is an introduction to the problems and examples of failures. A summary of the *Economics of Bridge Closure from Kansas* is utilized to show the need for replacements (Figure 1). Failures are shown and examples of failures to follow bridge postings are shown. Bridge preservation techniques are addressed to include bridge deck sealing, high-performance deck overlays, an ultra high-performance concrete (UHPC) deck overlay, and epoxy injections. Timber abutment encasements and timber pier encasements are shown. The presentation transfers to bridge replacements and various techniques are addressed in the replacement of bridges. The presentation details the utilization of railroad flatcars for bridges and is presented in a manner intended to provide sufficient information to allow a county to adapt it to their area and construct a railcar bridge (Figure 2). It also addresses the challenges in the construction of the railcars such as inspections. Costs are briefly addressed as are various substructure designs. Here in Buchanan County, we have 32 bridges constructed from railcars. We have been constructing railcar bridges here for over 20 years and have not had any maintenance issues or seen any deterioration. GRS-IBS (fabric) abutments are discussed. The traditional method is utilizing CMU block. Three other options are presented. One of
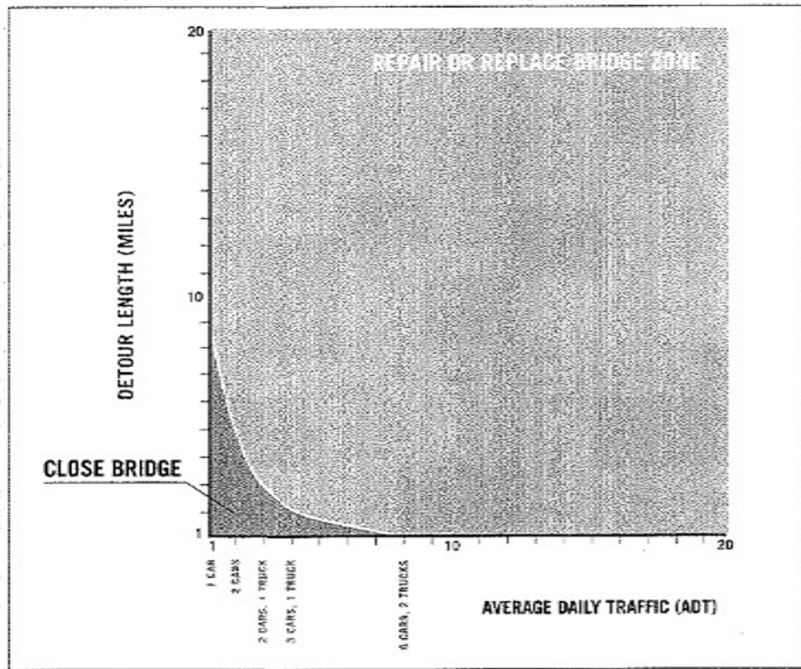
FIGURE 1 *Impact of Closing Low-Volume Rural Bridges* (Kansas DOT).

FIGURE 2 Typical 65-ft railroad flatcar bridge.

the options is to utilize sheetpiling. An additional option is to wrap the fabric back in what is often referred to as a burrito wrap and taper the face on a 2:1 slope and protect it with rip rap. A third method that has been utilized is to stack and taper the wraps on a 1:1 slope and face it with rebar-reinforced roller compacted concrete packed in place with a vibratory plate compactor on a hydraulic excavator. A relatively new technology of driving the H-piling utilizing a vibratory H-piling driver will be introduced. UHPC is discussed to include two UHPC PI beam bridges (Figure 3). The Jakway Park PI beam bridge was the first UHPC PI beam bridge constructed and the second is an international partnership with the Korean Institute of Construction Technology (KICT) who provided materials and Buchanan County employees constructed a 50 ft long UHPC PI beam bridge. We will also present on two cast on site slab bridges with internal curing concrete. The discussion of bridges will shift to to glue laminated timber bridges where we have three modern glue laminated timber bridges. One of these is a 132-ft long three-pin timber arch and the second is a 70- x 40-ft glue laminated bridge which was a partnership with a private firm from South Dakota. The third is a partnership with the US Forest Products Laboratory from Madison, Wisconsin. The presentation shifts to steel bridges where a free design software program was used to construct the 68- x 40-ft Jesup South Bridge using galvanized beams and galvanized rebar. The bridge was constructed on Buchanan County's heaviest traveled paved road
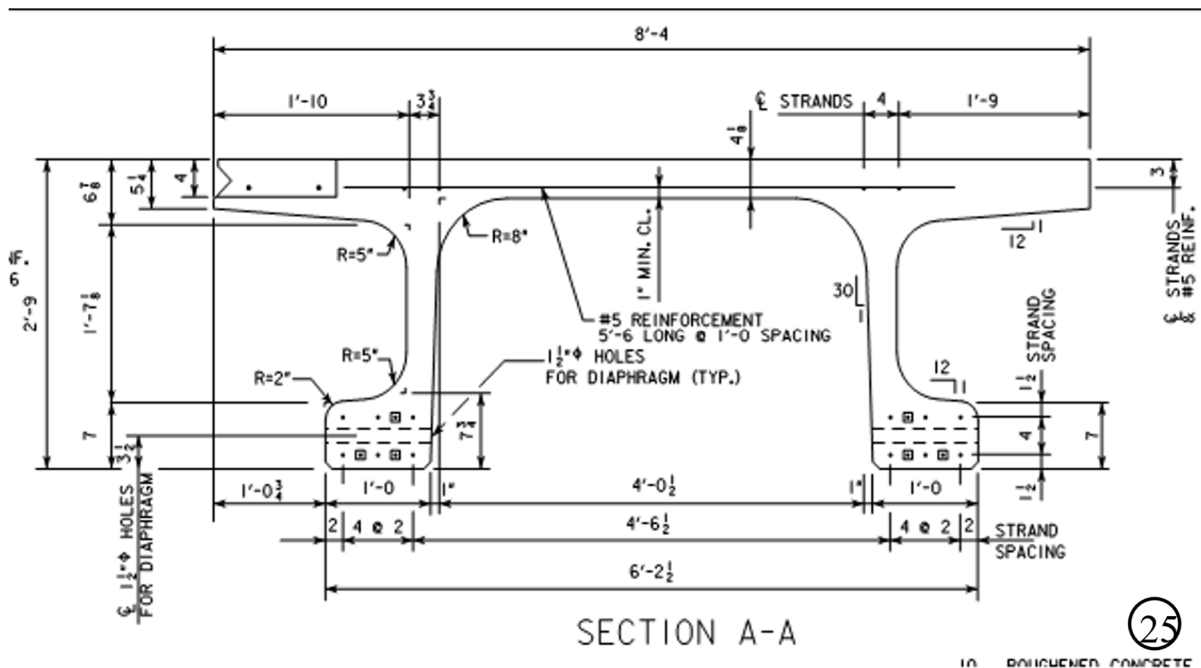
FIGURE 3 PI beam, note 4, and 1/8 in. thick deck.

by the county employees. This is followed by a presentation of a press brake tub girder on GRS-IBS abutments utilizing galvanized sheetpiling (Figure 4). It was an IBRD project and the technology has developed significantly since then and Press Brake Tub girders are now being fabricated in Nebraska. The mass production makes this a very economical and competitive method of bridge construction. We will also discuss buried soil structures. It is a common occurance where residents will ask wht not just remove the bridge and throw in a pipe?

A buried soil structure is basically a multi plate structure without a flowline that has the ability to span 60 ft (Figure 5). Economics can be enhanced by utilizing the old structure piling in some situations. This results in a simple economical structure.
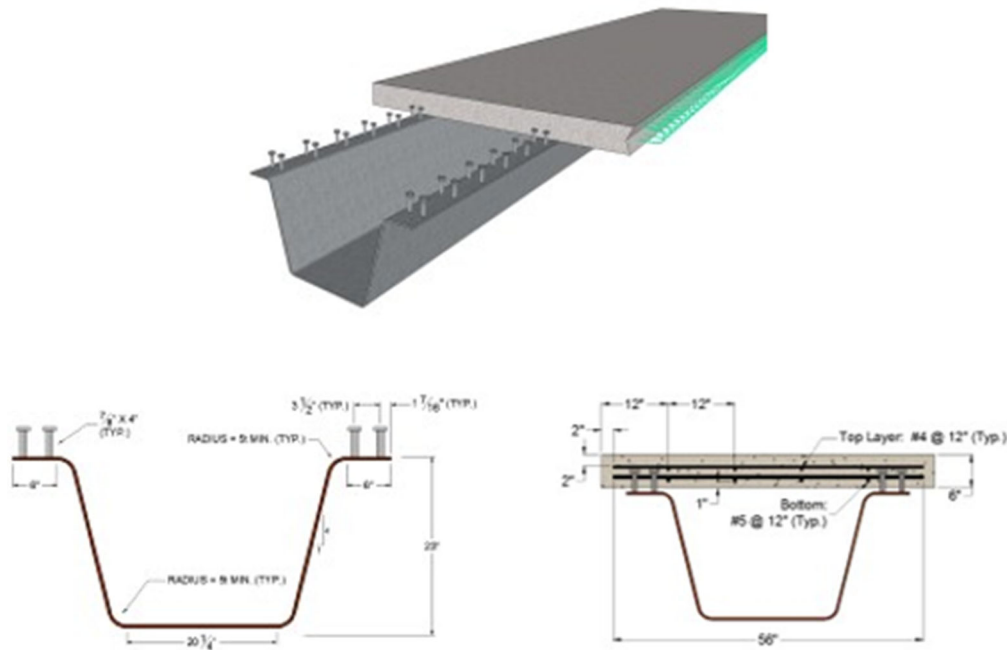
FIGURE 4 Press-brake-formed steel tub girders, galvanized or weathering steel options. Modules are joined using UHPC longitudinal closure pours. Modules can be shipped to site pre-topped or with a variety of deck options.

FIGURE 5 Buried soil structures.

This is followed by a three-span galvanized steel beam bridge where the H-piling were galvanized and painted. The desire to design this manner resulted from the need to repair the encasements of the piling in the bridge downstream that had been constructed about 20 years prior to this bridge and required repairs to the piling encasements. Using the galvanized and painting eliminated the need to construct coffer dams. It turned into a major method of accelerated bridge construction as three 50-year storms occurred during construction. It would not have been possible to construct coffer dams during that timeframe. Extending the road closure into the spring would have been disasterous.

Subsequent research in the accelerated corrosion chamber at Iowa State University has indicated that the galvanizing of the H-piling will produce a product to last over 100 years. The paint coating will extend this life. The search for economical bridge replacements is on going and as technology evolves so will the solutions. We recently constructed a bridge using Grade 65 steel. It is 30% stronger than Grade 50 steel and the price increase is anticipated to be 5% more. This allows for lighter beams with shallower sections. Both of those are often concerns for local entities. I have since learned more about metal decking beams and regret that I had not utilized them on my Grade 65 beam bridge. I plan to construct a bridge in the future using decking beams.

Findings

Economics dictates that closing many bridges is not an economical option. Funding does not exist to replace everything we have so we must make what we have last longer. Preservation concepts such as concrete sealing will make bridges last longer. Encasements in concrete can do that and recent research is very promising on utilizing UHPC for major repair Sections. Standard construction is good but economics dictates that we must search for more economical solutions where we can impliment them. There are more economical replacement answers that can last very long time. Technologies such as vibratory H-piling drivers are far safer, faster, and more economical. Specifications need to be modified to accommodate these types of new technologies.

Conclusion

Many of the bridge designs were developed when labor was cheap and materials were expensive. That paradigm has changed, labor is a commodity in short supply, and designs that reduce the manual labor even at the expense of materials must be considered. Economics dictates in the past and now. In many rural environments prefabricated/precast units should be considered. Preservation of our existing system becomes even more critical as we subject our structures to de-icying agents and even heavier loads. Change is inevitable and we must be prepared to accept new concepts and adapt to them.

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Eric J. Fitzsimmons, PhD. Lecture University of Kansas.

Steven d. Schrock, University of Kansas.

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Development of a Bridge Railing Optimized for Rural Low-Volume Roads

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A new steel bridge rail was developed for use on rural low-volume bridges. The railing consisted of a 31-in. tall, 12-gauge W-beam guardrail mounted on S3x5.7 posts, which were supported by steel square-tube sockets. These side-mounted sockets were attached to the deck edge using a unique bolted design that connects directly to coupling nuts and threaded anchor rods embedded into the bridge deck. Thus, during a crash, the tensile impact loads are transferred directly to the anchor rods, and the risk of damage to the deck edge is minimized.

Full-scale crash testing was conducted according to test 2-11 of the American Association of State Highway Transportation Officials *Manual for Assessing Safety Hardware* (MASH). The test vehicle impacted the bridge rail at 44.2 mph and an angle of 25.5 degrees and was successfully contained and redirected. Damage to the bridge rail consisted of bent posts and deformed guardrail. No damage to the deck or sockets was observed. The tests passed all evaluation criteria of MASH test 2-11. The new railing was deemed MASH Test Level 2 (TL-2) crashworthy with a post spacing of 75 in. and MASH TL-3 crashworthy with a post spacing of 37.5 in. BARRIER VII simulations showed that the new railing could be directly connected to the Midwest Guardrail System without a transition. Guidance was provided pertaining to the length of guardrail required adjacent to the bridge rail.

Expanding the Applicability of Press-Brake-Formed Tub Girders Through Improved Live Load Distribution Factor Equations

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The scope of this study is to develop improved live load distribution factor (LLDF) prediction equations for moment in interior girders of press-brake-formed tub girder (PBFTG) bridges. PBFTGs consist of cold-bent standard plate widths and thicknesses to form a trapezoidal box girder. Several restrictions limit the use of the American Association of State Highway and Transportation Officials Load Resistance and Factor Design Bridge Design Specifications (AASHTO LRFD BDS) LLDFs for bridges containing multiple tub girders. These restriction sections are based on the studies by Johnston and Mattock (1967). The researchers developed analytical tools to calculate LLDFs on 24 simple, straight box girder bridges. The tools were benchmarked against experimental testing on a one-quarter scale model of an 80-foot span composite steel-concrete box girder. The study was limited and restrictive due to a lack of variation in parameters to determine the LLDFs. Based on a survey of literature and field testing performed on PBFTG bridges across multiple states in the United States (Gibbs 2017 and Roh 2020), the empirical equations found in the AASHTO LRFD BDS have been found to be conservatively applicable to shallow steel tub girder bridges. Improvements can be made to AASHTO LRFD BDS provisions relating to shallow steel tub girders and their respective live load distribution factors (LLDFs).

To develop improved LLDFs, a finite element modeling tool was benchmarked against live load field test data from PBFTG bridges. Then, a sensitivity matrix was developed to determine the influence of specific parameters on live load distribution in PBFTG

bridges, and later a parametric study was performed to develop empirical equations for PBFTG LLDFs to be used with simplified line girder analysis. This study resulted in improved LLDF prediction empirical equations based on 17,971 bridges. The study demonstrated that these developed equations significantly improve the behavior of PBFTGs from a statistical perspective and are recommended for adoption in AASHTO chapter four. The simplified equations will be used with line girder analysis to simplify the design of shallow steel tub girder bridges and expand their application in the short-span bridge market in off-access bridges.

Methodology

Finite element analysis was conducted in this study using the commercial finite element software package Abaqus/CAE (Dassault Systèmes, 2020). The S4R shell element was used, which is a suitable element type for this analysis, as was shown by several researchers (Barth 1996; Righman 2005; Roberts 2004; Yang 2004). All materials were only modeled as linear, elastic, and isotropic mediums. This conclusion has also been made by other researchers. Eom and Nowak (2001) concluded, after testing 17 steel I-girder bridges in Michigan, that the observed response of these bridges under the application of live load was linear throughout their study. In addition, Barth et al. (2018, Vol. 4) determined that the in-service response of press-brake-formed steel tub girders in composite bridges was linear. The authors used guidelines to be adhered to with modeling beam- slab bridges from AASHTO LRFD BDS.

A total of 156 bridges were modeled in the sensitivity study to determine the effect of certain parameters on live load distribution in PBFTG bridges. The two most comprehensive studies, the NCHRP 12-26 (1991) and Tarhini and Frederick (1992), were used as a basis for the sensitivity study performed on PBFTG bridges. An important part in the development of simplified methods is the range of applicability. To ensure that common values of various parameters were considered, four common bridges were used as the basis for the matrix of bridges to be analyzed in the design study. The four standard bridges are described in Table 1. The results of the sensitivity study were qualitatively assessed to determine the effects of different parameters on LLDFs. Comparison diagrams were generated for each bridge in the study matrix and changes in LLDFs associated with changes in the potentially influential factors earlier identified were assessed individually.

TABLE 1 Sensitivity Study Average Bridge Dimensions

Bridge Number	Plate Size (in.)	Span Length (ft)	Number of Girders	Girder Spacing (ft)	Deck Thickness (in.)	Overhang Width (ft)
1	72 x 1/2	30	3	6	8	2.22
2	84 x 1/2	40	4	8	8	2.11
3	96 x 1/2	60	5	8	8	1.80
4	120x 5/8	80	6	10	8	1.29

For reference, two sample diagrams are included herein (see Figure 1). The figure shows LLDFs from the analytical modeling performed when spacing and overhang distance were varied in Figure 1*a*, and Figure 1*b* respectively. Generally, as found in previous studies, the LLDFs obtained from the analytical study were noticeably lower than those found using the empirical equations from the AASHTO LRFD BDS. It should be noted that many PBFTG bridges in this sensitivity study were outside of the requirements outlined in AASHTO LRFD BDS Articles 4.6.2.2 and 6.11.2.3. The sensitivity study concluded that spacing, span length, and girder size had a strong effect, and the number of beams and deck thickness had a moderate effect. The overhang distance had a negligible effect. The efforts of the sensitivity study were imperative in formulating the parametric study.

For the parametric study, a matrix of bridges was analyzed using Abaqus/CAE to determine the live load moment in the interior girders of PBFTG bridges. This matrix was developed based on the insights the sensitivity analysis provided, aiming to explore the parameters found associated with PBFTG live load distribution. The parametric study expanded the number of bridges, which is crucial because a larger sample size

(a)   (b)
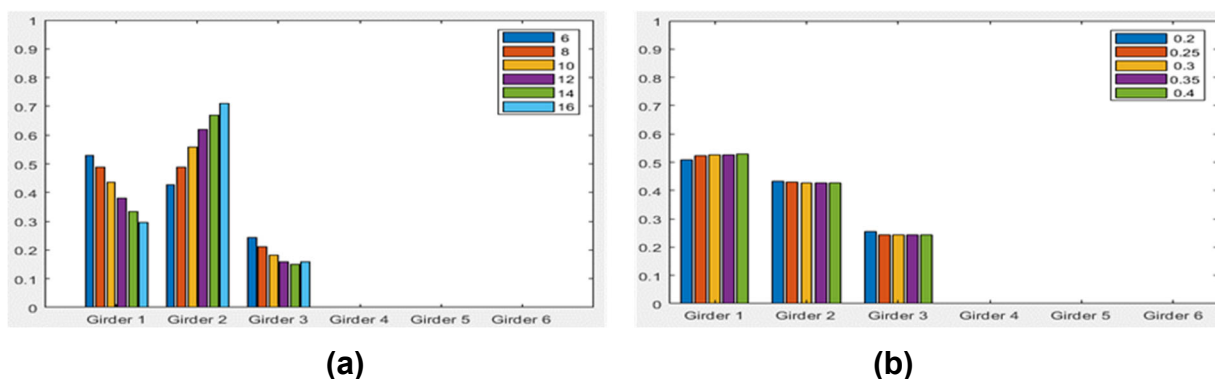
**FIGURE 1 Effect of (*a*) spacing and
(*b*) overhang distance on LLDFs in an interior girder (Girder 2).**

can help increase the precision and accuracy of prediction using empirical equations. With more data, patterns and trends in the data can be identified more efficiently, with more confidence and higher statistical power. The matrix of bridges for the parametric study consisted of 17,971 bridges and assessed the effects of a focused set of key parameters on PBFTG LLDFs. The data contained 9,800 bridges in the one-lane loaded scenario and 8,171 bridges in the two-lane loaded scenario. Not all the bridges in the one-lane loaded scenario can fit two design trucks; thus, there are fewer bridges in the two-lane loaded scenario.

Findings

This study developed simplified empirical equations by modeling the LLDFs obtained from the parametric study for the 9,800 and 8,168 one-lane and two-lane loaded PBFTG bridges, respectively. An initial basic model was first estimated. The authors followed the recommendations of the NCHRP Project 12-26 (1991) power model representation of LLDFs to identify a suitable equation function structure and used the insights of the sensitivity analysis to identify potential independent variables. A series of potential models were developed, and the final models were selected using the following criteria: The final models should maximize the goodness of fit and predictability, ensure unbiased estimators, and provide a simplified empirical equation that can be used in practice to predict live load distributions. The parameters of these models were estimated using Ordinary Least Squares (OLS).

Following the determination of the empirical equation, all coefficients were rounded to an acceptable number of significant figures, according to the industry practices. The OLS coefficient estimation technique minimizes the sum of the squares of the differences between the actual LLDFs and the predicted LLDFs, aiming to estimate the regression equation that best predicts the actual LLDFs. Therefore, it is expected that the OLS regression will roughly underestimate half and overestimate the other half of the LLDFs.

The estimated equations were multiplied by a modification factor to ensure that the proposed equations would result in conservative predictions. The modification factor was estimated with the goal of accurately predicting or overpredicting 100% of the LLDFs, using a 95% prediction interval (PI) for the individual predicted LLDFs based on the existing bridge sample. This prediction interval corresponds to approximately 98% confidence interval (CI) for the average expected LLDFs based on the sample. After analyzing the results of multiple combinations of parameters, Equations 1 through 4 are proposed to calculate LLDFs for PBFTG bridges.

For interior girders with one-lane loaded:

$$DF_{L>40} = 0.68 \frac{S^{0.43}}{L^{0.33}} \left(\frac{I^{0.7}}{N_b t_s^2} \right)^{0.1} \tag{1}$$

$$DF_{L\leq40} = 1.06 \times DF_{L>40} \tag{2}$$

$$DF_{N_b<6} = 0.685 \frac{S^{0.43}}{L^{0.17}} \left(\frac{I^{0.4}}{N_b^3 t_s} \right)^{0.1} \tag{3}$$

$$DF_{N_b\geq6} = 0.495 \frac{S^{0.56}}{L^{0.26}} \left(\frac{I}{N_b t_s^3} \right)^{0.04} \tag{4}$$

where *DF* is live load distribution for the corresponding girder and number of loaded lanes, *S* is girder spacing, *L* is the span length, *I* is the moment of inertia, N_b is the number of girders, and t_s is the deck thickness.

By construction, all empirical equations overestimate the true live load distribution factor obtained from FEA. However, as Figure 2 shows, the predictions of the proposed
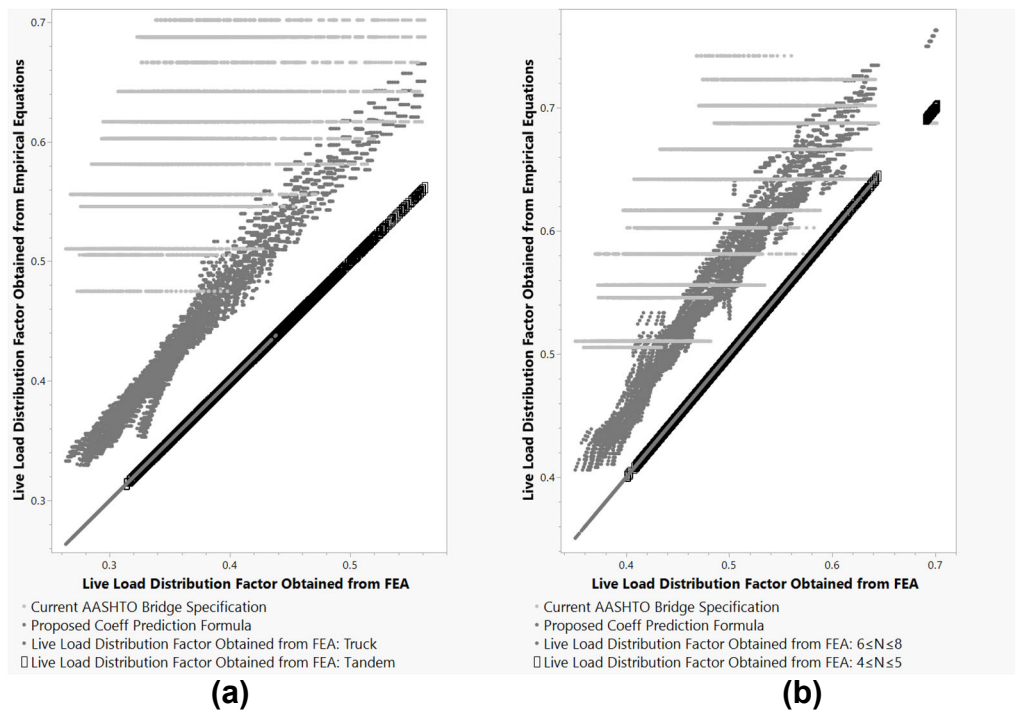
FIGURE 2 Q-Q plot of (*a*) one-lane loaded and (*b*) two-lane loaded LLDFs.

equations approximate the true live load distribution factor obtained from FEA far more accurately than AASHTO LRFD BDS provisions. Furthermore, the prediction errors (i.e., the differences between the FEA and the predicted values) are less dispersed (scattered).

Conclusion

Besides a higher accuracy and precision in predicting LLDFs, the proposed equations have a known statistical certainty, unlike the AASHTO LRFS BDS equation. Specifically, the proposed equations were estimated based on a 95% PI (and approximately 98% CI). Therefore, it is expected that, given enough samples, future LLDF for bridges that have characteristics within the range of the characteristics of the bridge sample used for this analysis will be accurately estimated or overestimated 95% of the time with the proposed equations (with a 98% or higher probability that the predicted LLDF will overestimate the actual LLDF obtained using FEA for the average LLDF).

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Vulnerable Users of Low-Volume Roads

Register's Annual Great Bicycle Ride Across Iowa and Related Low-Volume Road Management Perspectives

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RAGBRAI (The Register's Annual Great Bicycle Ride Across Iowa) is an eight-day bicycle ride across the state of Iowa. The event has been held each summer since 1973 (though no ride was held during 2020 due to the coronavirus pandemic). With registration generally capped at 10,000 participants, it is the largest multi-day bicycle touring event in the world. While the touring route changes from year to year, most of the cross-state bicycle tour is typically routed along low-volume roads. The 50th edition of the event will be moving its way eastbound across Iowa during the week of the 13th International Conference on Low-Volume Roads. The exact riding route was announced in May 2023.

RAGBRAI event organizers work closely each year with local and state agencies to plan for the event, identify the tour route, select host cities, and coordinate registration and event logistics to increase the likelihood that the event will run smoothly. Many of these partnerships among organizers and agencies have existed for decades, and both the scope and longevity of this annual event provides an opportunity for conference attendees to examine RAGBRAI as a case study of planning, operations, and maintenance associated with a very large "rolling event" that takes place predominantly on a low-volume road network.

Through interviews and discussions conducted with stakeholders from across the state of Iowa, this submittal will synthesize both the planning activities that event organizers and roads managers engage in during RAGBRAI planning, as well as the active event management processes in place along low-volume roads that occur during the event itself. The submittal will also identify both lessons learned and continuing challenges with organizing the event throughout its 50-year history, as stakeholders seek to preserve a safe and enjoyable experience for large numbers of vulnerable road users without significantly impacting accessibility for residents that need to maintain their own access to the road network.

Event Pre-Planning Among Organizers and Low-Volume Roads Managers

RAGBRAI event organizers work with road maintainers at the state, regional, and local level to identify each year's riding route. The cross-state route means that many partners are involved, and these partners must coordinate efforts both internally and with one another in preparation for the event. These coordination activities impact road construction and maintenance plans and priorities, and they also provide an opportunity for those who maintain low-volume roads across the state of Iowa to form lasting partnerships. These partnerships can improve the exchange of information and best practices in support of regional management of low-volume roads as transportation assets.

Event Management and Low-Volume Road Impacts

Each partner managing low volume roads along the RAGBRAI route must consider event management aspects such as detour planning, ingress and egress at key points along the route, routing for event support vehicles (including heavy vehicles), emergency management and public safety planning, and post-event maintenance and clean-up. Event organizers have established an approach to supporting road managers and local law enforcement through this process, and these established best practices could have broad applicability to other rolling events that occur on low-volume roads elsewhere around the world.

Mitigation of the Damage of Amish Buggies to Local Roads

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This paper summarizes the results of a study that was conducted to identify all possible changes that could be made to the horseshoes worn by the horses that Amish people use to pull their buggies to mitigate their damage to local roads. To this end, several alternative horseshoes to reduce damage to roads were identified, which included a horseshoe with a new calk design, a horseshoe coated with select tungsten carbide coating design, and a composite horseshoe. Laboratory tests were performed to quantify the reduction in local roads damage due to using alternative horseshoes. In addition, experiments were conducted to evaluate the effects of the new horseshoe alternatives on horse hoof and legs. Finally, comprehensive cost analyses were conducted to estimate the users costs as well as the Amish buggy routes life-cycle costs when the different horseshoes alternatives considered in this paper are used. The results of the lab tests indicated that the use of alternative horseshoes can significantly reduce road damage. The experiments performed to evaluate the effect of alternative horseshoes on horse hooves indicated that the alternative horseshoes reduced the stresses on the horse hooves by at least 45%, while maintaining traction. The results of life-cycle cost analyses indicated that using the alternative horseshoes with new calk design can result in reducing the annual repair costs of Amish buggy routes by at least 40%. In addition, the results suggested that using the identified horseshoes reduce the annual horse keeping costs encountered by Amish people when considering the effects of those shoes on the horse service life.

CLIMATE RESILIENCY

Infrastructure Adaptation and Climate Resilience for California's National Forests

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This paper describes the road infrastructure found on California's National Forests, their vulnerabilities, and specific measures that can be taken to adapt to projected climate change effects, thus minimizing damage from fires and storms. Over the past 40 years this region has been hit by numerous climate change-related events, including droughts, major forest fires, major storms, and flooding. Billions of dollars in damage have been sustained and numerous lives lost. Assessing vulnerabilities, ranking resources at risk, and prioritizing adaptation actions are needed.

The US Forest Service has recently been involved in infrastructure vulnerability assessment and adaptation strategy projects involving climate model studies, interviews, literature review, local workshops, website information, and publication of the project findings. Different agency vulnerability assessment methods have been reviewed to establish a functional assessment and risk analysis methodology. Efforts to mitigate the impacts of climate change included greenhouse gas reduction from agency vehicles, evaluating alternative transportation routes, implementing energy-saving measures, and identifying "stormproofing" road design measures to reduce the vulnerability of roads to extreme climate-related events.

Much of the effort has been the identification of road adaptation and resiliency measures, particularly measures that are practical and implementable with a minimum of cost. These measures include routine road maintenance, relocating road segments as needed, adding trash racks and diversion prevention dips to prevent culvert failures, building stream simulation projects, protecting bridges from debris and scour, covering soil with deep-rooted vegetation, and using soil bioengineering stabilization and deep patch shoulder reinforcement to prevent local slope failures.

Considering Sustainability for Rural Bridge Design

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When bridge owners select a bridge type for a bridge need, the historical decision criteria has typically been based on first costs for installing the bridge. Responsible owners may also consider life-cycle costs over the bridge service life. Neither of these consider sustainability benefits of one bridge over another. Sustainable design is predicated on the idea that society is willing to pay extra for reducing harmful effects on the environment. The objective of this study (Barker et al, 2022) is to evaluate the life cycle sustainability (cradle to grave) of two nearly identical, functionally equivalent steel and concrete rural bridges. Both bridges are short simple span and county crew built in Whitman County, Washington.

The prefabricated Seltice-Warner steel bridge (Figure 1) was built in 2020 consisting of seven rolled beam girders and a corrugated metal deck for a gravel riding surface. The bridge is 35 ft-8 in long and 28 ft wide. The prefabricated Thornton Depot concrete bridge (Figure 2) was built in 2019 consisting of eight precast prestressed rectangular girders. The bridge is 34 ft long and 32 ft wide.

FIGURE 1 Seltice-Warner steel bridge.

FIGURE 2 Thornton Depot concrete bridge.

Only the superstructure of the bridges is considered in this analysis for a direct comparison of rural steel and concrete bridges. Lifetime sustainability considers sustainability benchmarks at each phase in the bridge life cycle, including acquisition of the materials, the manufacturing process of bridge components, construction, maintenance, and demolition at the end of the life cycle.

A second objective of the work is to apply the sustainability results to develop procedures where the owner or society can consider sustainability benefits in the design of bridges. A simple decision-making procedure is presented that considers monetized sustainability benefits for any number of bridge alternatives associated with a bridge project. The procedure is framed as reductions in emissions or reduction in energy consumed for every extra dollar spent.

Methodology

To assess quantitatively the sustainability of rural steel and concrete bridges, four sustainability criteria are applied to the study bridges: embodied carbon emissions, which will be referred to simply as emissions herein, energy consumption, waste management and recyclability, and life cycle costs. The study considers sustainability benchmarks and costs over three phases of bridge life: construction, maintenance, and demolition. Whitman County personnel supplied data on costs, construction information and estimations for yearly maintenance and demolition. Whitman County sources were

considered for recycling and landfill information and national average recycling rates were applied to both the steel and concrete materials.

Information on specific component emissions and embodied energy is given in product specific Environmental Product Declarations (EPDs). These EPDs are generated by companies or industries and externally reviewed for accuracy. The emissions and energy consumed for equipment usage were based on the amount of greenhouse gas produced every hour the equipment was in use.

To determine the total emissions and energy consumption for the Seltice-Warner steel bridge and the Thornton Depot concrete bridge, the analysis is divided into the sub-totals for the superstructure (prefabricated bridge and additional materials), construction equipment, maintenance equipment and demolition equipment.

At the end of the lifespan of the bridge, it will be dismantled and discarded. The volume of material that will be recycled or dumped in a landfill is considered for each bridge. It is assumed that 98% of the steel used in a project is recycled at the end of its life and that 80% of concrete is recycled.

The life cycle costs of the two bridges are broken into two categories: initial costs and the present value of future costs using an appropriate discount rate of 1.7%. Initial costs include the cost of the prefabricated superstructure and the construction labor, materials, and equipment costs. Future costs include maintenance and demolition costs.

Findings

The total emissions and energy consumption for the superstructure components and equipment used during construction, maintenance, and demolition of the Seltice-Warner and Thornton Depot bridges are shown in Table 1. The emissions produced by the Seltice-Warner bridge over its 75-year lifetime totaled 47,284 kg of CO_{2e} and the Thornton Depot bridge totaled 59,726 kg. The concrete bridge will produce 26.3% more CO_{2e} emissions (2.3 passenger car years or 31269 mi) over its lifetime than the steel bridge. The energy consumed by the Seltice-Warner bridge over its lifetime totaled 667,459 MJ and the Thornton Depot bridge totaled 725,780 MJ. The concrete bridge will have consumed 8.7% more energy (0.6 home years) than the steel bridge over their equivalent 75-year lifespans.

The total life-cycle cost of each bridge is the sum of the present value initial, maintenance, and demolition costs. Table 1 displays the present value costs and the life-cycle costs. The life-cycle cost for the Seltice-Warner bridge over its 75-year

TABLE 1 Emissions, Energy Consumed, and Life-Cycle Cost Results

Emissions (kgCO$_{2e}$)					
	Superstructure	**Construction**	**Maintenance**	**Demolition**	**Total**
Steel	23554	4370	18270	1091	47284
Concrete	34759	4768	18270	1929	59726
Energy Consumption (MJ)					
	Superstructure	**Construction**	**Maintenance**	**Demolition**	**Total**
Steel	328683	62379	260820	15577	667459
Concrete	369355	68074	260820	27531	725780
Life-Cycle Cost					
	Superstructure	**Tot Initial**	**PV Maintenance**	**PV Demolition**	**Total LCC**
Steel	$57,324	$77,657	$16,485	$1,302	$95,445
Concrete	$73,569	$96,845	$16,485	$4,320	$117,650

lifespan is $95,445 while the life-cycle cost of the Thornton Depot bridge over an equivalent lifespan is $117,650. The Seltice-Warner bridge has $22,205 less life-cycle cost.

If an owner was considering these two bridges for a bridge need, the steel bridge has both higher sustainability benefits and lower cost, so it is clearly the best option and the decision of which bridge to build is trivial for these two bridges. When the decision is not trivial, for instance if the steel bridge had higher life cycle costs, to consider sustainability in the decision of which bridge should be built, the question to answer is, "what additional cost would society or the owner be willing to pay to increase sustainability benefits?"

If society or the owner determines an acceptable cost for reducing emissions, energy consumption and material going to the landfill, an equivalent cost can be considered in the decision-making process. For instance, if society or the owner is willing to pay $0.20 for every 1 kg of CO$_{2e}$ reduced and $0.04 for every 1 MJ of energy reduced, then the owner can consider a monetary benefit cost of $0.20 × 12,442 = $2,488.40 for reducing emissions by 12,442 kg CO$_{2e}$ and $0.04 × 58,321 = $2,332.84 for reducing energy consumption by 58,321 MJ as a benefit for the steel bridge.

A similar method can be applied to landfill use. If society or the owner is willing to pay $50 as a societal benefit for every ton not sent to the landfill, then the owner can consider a monetary benefit cost of $50 × 20 = $1,000 for 20 tons less to the landfill as a benefit to the steel bridge.

The benefit to the steel bridge, a total of $5,821, can be subtracted from the steel bridge cost and compared to the cost of the concrete bridge. For this comparison, the steel bridge could cost $5,821 more than the concrete for the two bridges to be equivalent considering the monetary sustainability benefits.

This process can be extended for consideration of any number of alternatives with differing initial costs or life cycle costs for a bridge project. An equivalent cost, that considers sustainability benefits, can be determined for each alternative and compared for lowest equivalent cost. Using the lowest initial cost (or lowest life-cycle cost) alternative as the basis, an equivalent cost can be determined:

Equivalent Cost =   [Initial or Life-Cycle Cost]
        –   [Reduced kg CO_{2e} from Base CO_{2e}] × (Accepted \$/kg CO_{2e})
        –   [Reduced MJ from Base MJ] × (Accepted \$/MJ)
        –   [Reduced Landfill tons from Base Landfill tons] × (Accepted \$/ton)

Table 2 demonstrates the procedure for five alternatives with differing initial costs, emissions, energy consumption and landfill use. The analysis can be used for initial costs and initial sustainability characteristics or life-cycle costs and life-cycle sustainability characteristics as preferred by the owner. If the earlier accepted sustainability costs are applied (\$0.20/kg CO_{2e}, \$0.04/MJ, and \$50/ton landfill use), the equivalent costs can be compared to select the optimal bridge.

Alternative 1 has the lowest initial (or life-cycle) cost and would be selected based on a lowest first (or life-cycle) cost criterion. Alternative 1 is the basis for the sustainability cost benefits (zero total cost benefit) so the equivalent cost is the initial (or life-cycle) cost.

With the societal accepted costs for emissions, energy consumption, and landfill use, Alternative 4 has the highest additional sustainability benefits of \$5,956, but it also has a higher initial cost of \$7,000 (incremental benefit–cost ratio less than 1.0) and the sustainability benefits do not overcome the higher initial cost compared to Alternative 1.

Alternative 3 costs \$5,000 more than Alternative 1, but it returns \$5,821 worth of additional sustainability benefits for an equivalent cost of \$99,179. Considering sustainability in bridge selection criteria, Alternative 3 should be chosen over Alternative 1 even though it is more expensive. The initial cost of Alternative 3 is \$5,000 more than Alternative 1, but the societal sustainability benefit is \$5,821 (incremental benefit–cost ratio greater than 1.0). According to society's acceptable costs for reducing emissions, energy consumption and landfill use, although Alternative 3 costs more than Alternative 1, society is reaping a \$5821 return on the \$5000 extra cost investment for Alternative 3.

This procedure is the same as an incremental benefit–cost analysis. However, most owners are not familiar with incremental benefit–cost analysis and the equivalent cost

TABLE 2 Equivalent Cost Comparison Considering Sustainability

Bridge	Initial or Life-Cycle Cost	Initial or Life-Cycle Total kg CO$_{2e}$ MJ Consumed Landfill (tons)			Reduction kg CO$_{2e}$ MJ Consumed Landfill (tons)			Cost Benefit kg CO$_{2e}$ MJ Consumed Landfill (tons)			Total Cost Benefit	Equivalent Cost
Alt 1	$100,000	59726	725780	21	0	0	0	$0	$0	$0	$0	**$100,000**
Alt 2	$105,000	70000	720000	10	−10274	5780	11	−$2,055	$231	$540	−$1,284	**$106,284**
Alt 3	$105,000	47284	667459	1	12442	58321	20	$2,488	$2,333	$1,000	$5,821	**$99,179**
Alt 4	$107,000	45000	664000	10	14726	61780	11	$2,945	$2,471	$540	$5,956	**$101,044**
Alt 5	$107,000	44000	750000	1	15726	-24220	20	$3,145	−$969	$1,000	$3,176	**$103,824**

procedure developed here is directly comparable to historical first cost comparisons or life-cycle cost comparisons and would be better understood by owners.

Conclusion

Sustainability is becoming an important consideration for bridge design. Presented is a simple decision-making process that considers monetized sustainability benefits for any number of bridge alternatives associated with a bridge project. The owner or society determines the acceptable additional costs they are willing to pay for reducing emissions, reducing energy consumption, or reducing material sent to the landfill. The initial cost, or life-cycle cost, is adjusted for the monetary sustainability benefits associated with reducing harmful environmental impacts to determine an equivalent cost that can be compared among alternatives to select the best alternative. The procedure is flexible in that it can be applied to the initial or life-cycle bridge and costs, it can consider maintenance and demolition or ignore either or both, it can be applied for any combination of emissions, energy consumption and landfill use, or other sustainability metric. The alternative bridge that is built can be decided based on consideration of acceptable additional costs that gains desirable sustainability benefits.

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Road Stream Crossings for Storm Resiliency and Aquatic Organism Passage
A Pennsylvania Perspective

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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 convey. These undersized crossings, coupled with increasing storm magnitude and more frequent flooding events due to climate change has led to significant infrastructure damage and economic impact in Pennsylvania (*3*, *4*). In addition, undersized crossings can impact water quality due to channel and bank erosion, and impact aquatic health through lack of channel continuity and blockage of aquatic organism passage.

Pennsylvania's Dirt, Gravel, and Low Volume Road Maintenance Program (Program), run through the State Conservation Commission (SCC), provides $28 million annually in funding to local municipalities to implement environmental improvements on unpaved and low-volume paved roads [<500 average daily traffic (ADT)]. Starting with a funding increase in 2014, the Program has increased its focus on stream crossing replacements, now funding about 100 annually. Through the Program's quality assurance/quality control (QA/QC) process and a Trout Unlimited (TU) study it was determined that many of the projects were not meeting the goals of the Program, including aquatic organism passage, proper installation, and stream channel continuity. To address these concerns, an effort to define comprehensive standards and guidance was undertaken in 2021 leading to the Program's new Stream Crossing Design and Installation Standard and accompanying technical manual (*5*, *6*). This abstract summarizes the recent efforts of the Program from 2019 through 2022 and the development of the Standard and technical manual. 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

Since the inception of the Program 25 years ago, funding of projects has been focused on sites where the greatest environmental improvement can be achieved. To target funds at stream crossings with the greatest environmental benefit, the Program implemented a policy to limit the amount and type of structures eligible for replacement. In 2014, the Program enacted policy that 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 the top of the bank where the stream accesses the floodplain and a roughly 1.5-year recurrence interval (7). The existing structure opening width to bankfull channel width ratio to be eligible for replacement with Program funds was originally set at ≤50% but has since been increased to ≤75% to allow more structures to be eligible.

Evaluation of Past Effectiveness

While adequate structure sizing is one of the most important steps in replacing stream crossings, there are a wide variety of other complex issues that impact the overall quality of the installation such as establishing substrate and continuity through the structure, creating a stable slope, and dealing with legacy sediment wedges and plunge pools caused by the undersized structures. TU conducted a survey in the winter of 2018–2019 to evaluate randomly chosen stream crossing replacements across Pennsylvania. Using the North Atlantic Aquatic Connectivity Collaborative (NAACC) crossing assessment protocol surveys were completed at 45 sites. In additional the Program has a QA/QC process where each county's projects are evaluated every 3 years. To support the stream crossing projects, design assistance and education is provided to the Program through the Penn State University Center for Dirt and Gravel Road Studies (Center) and a partnership with TU. This education and technical

assistance initiative aims to improve the quality of stream crossing installations and support the new standards.

Defining New Standards

In 2014 the policy for replacement consisted of 4 major points: 1) structures must have a single opening that at least spans 100% bankfull width channel; 2) be properly aligned with the channel when possible; 3) consider additional floodplain connectivity; 4) be designed and constructed to accommodate aquatic organism passage (AOP). No additional standards or significant technical guidance was provided to guide engineers in proper design and many projects were found to not be meeting the policy, especially with regards to AOP. Starting in 2021 a year long effort was undertaken by the Program to clearly define new standards for stream crossing design and provide an accompanying technical manual for engineers and designers. This effort relied heavily on the US Forest Service "stream simulation approach to road- stream crossings" and guidelines and publications from Vermont and Massachusetts (*8*, *9*, *10*)

Findings

Focusing on Undersized Structures

While the determination of bankfull channel width is somewhat subjective, it has provided the necessary quantification to focus funding on undersized structures. The first step in defining eligibility within the Program is to verify that the structure is undersized and qualifies for funding. Using the threshold of ≤75% structure to bankfull ratio allows 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 and restore AOP (Figure 1).

Evaluation of Past Effectiveness

With 100 stream crossing replacements completed annually at the local level, it is important to evaluate effectiveness and provide education. Through the 2018-2019 TU study it was determined that many of the sites had NAACC course screens of "no AOP"

(a)   **(b)**

FIGURE 1 (*a*) Before and (*b*) after the replacement of a 3-ft corrugated pipe with an 11-ft bottomless structure. The existing pipe was constricting the 8-ft bankfull channel and caused large outlet drop and scour hole.

(12 sites) or "reduced AOP" (27 sites) for a combined total of 87% of the sites. Only six of the sites received a "Full AOP" rating in the NAACC course screen. NAACC course screen results by structure type are provided in Table 1. Only open-bottom arch bridges and culverts and pipe arch–elliptical culverts surveyed received "full AOP" ratings. No round culverts or box culverts that were surveyed were rated as "full AOP". This study along with QA/QC visits highlighted problems with AOP and structure substrate depth which solidified the need for more comprehensive standards.

Defining New Standards

After a year of development, the Program's new Stream Crossing Design and Installation Standard and accompanying technical manual were adopted by the SCC and went into effect on July 1, 2022. Two of the biggest changes to the replacement standard are an increase the minimum structure size to $\geq$125% of the bankfull channel, and a

TABLE 1 NAACC Course Screen Results by Structure Type

Structure Type	No AOP	Reduced AOP	Full AOP	Total
Round culvert	7	10	0	17
Open-bottom arch bridge–culvert	1	5	5	11
Pipe arch–elliptical culvert	3	10	1	14
Box culvert	1	2	0	3
Total	**12**	**27**	**6**	**45**

requirement the structure pass the 100-year discharge (Q100) at 80% of the finished opening height. This allows for the construction of not only low-flow and bankfull channels and bank margins inside the structure, but additional flood capacity and increased resiliency of the structure (Figures 1 and 2). While this has caused some pushback from local municipalities and engineers, mostly in regard to added cost, it has generally been accepted. Since the Program is funding the crossing replacements, adequate sizing to meet the standard is a requirement for a project to proceed. Although this requirement recently went into effect, road owners who have been historically installing structures >125% of bankfull typically report a reduction in maintenance such as washouts and gravel deposition associated with the new larger structures.
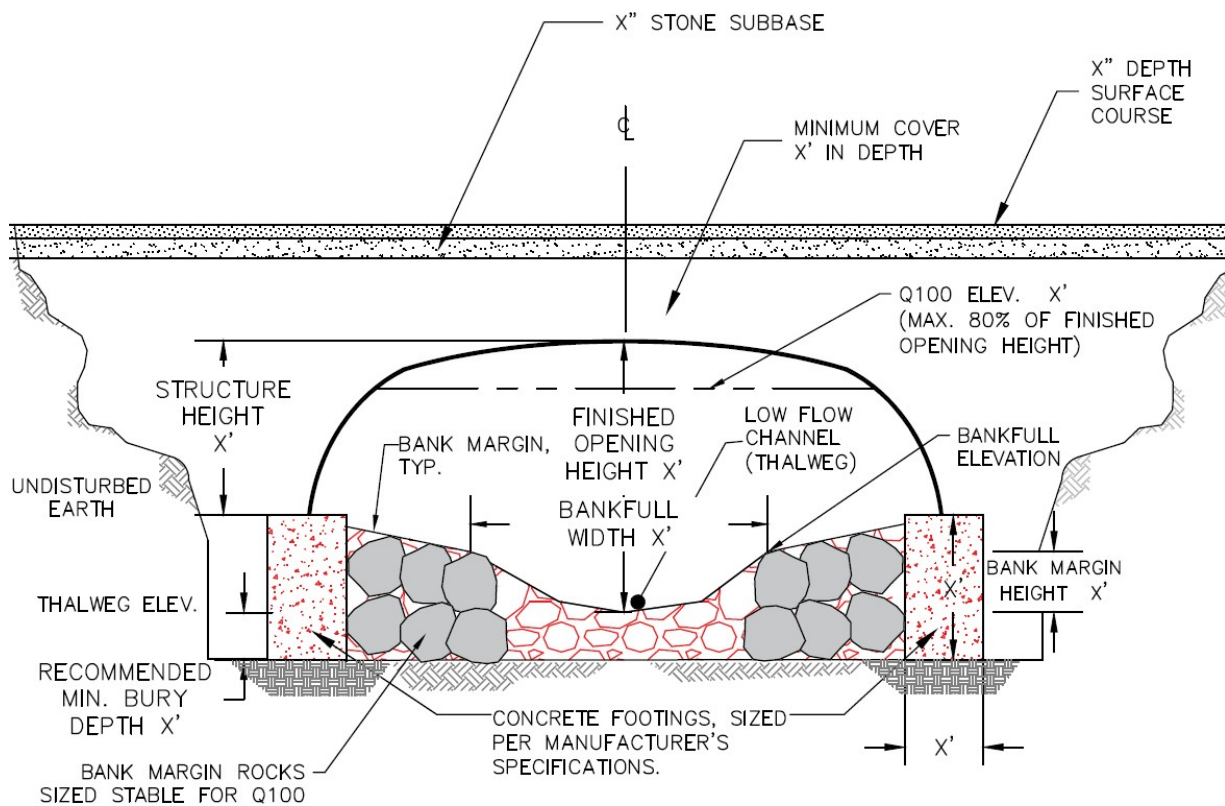
FIGURE 2 A typical cross section at a grade control showing channel shape through a culvert. The dashed line shows the water surface elevation of the 100-year discharge.

In addition to changes in structure sizing and hydraulic capacity, the following elements were added to the standard to ensure successful stream crossing replacement projects.

- Require longitudinal profile survey 150 ft upstream and downstream of the structure.
- Require added streambed depth in structures.
- Requiring bottomless structures over 4% stream slope.
- Construction design plan requirements from engineers.
- Increased oversight and inspection from engineers.
- Engineer certification at completion of the project.

Conclusions

Through the TU study and the Program's QA/QC process it was determined that many stream crossings funded by the Program did not meet the goal of providing AOP. The development and implementation of the new Standard and accompanying technical manual will help ensure future stream crossing replacements meet the environmental goals of the Program, including improving AOP while providing increased storm resiliency considering increased flooding. The use of bankfull channel width as a quantification threshold, while slightly subjective, has been a great benefit in both determining crossing eligibility for replacement, and as a basis for sizing new structures. While there have been some past issues resulting in less than desirable projects, the new standard and educational efforts of the Pennsylvania Dirt, Gravel, and Low Volume Road Maintenance Program should result in a higher number of successful projects. The Program will continue to evolve policies and educational efforts based on the success of future projects using the new Standard and technical manual.

Acknowledgments

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Planning of LVR Drainage Works, Considering Climate Change in Selected Developing Countries

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Climate change, including the ongoing increase of the CO_2 emissions, global temperature, sea level rise, storm intensity increases, and the related characteristics of extreme climate events, are analyzed in terms of planning and implementation of cost-effective low-volume road (LVR) drainage works, including low-cost solutions of sediments and debris traps and related check- dams used successfully in selected Caribbean and South American (CSA) areas to avert clogging and blockage of LVR drainage systems. Lessons learned indicate that storms intensity projections (SIP) could vary between –70% to +45% of the highest registered storm intensities (*1–4*). Also, SIP values of less than 20% have been associated with prolonged draught periods that have decreased the ability of the local native vegetations to support the stability of road slopes on hilly or mountainous terrains, during the occurrence of following high-intensity storms with SIP> +20%. This paper addresses ongoing climate change challenges in terms planning and implementation of low-cost LVR drainage works used successfully to avert clogging and blockage of LVR drainage systems, and thus averting flooding damages during high intensity tropical storms in selected CSA areas.

Impact of Climate Change on Planning of LVR Drainage Systems

Observed and projected changes of the global average temperature, CO_2 emission, and sea level rise (SLR) are given in Figure 1*a* to 1*d*. Figure 1*a* presents projected changes of the global average temperature under four CO_2 emission scenarios, indicating that the projected Year 2100 global surface temperature increase could vary from 0.5°C to 4.1°C for low- and high-emission scenarios. These temperature changes are relative to the 1986–2005 average global temperature, see the IPCC

(a)

(b)

(c)

(d)
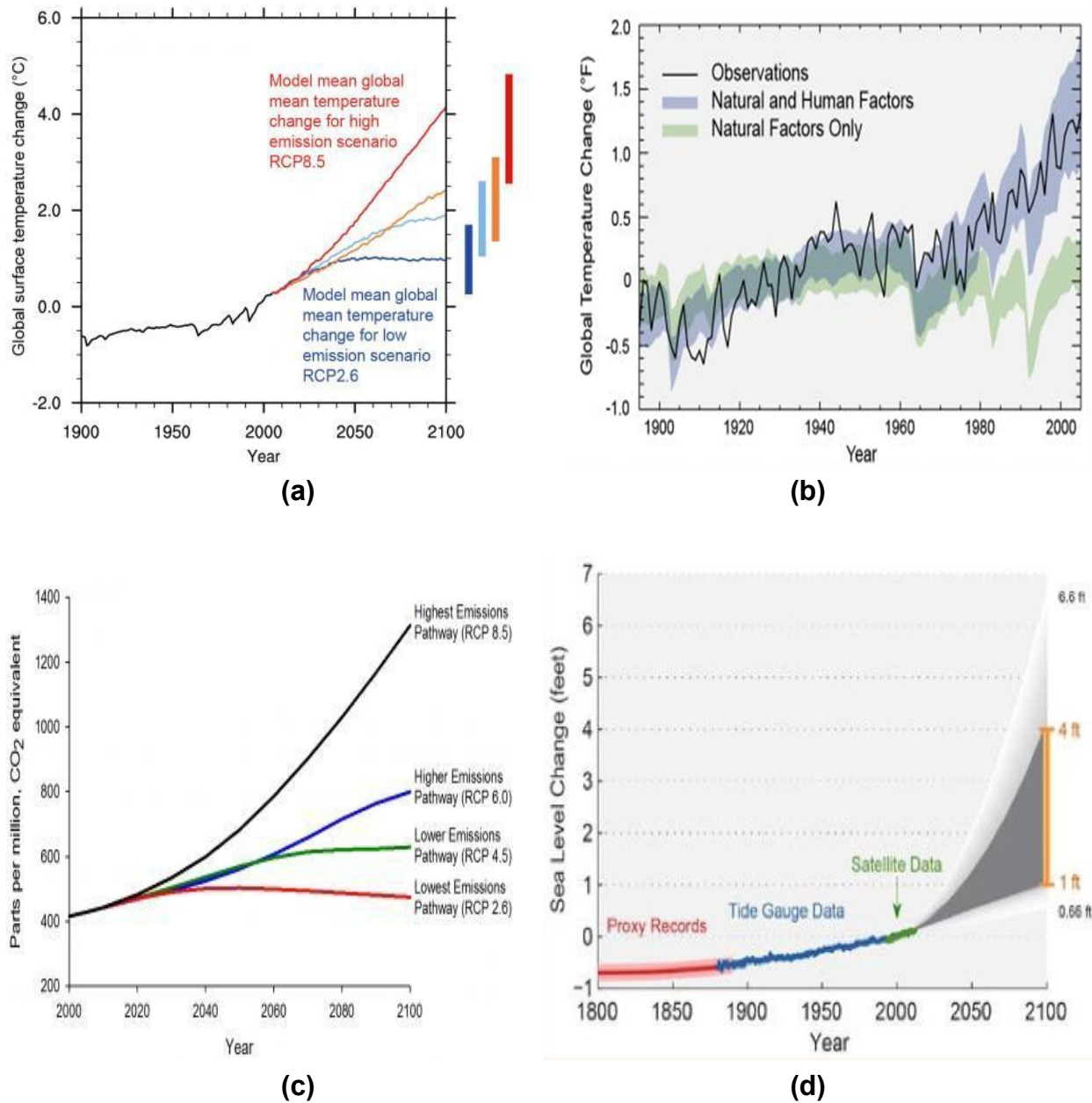
FIGURE 1 Observed and projected changes in global temperature, CO₂ emissions, human impact on temperature change, sea level rise (1): (*a*) past and projected global surface temperature change; (*b*) human and natural influences on global temperature change; (*c*) CO₂ representative concentration pathway (RCP); and (*d*) past and projected sea level rise.

reports (*1*). Figure 1*b* presents human and natural influences on the global temperature changes, showing that the global temperature has changed from –0.6°F to +1.2°F, from the late 1800s to the early 2000s. Also, Figure 1*b* indicates that since the 1970s the human influences are more significant than the natural Influences on the increase of the global temperature. Figure 1*c* presents projected greenhouse gas of carbon dioxide (CO_2) concentrations of four different emissions pathways. The top pathway assumes that greenhouse gas emissions will continue to rise throughout the current 21st century. The bottom pathway assumes that CO_2 emissions would reach a peak between 2010 and 2020, declining thereafter. It has been concluded that this CO_2 bottom representative concentration pathway (RCP) has not been achieved. Therefore, it is more logical to assume that the highest emission pathway of RCP = 8.5 could be reached in the year 2250, indicating that the CO_2 concentrations would be around 2000 ppmv, and the global temperature rise could be over +2°C, IPCC-2022 reports (*1*).

Lessons learned from coastal areas of the Caribbean and South America areas indicate that excessive land development, higher ambient temperatures, prolonged draught have been associated with significant decrease of the ability of native vegetations to support the stability of road slopes and avert disastrous erosion and slope failures during succeeding tropical-storms events. Also, the ongoing rise of the global temperature has been associated with an ongoing sea level rise (SLR), expected to continue rising between 0.66 ft to 4.0 ft or up to 6.6 ft, by 2100, see Figure 1*d* (*1*).

IPCC reports (*1*) conclude that these climate changes are linked to the rising levels of CO_2 and other greenhouse gases in our atmosphere, caused by human activities such as burning fossil fuels for energy. Also, to reduce urgent climate changes related risks, these IPCC reports have recommended accelerating the implementation of the political commitments and the related financial, and technical actions needed to reduce the emissions of the greenhouse gases that have contaminated and warmed our planet. In this regard, Figure 2, see the upper fivephotos (*a* through *e*), presents significant flooding and erosion damages caused by Hurricanes Harvey (Barbados, August 2017) and Erika (Dominica, August 2015) in selected coastal areas of these Caribbean nations. Lessons learned in terms of planning and implementation of low-cost solutions of sediments and debris traps are given in Figure 2*f* through 2*i*, see lower photos/charts. These engineering solutions include:

1. Check Dams used to trapping sediments and debris along the gullies to reduce the entrance of debris into the drainage system and thus averting severe flooding;
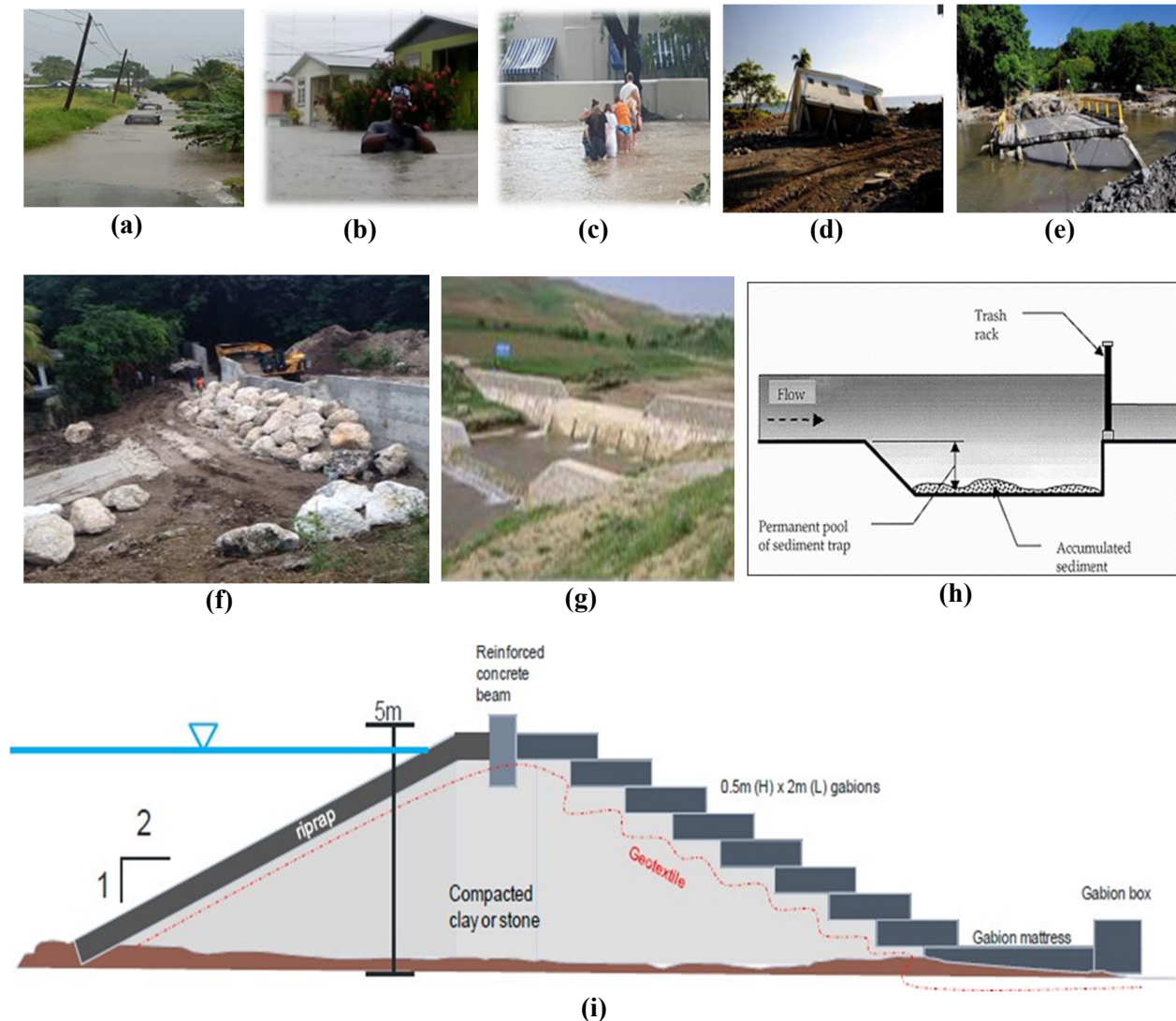
FIGURE 2 The photos *a* through *e* present severe flooding damages caused by hurricanes, associated partially by climate changes: (*a, b, c*) in Barbados, Hurricane Harvey, August 2017 and (*d, e*) Dominica, Hurricane Erika, August 2015. The photos *f* through *h* present low-cost solutions of sediments and debris traps used successfully in the Caribbean areas to avert clogging and blockage of the drainage systems, including: (*f, g*) a Check Dam (under construction, and completed) used to trapping of sediments and debris in the gullies to reduce the sediment and debris deposition into drainage system and thus reducing the effective gradient and averting flooding and (*h*) gross sediment-pollutant traps consisting of a combined sediment basin and trash rack usually located at the upstream or downstream of the constructed stormwater pipe or the constructed drainage channel, as applicable to avert flooding. The lower chart (*i*) presents a retention structure (dam) designed to reduce the volume of flow reaching Caribbean coastal plains. Also, for the Dominica's rocky topo and geotechnical characteristics, this 5-m dam could retain approximately 50,000 m³ of water, while excavating this site by an additional 6 m could triple the retention capacity to 155,000 m³.

2.   Gross sediment-pollutant traps consisting of a combined sediment basin and trash rack usually located at the upstream or the downstream of the constructed stormwater pipe- system or the drainage channel, as applicable to avert flooding; and

3.   Retention structures designed to reduce the volume of the flow reaching the coastal plains of the Caribbean areas to avert clogging and blockage of the drainage systems. Also, the implementation of the drainage solutions given in figure 2 in selected Caribbean coastal areas, including the Dominica's Mero and the Barbados Holetown-Trents communities, have performed well and averted the destruction damages caused by the tropical storms Erika and Harvey in other locations of these nations (*2*, *3*). In this regard, it is important to indicate that donors, including the World-Bank, Inter-American-Development-Bank, and USAID that have financed LVR projects in developing countries, have requested to incorporating climate changes adaptation analysis into the planning, design and construction of LVR drainage works (*2*, *3*, *5*).

Climate Change Impacts on LVR Planning of Drainage Works

Developing countries have faced excessive land development, higher ambient temperatures and storms intensity changes that have varied between –70% to +45% of the local registered storm intensities (*1–4*). In this regard, significant prolonged draught in the Caribbean areas of Barbados and Dominica was associated with storm-intensity reduction of approximately –20%. As an example, a 4-year drought (2011–2014) associated with high ambient temperatures had decreased the ability of the Dominica's local native vegetation to support the stability of the road slopes on the islands' mountainous/hilly terrains. However, this prolong draught pattern was changed significantly in August 2015 when the Tropical Storm Erika struck Dominica and killed 20 people (*4*). Erika was one of the deadliest and most destructive natural disasters of Dominica since Hurricane David in 1979 and was the fifth named storm of the 2015 Atlantic hurricane season, with the following destruction characteristics: highest wind speed: 53 mph; duration: August 23 to September 3, 2015; and the actual damages reached $511.4 million.

Erika's storm data was collected at the climate station Doleac Gommier, located at the mountains area of Dominica on August 27, 2015. Erika's rainfall accumulation between 1:00 a.m. and 5:00 p.m. was 17.08 in. (434 mm), of which 14.1 in. (359.7 mm) accumulated from 4:00 to 9:00 a.m. These characteristics were approximately +25% above the rainfall intensities that strike Dominica in the previous 35 years, namely 1980

to 2015. As a result of this intense rainfall and in combination with the steep topography and the relatively short distance of 6 mi from the center of the mountain ridge to coastal areas, flash flooding rapidly ensued with little warning to the population (*4*).

The back calculation of the eroded Caribbean slope failures has resulted in a friction factor of less than 1.0, considering the local topographical, geological, geotechnical, slope-vegetation characteristics and SIP =+25%.

Considering the Caribbean's areas significant damages caused by the tropical storms David (1980), Erika (2015), and Harvey (2017), and the conclusion that the friction factor against slope failure could reach the critical value of 1.0 during future similar tropical storms, Donors and local governments decided to upgrade the drainage systems of local coastal communities of Dominica and Barbados. These drainage improvement works were design to retaining of up to 155,000 m^3 of water and to screen out over 1,500 m^3 of granular-cohesive sediments and debris, considering the local topo, geological–geotechnical and climatical characteristics, projected storm intensities and related precipitation patterns changes.

The implementation of debris traps in Dominica and Barbados was necessary and sufficient to sustain the movement of water to the sea, and thus averting the increasing of marine loading, and hence complying with the Endangered Species Act (ESA) of the United States and the Caribbean nations.

Conclusions

A simplified methodology of how to incorporate ongoing project-level climate changes impacts, in terms of SIP, into the planning and implementation of LVR drainage works is presented. Related climate change indicators include:

    a. Probable increase of CO_2 emissions and related increase of the global temperature, expected to rise in the range of 0.5°C to 4.8°C (8.6°F) by 2100, with a most likely increase of 2.7°F (1.5°C) for all scenarios, except the one representing the most aggressive mitigation of greenhouse gas emissions;

    b. Probable sea level rise, estimated to be over 20 cm in 50 years and over 30 cm by the year 2100;

    c. Probable related increase of future storms intensity characteristics;

    d. Probable continuation of the ongoing shifting of snow and rainfall patterns in terms of producing more extreme climate events with increased SIP of up to +45%, although with high level of uncertainty;

e. Probable SIP changes in the range of –70% to +45% of previous high registered storm intensities;

f. Probable more severe flooding damages caused by extremely high tropical storms that produced rainfall intensity above and beyond previous storms; and

g. Probable prolong drought periods, associated with the rise of the ambient temperature, that have decreased the ability of local native vegetations to support the stability of road slopes in selected developing countries (*2–4*).

This paper presents cost-effective drainage adaptation solutions used successfully to protect Caribbean coastal areas from severe flooding caused by Tropical Storm Erika (August 2015) that struck the island of Dominica and killed 20 people and caused infrastructure damaged of $511.4 million. In this regard, the climate change characteristics of Dominica included higher ambient temperatures, prolonged drought that had decreased the ability of the natural vegetation to keep the stability of the road-slopes on the islands' mountainous and hilly terrains from the high rain intensity of Erika, which was one of the deadliest most destructive natural disasters since Hurricane David in 1979. Also, these drainage adaptation solutions used successfully to protect coastal areas of Barbados from future tropical storm such as Harvey (August 2017) that hit the Caribbean Gulf areas and forced about 39,000 people out of their homes, disconnected 200,000 homes from their power, and destroyed at least 1 million vehicles.

International donors and their member countries require to optimize the design and construction of LVR drainage works, addressing the

a. Local climate change in terms selecting the most probable project-level projected storm intensity and duration and related sea level rise;

b. Land-use planning and related development investment activities;

c. ESA and all other local environmental preservation legislations and regulations; and

d. Producing a quantitative benefit–cost analysis needed to justify the LVR investment and maintenance costs in terms of producing an economic IRR of over 12%.

In this regard, the BCA of the drainage systems improvement works in Dominica and Barbados, including related environmental mitigation and remediation works, resulted high economic return, namely BCA of over 1.8 and IRR of over 15%.

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Evidence-Based Targeting of Infrastructure Investments to Foster Human Capital Development and Reduce Accessibility Gaps in Rural Areas

The Khyber Pakhtunkhwa Rural Accessibility Project, Pakistan

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Equitable access to basic services, e.g., food, healthcare, education, and administrative services, is a fundamental right (*1*). The inadequate availability or unequal access of communities to these services can have a detrimental impact on household welfare, development outcomes, and poverty alleviation efforts (*2–4*). Accessibility is often restricted by characteristics of the physical and built environment, including geography and the availability and quality of infrastructure. Infrastructure investments undertaken to improve road networks can play a vital role in connecting disadvantaged rural and remote areas to services and economic opportunities (*3*, *5*). However, too often, road network investment decisions are based on subjective considerations and fail to quantify accessibility gains. Projects and policies that factor in community accessibility tend to rely on limited, imprecise, or poorly documented data (*6*). Data to inform project design and targeting tend to be derived from larger-scale surveys or case studies with a limited geographic scope (*6*).

In addressing these challenges, the World Bank (WB) approved the Khyber Pakhtunkhwa Rural Accessibility Project (KP-RAP) in 2022, a rural road rehabilitation project that goes beyond traditional road infrastructure investments. The project leverages data to prioritize rural, low-volume roads for upgrading that maximize

accessibility gains for communities in the most vulnerable districts of rural Khyber Pakhtunkhwa (KP), generating tangible social and health benefits, particularly for women and girls. KP is a province in northwestern Pakistan, bordering Afghanistan. It is affected by many natural and manmade hazards, e.g., climate change and terrorism, both disrupting access and connectivity. With a population of more than 35 million, it is the third most populous province, with the second highest poverty incidence in the country (*7*). Despite a growing population, KP has the lowest level of urbanization, with around 80% of inhabitants residing in rural and often remote areas (*8*).[1] The province is confronted with sizeable urban–rural disparities in human development indicators, with vast gaps in accessibility to and availability of basic services and opportunities between urban centers and remote areas.

The project leverages a data-driven approach for targeted infrastructure investments, fostering maximal human capital development and minimizing accessibility gaps. To generate a critical evidence base of current and future accessibility and maximize impact, the project team developed a methodology and toolkit to measure and visualize disparities in service accessibility at small scales using geospatial analysis. Findings contributed to the early-stage planning and design of the project and were used to propose the selection of low-volume rural roads for rehabilitation based on the highest potential impact in improving accessibility by prioritizing underserved communities within the resource constraints of the project. This is of particular importance in KP, where limited resources are allocated to meet road maintenance needs, necessitating an out-of-the-box solution to addressing its poorly managed road network and maintenance backlog. The analysis identified over 600 km of roads in 14 districts to be rehabilitated and maintained with priority. KP-RAP is projected to benefit 2 million rural inhabitants by improving access to schools, healthcare facilities, and markets. This will increase year-round connectivity, boost human capital development through accessible education and healthcare, and generate economic opportunities, especially for farmers, thereby reducing poverty and strengthening drivers of inclusive growth. The project has garnered much acclaim at all levels of the country's government and won the WB's Vice-Presidential Unit award in fiscal year 2022.

Methodology

The project seeks to build an evidence base for targeted resource allocations and interventions to reduce accessibility inequalities at a highly spatially disaggregated level. To begin, current accessibility gaps between districts (second-level administrative units

in Pakistan) were established. For this, KP was broken up into gridded "rasters" with cells of size 30 m by 30 m. The model then created "friction surfaces" by assigning travel speeds for a standard car or pedestrian across each raster, factoring in the local terrain (slope and elevation, water bodies, type/surface/condition of road, seasonality, land cover, etc.). Three such friction surfaces were created: (1) one modeling multimodal transport using the fastest available option (vehicles on roads, walking off roads), (2) one modeling adult walking speeds only, and (3) one modeling children's walking speeds. For on-road vehicles, speeds were assumed as the speed limit, while walking speeds were assigned as a function of terrain and age. Differences in average travel speeds across three seasons—summer, winter, and the rainy season—were considered to better reflect ground conditions. Slower travel was assumed during the monsoon (reflecting precipitation) and winter seasons (at high altitude areas only, reflecting snow).

Next, these friction surfaces were used to determine travel times between each populated grid cell and the nearest school, healthcare facility, and agricultural market (particularly significant in KP, with a large agricultural dependence). A broad set of data inputs were utilized, summarized in Table 1 below.

For primary schools, usually located within or near communities, travel times were calculated based on children's walking speeds. For all other points of interest (middle and secondary schools, healthcare facilities, and markets), the fastest combination of

TABLE 1 Accessibility Modelling Data Inputs

Type of Data	Data Source
Location of facilities	Primary data sources and existing geodatabases, e.g., GeoNames
Administrative boundaries	WB's World Subnational Boundaries, United Nations Office for the Coordination of Humanitarian Affairs, 2018
Proposed road network	Government counterparts (KP), OpenStreetMap data
Population	National population census, global population data layers from Facebook (high-resolution satellite layers) or WorldPop.
Development outcome indicators	Government of Pakistan, Pakistan Social and Livings Standards Measurement Survey
Speed (seasonal and road-type walking/vehicle speeds)	Irmischer and Clarke (9); WB Poverty and Transport Global Practice
Services	Management Systems International in Pakistan/United States Agency for International Development, KP school data

Source: (10).

walking (for adults) and motorized transport was chosen. In most cases, the routes chosen primarily featured vehicular travel, unless the facility in question was very close to the populated place. Road data in priority districts, obtained from government sources, was improved via manual "tracing" of satellite imagery in OpenStreetMap (OSM). Modeled travel times in each grid cell were weighted by the underlying population density and aggregated per administrative unit, weighing each season's average by the average length of said season in that unit. The resulting "accessibility index," allows comparisons across administrative units, i.e., tehsils[2], revealing highly detailed gaps in service provision.

Finally, to prioritize roads for upgrades, the project compared the degree to which upgrading a particular road improves accessibility, relative to other roads. For this, the analysis first determined populated places that are likely to use a particular road to access the nearest facility of a certain type. It was assumed that travel speeds on an upgraded road would be faster, resulting in time savings, which were calculated for each facility subcategory (e.g., primary, middle, high schools) and used to obtain an "accessibility improvement index" for each service type (i.e., schools, healthcare facilities, and markets).[3] These indices were then aggregated into an overall accessibility improvement index, allowing to compare access gains and select particular roads for upgrade.

Findings

The geospatial analysis was carried out at the district and tehsil level for KP. Approximately 40% of the rural population requires more than an hour of travel by motorized vehicle to access a health facility, while 44% must drive more than 30 min to reach a primary school (80% in some districts). Moreover, 90% of the population lives more than two hours away from an urban center (*11*). This has direct implications for human capital and economic development indicators in the province, which already experiences high mortality and out-of-school children's rates (*12*).

The highest accessibility gaps lie in the northern and southern regions of KP (Figure 1), where access to health facilities, schools, and markets is limited. Household surveys show 40% of rural middle schools and 50% of rural secondary schools are located more than 2 km away (*12*). Furthermore, there is significant gender disparity in access to education in these areas, with the northern region showing the highest disparity: girls' access may be 20% to 50% lower than boys' (*10*).
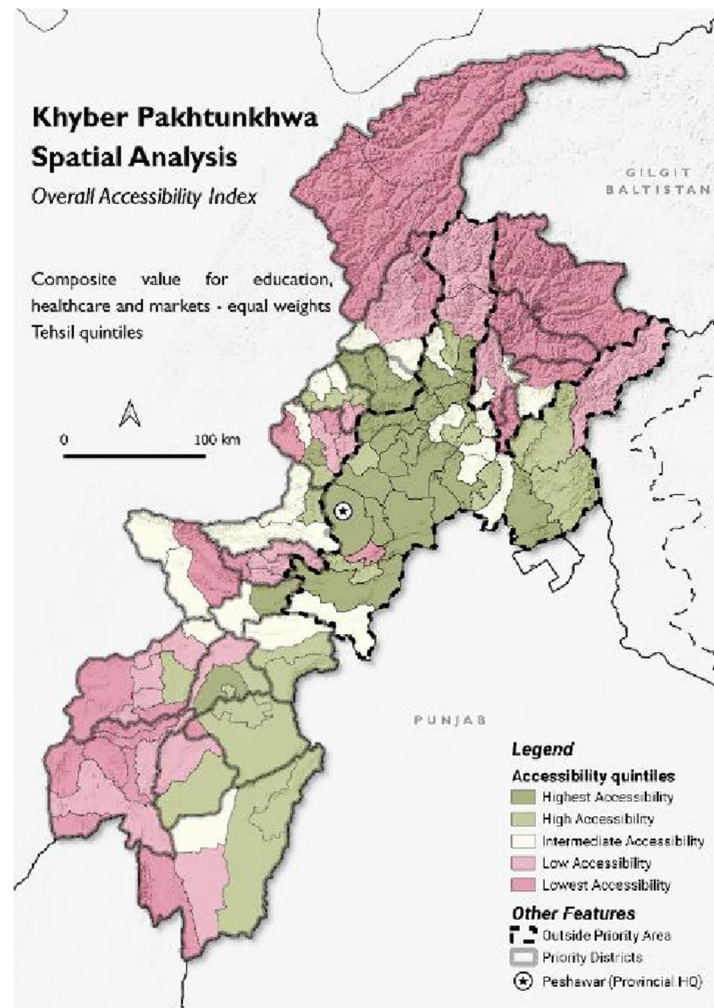
FIGURE 1 Accessibility to education, health care, and markets (equal weights assigned). [Source: Internal World Bank Project Document (*13*).]

This disparity is partly due to gender segregated schools in rural KP, with fewer schools catering to girls. Girls, therefore, travel longer distances than boys in all districts, with 50% longer travel times. The travel time gap worsens with level of education, as middle and secondary schools are far fewer in number than primary schools. This, in turn, impacts girls' enrollment and attendance rates. Based on the geospatial accessibility analysis, Figure 2 depicts the ranking of districts for overall accessibility, using equal weights for the three metrics (access to education, health centers, and markets), with priority districts indicated by asterisks. Districts are indexed in terms of how inaccessible they are relative to each other: a higher inaccessibility score indicates a poorer level of access. Kohistan is the worst-performing priority district, while Lower Dir is the best (Figure 2).

Presently, the government of KP has identified 79 sections considered for upgrading under KP-RAP based on the geospatial accessibility analysis. The methodology was applied to the proposed roads comparing hypothetical post-investment improvements in accessibility. Longer roads may improve accessibility more than shorter ones. Improving roads is likely to improve educational access more than access to health facilities and markets, as more schools are present.

Conclusion

The geospatial accessibility analysis shows that upgrading roads with the lowest accessibility and highest gender disparity yields maximum accessibility benefits, building the case for investments in areas usually overlooked by governments. The analysis can be replicated using data on roads, population, terrain, and the environment, which tend to be publicly available. Any data gaps from government sources can be narrowed by public data such as OSM. For this model, only public service data were available, meaning private sectors in health and education were not accounted for.

This analysis can be used in road construction, rehabilitation, and service accessibility projects, as it provides information on how to prioritize investments to maximize accessibility benefits, optimizing the use of limited resources rather than being driven by other motivations. Moving forward, the developed toolkit can help optimize site selections for new schools and hospitals, based on accessibility to population centers. It serves as an evidence base for government counterparts to
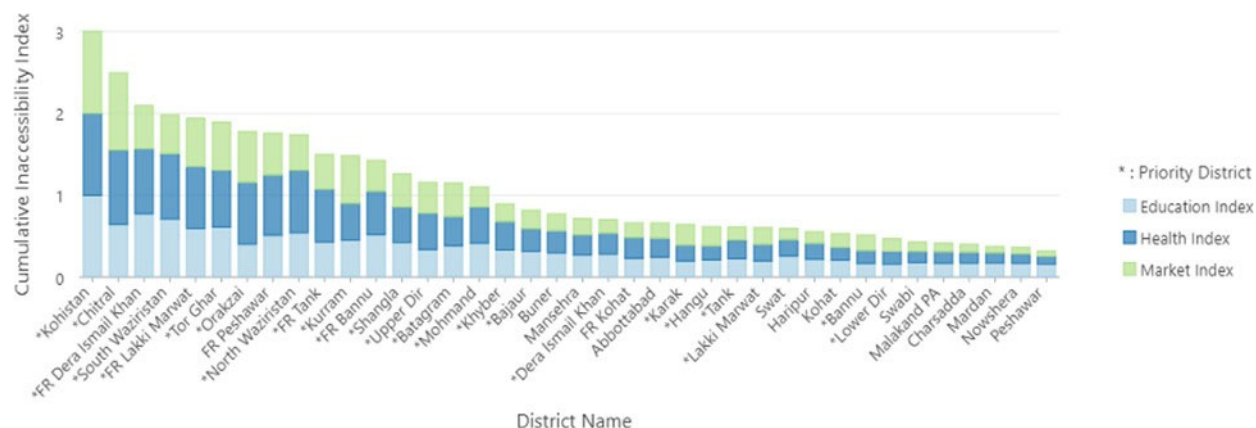
FIGURE 2 Districts ranked by accessibility (equal weights across all three sectors). (Source: Author's estimations.)

reduce accessibility gaps and improve human capital development indicators. This is an innovative approach to typical infrastructure projects in areas that have physical constraints, where decisions are based on closing the gap in service accessibility. It helps to provide accurate data at a highly granular level for remote areas, guiding investment and policy decisions.

Moreover, it allows focus on specific vulnerable groups (i.e., girls' and women), which may be difficult to achieve otherwise. This model is very flexible and may be used for any services having spatial features, including healthcare, markets, education, etc. This analysis is replicable and scalable for other geographical regions and is an ongoing research effort. It has previously been used in Nepal, Bangladesh, Bhutan, Sri Lanka, and Afghanistan.

Notes

1. Excluding Federally Administered Tribal Areas (predominantly rural), where only 2.8% of inhabitants reside in cities.
2. Small-scale third-level administrative units.
3. The model's built-in feature allows assigning different weights to any metric, according to users' priority, when determining this index.

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Forest Service Road-Stream Crossings

Designing for Flood Resilience, Northern Wisconsin Case Study

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In a changing climate, extreme weather events are becoming more common across the United States, with direct negative impacts on transportation infrastructure, particularly at road-stream crossings. As an example, on July 11–12, 2016, up to 9 in. of rain fell in Northern Wisconsin over a 6-h period, resulting in the failure of over 40 road–stream crossings and damaging 300 mi of road infrastructure on the Chequamegon–Nicolet National Forest. The best available data suggests that these rainfall amounts exceeded the predicted 500- and 1,000-year recurrence interval amounts.

Although 40 road–stream crossings failed during the flood, all but one of the crossings that were replaced within the previous 15 years using the Stream Simulation design approach survived this catastrophic flood event. While this paper summarizes data from the Wisconsin flood, the US Department of Agriculture (USDA) Forest Service has found flood resiliency efficacy for the Stream Simulation design methodology from other recent large flood events across the country.

Stream Simulation is a methodology for designing road–stream crossings that was developed by the USDA Forest Service to simultaneously provide flood resiliency for the transportation network as well as unimpeded migration for fish and other aquatic organisms (USDA Forest Service, 2008; Cenderelli et. al, 2011).

The premise behind Stream Simulation is to reconstruct a suitable stream channel though a road–stream crossing and span that new channel with a bridge or culvert structure. The width and rise of this new culvert–bridge structure is based on the natural channel dimensions present in the vicinity of the road–stream crossing. A reference reach of similar gradient to the crossing is located with a specific methodology, primarily based on the longitudinal profile. Once a suitable reference reach is located, stream attributes like width, scour depth, bed material sizes, bedform type and spacing, etc.,

can be measured and then used as a template for designing a streambed through the new crossing. Basing the replacement structure dimensions on observations of the stream channel is a substantial departure from historic hydrology and hydraulic (H&H) design methods, which bases structure sizes on an estimated flood discharge and an acceptable level of ponding at the structure inlet. This hydraulic check is still performed in Stream Simulation but is not used as the primary sizing criterion. Stream Simulation typically seeks to maintain 20% clearance (freeboard) during the 1% annual exceedance interval flood (USDA Forest Service, 2014). With climate change, the predictability of flood frequency is becoming difficult, but the recent floods occurring on Forest System Lands reveals that Stream Simulation designs are surviving storm events with average recurrence intervals of 500 to 1,000 years, whereas the smaller hydraulic culverts are failing catastrophically.

In reviewing structure survivability from the July 2016 flood event in northern Wisconsin, 17 road–stream crossings sites were analyzed that were designed using Stream Simulation. Of those 17 Stream Simulation sites, 16 survived. This data provides insights on how to design road-stream crossings to provide resilient infrastructure in response to a changing climate with economic, social, and natural resource benefits.

Not only does Stream Simulation meet the Forest Service's requirement for sustainable multiple-use land management, but it is also proving to be an economic benefit in terms of flood resilience and dramatically reducing maintenance costs. The success of Stream Simulation during Tropical Storm Irene in Vermont proved to be an economic and ecological benefit (Gillespie et al., 2014). Analyzing the lifetime cost of "ecologically designed" culverts by O'Shaughnessey et al. (2016), which did not consider environmental costs or benefits, concluded that there is a positive net fiscal benefit in many cases.

Methodology

For the northern Wisconsin flood event, precipitation data were collected from the National Weather Service, to estimate total storm rainfall for the watershed at each road-stream crossing. US Geological Survey (USGS) regression equations were used to develop flood frequency curves for each site (Walker and Krug 2003).

Additionally, the post-flood high-water mark at each site was identified by physical characteristics such as debris, seed, and mud lines (Koening et. al. 2016) and this elevation was used to determine headwater elevation for input into the same HEC-RAS

model that was used for the original structure design. Each model included at least four cross sections (two upstream of the crossing and two downstream of the crossing), the culvert size, culvert invert elevations, and road profile as required by HEC-RAS (Brunner 2016).

From these data, peak discharges from the flood event were estimated (Table 1).

Several valley and stream characteristics considered important to the road–stream crossing survival rate from the flood were characterized for each of the 17 sites. This included flood prone width, entrenchment of the stream (flood prone width divided by bankfull width) (Rosgen, 1994), road fill height, channel slope, and the angle at which the road and stream cross.

An evaluation of design methodologies must eventually consider replacement costs. To that end, a cost comparison is included to help provide a basis of comparison between Stream Simulation design and the preceding structures referred to as in-kind replacement. Some of the in-kind replacements may have been designed using the familiar H&H techniques but it is likely that some sites received no analysis.

TABLE 1 Road-Stream Crossing Summary

Site Name	Rainfall (in)	Drainage Area (sq mi)	2016 Flood Peak Flow (cfs)	0.2% Annual Exeedance Probability (cfs)	Bankfull Width (ft)	Culvert Width Original (ft)	Stream Sim (ft)	2020 Construction Cost Replace In-Kind	Stream Sim
UNT Whiskey, W	9.10	0.21	160	40	5.6	3.50	5.33	$ 16,159	$ 17,230
20 Mi	8.26	0.78	1335	127	7.0	3.00	5.92	$ 15,794	$ 18,111
UNT Morgan Falls	9.02	0.29	220	115	5.0	4.00	7.25	$ 17,436	$ 19,009
UNT Marengo, S	8.11	0.37	215	73	4.2	4.08	7.25	$ 19,307	$ 20,844
UNT Whiskey, M	9.20	0.65	450	92	5.0	4.08	7.25	$ 19,030	$ 22,119
UNT Marengo	8.55	0.21	138	125	4.0	1.75	5.33	$ 14,827	$ 22,466
20 Mi	8.27	0.31	210	59	4.4	2.92	5.92	$ 18,762	$ 22,938
UNT Trout, E	8.02	0.61	365	117	6.0	3.50	7.25	$ 17,979	$ 23,575
UNT Marengo, N	8.11	1.43	350	188	8.6	5.33	7.25	$ 22,245	$ 24,069
UNT Hawkins	8.80	0.94	1150	360	7.0	3.00	7.92	$ 19,380	$ 27,000
UNT Chippewa	7.16	2.16	220	133	8.6	4.75	8.58	$ 21,303	$ 27,405
McCarthy	7.64	1.35	470	155	9.7	6.92	10.00	$ 25,664	$ 52,708
Brush	5.38	6.84	410	319	15.3	5.00	16.00	$ 24,155	$ 64,050
Preemption	7.39	2.67	700	437	10.5	4.75	12.00	$ 27,538	$ 68,573
18 Mi	7.83	20.47	2550	1240	13.0	3.50	16.17	$ 17,956	$ 106,602
Trout	8.08	5.09	1125	640	15.9	4.00	30.00	$ 20,081	$ 217,326
Hawkins (failed)	8.94	3.37	3435	713	16.0	6.92	24.58	$ 25,889	$ 278,710

UNT - Unnamed Tributary

Construction costs for Stream Simulation structures were based on budget records, which include a mix of free market construction contracts as well as installation using construction crews employed by the Chequamegon–Nicolet National Forest. Costs for in-kind replacement structures were developed using similar methods to those of Diebel et al. (2014) and quantities were estimated for each site to match the corresponding structure. All costs were adjusted to 2020 values using the Construction Cost Index (ENR, 2021). Note that survey, design, and contract administration costs were not included in the reported cost data.

Findings

Of the 17 Stream Simulation structures that were analyzed following the July 2016 flood, the only structure that failed was Hawkins. Figure 1 is a map from the National Weather Service depicting the July event with the predicted Annual Exceedance Probabilities for a 6-h rainfall (National Weather Service, 2016). A portion of the Chequamegon–Nicolet National Forest is outlined in pink with the area of culvert analyses taking place within the dotted oval. Although 13 of the 16 culverts that survived overtopped the road, their survivability is attributed to the increased capacity of the Stream Simulation design. During the flood, the tailwater elevation increased with the headwater elevation so if overtopping did occur, there is less head to pressurize flow. The increased capacity and the consistent water elevation allowed debris to pass through the structure unimpeded. Table 1 summarizes rainfall, peak flood flow, and the estimated average 500-year flood flow (i.e., 0.2% annual exceedance probability) at each of the 17 sites. All sites exceeded the estimated 0.2% annual exceedance probability, many by a factor of two to five times, emphasizing the extreme nature of the flood.

In analyzing the factors that contributed to structure survivability, one of the most important factors is having a structure that spans the entire bankfull width and allows additional width for banklines. Table 1 shows bankfull width, original culvert width, and the Stream Simulation structure that was installed within the 15 years prior to the flood.

In general, higher fill heights increase the likelihood of structure failure at overtopping. Fill height was determined by measuring the distance from the low point of the road surface to the floodplain elevation adjacent to the channel on the downstream side of the crossing. The failure due to higher fill heights is likely due to the greater head, velocity, and turbulence at overtopping.
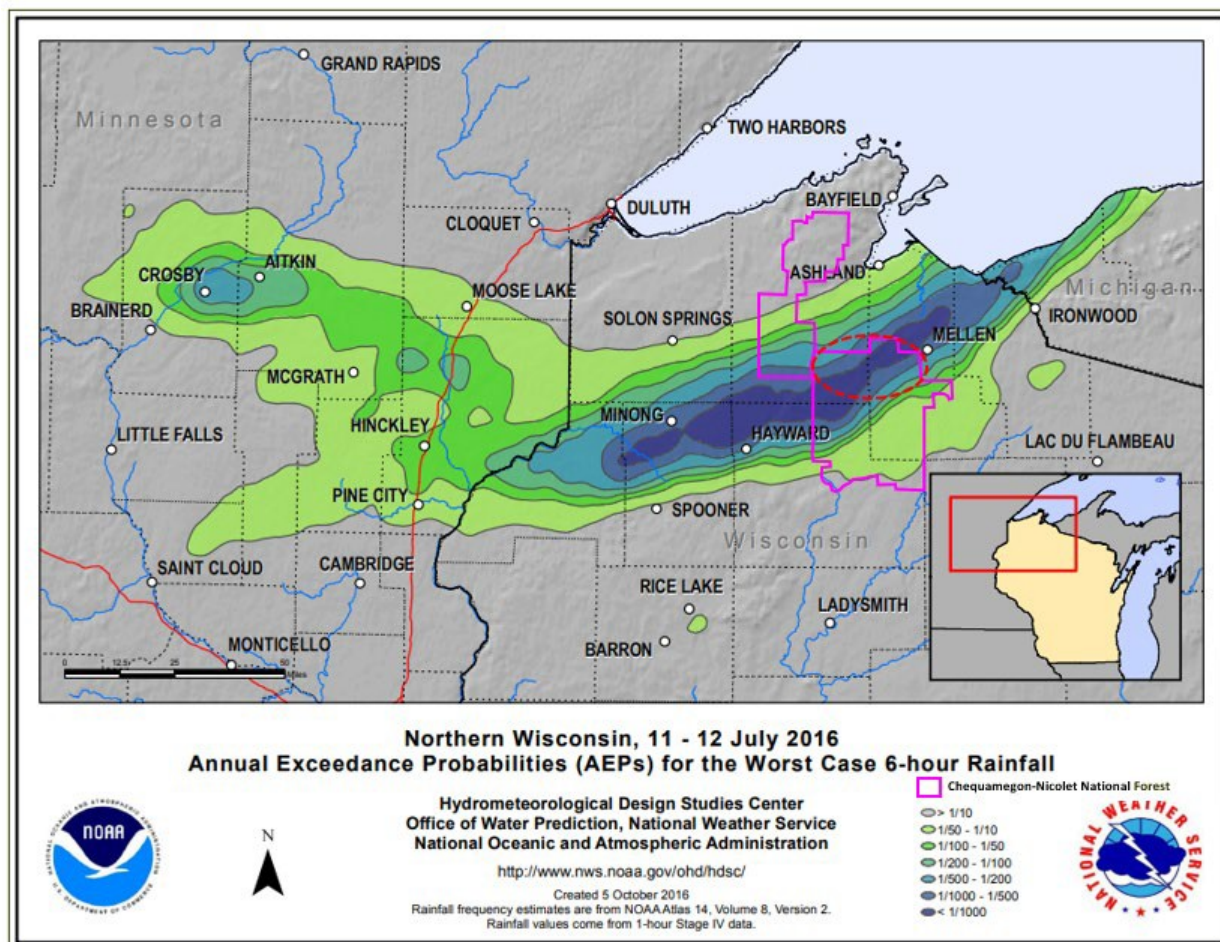
FIGURE 1 Northern Wisconsin July 2016 flood, annual rainfall exceedance probabilities including the Chequamegon–Nicolet National Forest boundary in the flood region (National Weather Service, 2016).

Another potential cause of structure failure, from this large flood, is the skew of the stream to the road. Road–stream crossings where the road and stream run perpendicular to each are not as prone to failure when a road overtops. For the Hawkins structure that failed, the road had a high fill height and a skew of 30 degrees from perpendicular to the stream.

In analyzing the cost of replacing a road–stream crossing with an in-kind structure versus with a Stream Simulation design, the numbers revealed that construction costs generally increase with increased structure size and site complexity; however, the increased cost for structures less than 10-ft wide was generally $1,000 to $7,600 (7% to 52% increase), which is reasonable given their reduced maintenance, longer lifespan, and ecological benefits of stream system connectivity. In Table 1, the structure cost for

Stream Simulation generally correlates with structure width. When comparing the structure area of Stream Simulation to replace in-kind, the cost per square foot is less for Stream Simulation structures (data not included in Table 1).

Conclusion

As many areas of the United States and around the world are seeing increased flood frequency and flood intensity (Climate Central, 2018), which comes with associated high-dollar infrastructure repair costs, the Forest Service's Stream Simulation design at road–stream crossings is a proactive solution for flood resilience. Since these data were collected in 2016, other floods have occurred across National Forest System Lands with similar results.

Because each road–stream crossing is unique within its geomorphic setting, we have come to realize that our standardized H&H design approach leaves road infrastructure at risk. When considering that Stream Simulation structures have longer lifespan, are less prone to plugging thus have significantly lower maintenance costs, and can survive large flood events, the benefits are appealing. Those benefits include improved safety, reliant access, flood resilience, reduced maintenance cost, longer structure lifespan, connected ecosystems, geomorphic connectivity for natural processes and resilient infrastructure.

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TRAINING AND RESEARCH:
PAST, PRESENT, AND FUTURE

A History of the Local Technical Assistance Program

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Local highway and public works agencies manage most of the road mileage throughout the United States and despite this vast responsibility, the men and women working within these agencies often have very little formal education or training outside of learning on the job. As the nation's road system grew, it became apparent that local roads agencies would need some access to training, education, and technical assistance. Today, these services are provided by Local Technical Assistance Program (LTAP) centers located in every state, Puerto Rico, the US Virgin Islands, with an additional Tribal Technical Assistance Program (TTAP) serving US tribal lands. By serving local roads workers and managers on the ground with low-cost, highly accessible training, LTAP centers provide an essential public service to agencies with little access to workforce development and support opportunities.

By examining the history leading up to the creation of today's LTAP program, we can understand how support for the management and maintenance workforce of the national local roads system has evolved and why it is vital to continue that support. This historical survey also explores the positive value that LTAP centers, and the support programs predating them, bring to the largest jurisdictional sector of US roads. By coordinating with major research universities, state departments of transportation, the Federal Highway Administration (FHWA), and numerous industry groups, LTAP centers can deliver benefits to local transportation professionals at a lower cost than any other comparable option.

Introduction

Local governments manage over three-quarters of the centerline miles in the United States, often on limited budgets and limited technical support (*1*). Today, the single most significant source of dedicated technical assistance support for local highway and public works agencies comes from LTAP centers. Currently, 51 of these unique

programs are operating in every state, Puerto Rico, and the US Virgin Islands (*2*). There is also a dedicated TTAP with seven centers that serve US tribal lands. Each LTAP and TTAP center engages in various low-cost, accessible training, education, and support activities tailored specifically to the needs of the audiences they serve within each state. When the phrase LTAP center is used in this paper it includes both the LTAP and TTAP centers unless specifically called out.

　　Since 1981, the LTAP program has bridged the gap between need and support for local roads agencies across the nation (*3*). Small, underserved agencies are provided a variety of personalized training courses, conferences and events, technical assistance, guides, resources, and more to help them undertake the daunting task of managing their systems. In most cases, LTAP center staff and trainers travel all around their states to bring the training to the doorsteps of the people who need it most. LTAP is the result of over a century of progress where the nation evolved toward understanding that local roads, and the agencies that maintain them, require dedicated support structures to work.

History of Technology Transfer for Local Agencies

Early Evolution of the Nation's Road System in the Context of Local Support

When our nation was young, there were very few formally managed roads, and almost all would be considered local by today's standards (*4*). From colonial times to the post-Civil War era, roads were very much purpose-built, and the few that did extend beyond community areas consisted of ancient Native American footpaths, settler migration trails, early post-carrier routes, or military roads.

　　During this period, roads and bridges were managed by the collective efforts of local communities utilizing little more than traditional methods and grit. In fact, most "roads" were little more than well-worn footpaths and muddy trails (*5*). Formalized standards were few and far between, and federal efforts to provide funding or material support were even less common. The military or limited commercial interests drove some early efforts to organize formal road systems. Examples of early American overland road systems include the Braddock Road and the Cumberland Road (*6*). These roads led to the formation of the National Road, which eventually spanned from Cumberland, Maryland, to Vandalia, Illinois, before the federal effort ended and management transferred to the states. The invention and proliferation of rail lines and canal systems also played a part in limiting the development of long-range road-building programs.

Formal nationwide support for a more modern road system would take a movement. Initially, organized groups of bicyclists brought together educational institutions, rural civic organizations, agricultural interests, and others to form what is now known as the Good Roads Movement, a road support advocacy campaign that took place between the 1870s and the 1920s (*7*). The Good Roads Movement was facilitated by an advocacy publication called the *Good Roads Magazine* (*8*). As the movement progressed, good roads associations formed in many states, and dozens of local chapters sprang up. Another early feature of the Good Roads Movement included tying the demand for better roads to the expansion and development of American agriculture and the need to get goods to growing urban markets efficiently.

While providing funding and establishing higher standards for America's roads was a crucial initial goal of the Good Roads Movement, there was also an understanding that the people doing the work locally also needed support and training (*9*). Initially, supporting local road agencies with training, assistance, and education was an effort that took many different forms at the individual state level. Some early state leadership efforts and seminal moments in local roads training and education include:

- In 1881, New Jersey became the first state to grant monetary aid for public road building. The legislature provided aid to counties in the construction of highways up to one-third of their cost and appropriated $875,000 annually to the State New Jersey Secretary of Agriculture for use as the administrator of roads (*4*).
- In New York State, interest in supporting local roads coalesced at the state Land Grant School, Cornell University, where a group of experts and professionals came together for a "Good Roads Week" conference held on campus in 1905 (Figure 1). Over subsequent years, Cornell University held several more conferences and events dedicated to local roads in New York State, which led to the creation of an Annual School for Highway Superintendents in 1938 and eventually the Cornell Local Roads Program itself. The Cornell Local Roads Program would later become the official LTAP center for New York State in 1984 (*10, 11*).
- Auburn University has hosted an annual Alabama Transportation Conference every year since 1958. In 1985, the Auburn University Highway Research Center was formed. Auburn also partnered with the National Asphalt Pavement Association Research and Education Foundation to create the National Center for Asphalt Technology in 1986 (*12*).
- Indiana created the Highway Extension and Research Project for Indiana Counties (HERPIC) at Purdue University in 1959 (Figure 2). The HERPIC was organized as a cooperative effort between county commissioners in Indiana and Purdue

University, with the project mirroring many modern LTAP programming activities, including the creation of publications and hosting workshops. The program would help "lend guidance and assistance to county highway officials in their problems with management, planning, and operation of county highway departments throughout the State." Later the HERPIC expanded by adding Cities and Towns, becoming HERPICC. Purdue University would go on to pilot one of the first LTAP centers in 1982 (*13*).

GROUP OF GOOD ROADS WEEK VISITORS.
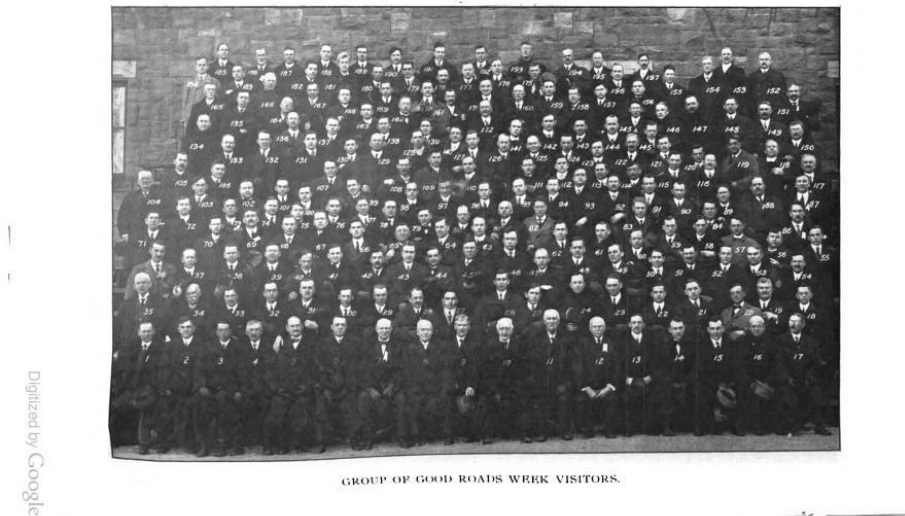
FIGURE 1 Group of Good Roads Week visitors, Cornell University, 1915.

FIGURE 2 Attendees at the Purdue Road School, 1922.

- In 1972, Dr. Jim Shamblin, a professor in the Industrial Engineering Department at Oklahoma State University, helped form the Center for Local Government Technology (CLGT). Initially, Shamblin secured a grant from the National Science Foundation (NSF) to help small towns or local governments use computer technology. Following the success of this earlier effort focused on software and computer technology, another proposal was submitted to the NSF to provide broader assistance with engineering technologies to local government units through the College of Engineering, Architecture, and Technology. From this initial funding, the CLGT evolved and eventually served as the model for the LTAP system of today. Oklahoma State University would go on to host one of the initial LTAP centers (14).

Rural Technical Assistance Program

The LTAP program was established as the Rural Technical Assistance Program, or RTAP, on Dec 23, 1981, when President Ronald Regan signed House Bill 4209 into law (*3*). Managed by FHWA, the program was allocated $5,000,000 for numerous projects related to rural transportation agency assistance, with the establishment of the state Technology Transfer Program for Local Transportation Agencies (T2 for Locals Program) being the largest. At first, 10 states hosted T2 centers in each FHWA Region as part of the initial pilot program at locations shown in TABLE 1.

Although the Cornell Local Roads Program (CLRP) was not among the initial pilot programs, CLRP did serve as a model for the early LTAP system due to its history and experience providing training and technical assistance to local highway agencies throughout New York State since 1951. Former CLRP Director Dr. Lynne Irwin explained his perspective regarding the beginnings of the LTAP system in a talk he gave in 2002 at the 7th International Conference on Low-Volume Roads (*15*):

TABLE 1 Location of Initial 10 T2 Centers

State	Location of T2 Center
Alabama	Auburn University
California	University of California Berkeley
Georgia	Georgia Institute of Technology
Indiana	Purdue University
Iowa	Iowa State University
Kansas	University of Kansas
Montana	Montana State University
Pennsylvania	Pennsylvania State University
Oklahoma	Oklahoma State University
Vermont	St. Michaels College

"It all began twenty-one years ago, on a fateful morning in January 1981, during the Annual Transportation Research Board Meeting in Washington, DC. I was sharing a room with a friend and former Cornell faculty member, Dr. Dwight Sangrey. The night before he had gone to spend the evening with the man who had been the best man at his wedding. As I crawled into bed at much too late an hour, Dwight awoke to tell me that his friend wanted to meet with me in his office in the Nassif Building at 7 am the next morning. 'He really needs to talk with you,' Dwight said.

Thus it was that I met Bob Betsold. Bob had just taken over as the head of the Office of Implementation at the Federal Highway Administration. He was responsible for distributing the many research reports and briefs that flowed from the FHWA. 'At the present time we have 50 customers,' Bob said. 'I believe that we have a lot to share with local governments, products that they really need to know about. If we can figure out how to go about it, I will have 5,000 customers.'

Bob was just moving into his new office, and it was littered with so many boxes that we had to move a few to have a place to sit and talk. 'I just moved in after the Christmas Holidays, and I have not yet had time to unpack,' he explained.

I had been the Director of the Cornell Local Roads Program for eight years at that time. During their dinner Bob had told Dwight about his idea to share the FHWA technology with local governments, and Dwight knew I did that sort of thing on a daily basis. He suggested that Bob and I try to meet while I was in town. Thus early next morning Bob and I were sipping hot coffee and talking about how FHWA could reach out to local governments.

Bob wanted to know everything about how the Cornell Local Roads Program worked. Over the previous few months I had been giving a lot of thought about new directions that I wanted to take the Local Roads Program into, and I shared my ideas with Bob. We pulled out a long yellow legal pad and I jotted down six activities that I planned to initiate. He said he wanted to 'clone' the Local Roads Program and get every other state using the model.

On the pad I wrote:

- develop a mailing list
- publish a quarterly newsletter
- provide an information service
- provide technical assistance
- teach workshops and seminars
- periodically evaluate the effectiveness

In early 1983 the FHWA sent out a request for proposals to establish the first ten Rural Technical Assistance Program centers. To my surprise the list was right there on the first page of the RFP, still in the same words and in the same order as it had been when Bob and I first met. While I am sure that I was not the only person who influenced the creation of RTAP, I am very pleased to say that I had a part in it."

— Dr. Lynne Irwin

Local Technical Assistance Program

In 1991, the audience served by the RTAP changed when the Intermodal Surface Transportation Efficiency Act widened the program's scope to include urban areas with populations over 50,000, effectively expanding the definition of who would be served from *Rural* to *Local*. The TTAP, serving American Indian tribal governments, was also established at this time.

Over the years, the program expanded in size to bring in more centers from across the nation, and by the early 2000s, there were LTAP centers in every state. The success of the LTAP program was evident from the very beginning. To quote a National Highway Institute Evaluation Report from 1984, the state centers at that time were "...fulfilling all of the objectives of the Rural Technical Assistance Program" and "....are responsive to local agency needs" (*16*).

Examples of LTAP Leadership

Over the years, the LTAP system has benefited from extraordinary leaders who have brought passion and expertise to the work of their centers. Some early LTAP leadership examples include the former Director of the Kentucky LTAP Center Patsy Anderson, the Kentucky center's first employee. Anderson was instrumental in starting the first Safety Circuit Rider Program and the first Roads Scholar Program in 1988 to provide local road personnel recognition and credentialing. Both the Kentucky LTAP Center Safety Circuit Rider program and Roads Scholar Recognition program became national models emulated by many other states today (*17*). In 2007, Anderson was awarded the National Achievement Award from the National Local Technical Assistance Program Association (NLTAPA) for her outstanding leadership, service to the industry, and profound impact on the national LTAP program.

In Iowa, the program's first Director Stan Ring was known as "Mr. LTAP." Before Iowa State University became the state's official center, Ring led the university's Civil Engineering Extension Program, which provided outreach and support to the state's transportation workforce (*18*). Ring also had an enthusiastic passion for developing an extensive library of resources to benefit the Iowa local transportation community, a legacy that endures today at the Iowa LTAP center. Even after his retirement in 1988, Ring continued to serve as the LTAP's part-time librarian for another 12 years.

LTAP and TTAP Today

Local Roads Today

Why do we need a formal state-by-state program to support local road agencies and their staff? The answer is simple: because local roads are managed by agencies that face unique challenges in accessing training and education. According to the 2017 Census of Governments, there are over 90,000 units of local governments in the United States (*19*). The sheer number of agencies that manage local roads and their wide geographic distribution mean that these agencies are often isolated from sources of support. It is also essential to consider that the workload of a local highway agency is heavy and leaves little time to travel far or stop work for training.

Local highway and public works agencies manage most of the roads and bridges in the United States. In fact, nearly 78% of the nation's roads are under local government jurisdictions (*1*). When we look at the sheer size of our local roads system, it becomes clear how vital this local network is for nearly every part of American society.

Many local roads are also classified as "rural." As stated in a 2018 Congressional Research Service report *Rural Highways* (*20*):

- According to FHWA statistics, **2.9 million miles, or 71%, of the 4.1 million miles of public-access roads in the United States are rural roads.** Roughly 45% (1.3 million miles) of this rural mileage is unpaved. More than three-quarters of rural road mileage comprises minor collectors and local roads, which are generally not eligible for federal funds.

Safety is also a significant concern on our nation's rural roads, as highlighted by NHTSA (*21*):

- According to the Census Bureau's 2019 American Community Survey, an estimated 19% of the US population lived in rural areas, and according to the FHWA, only 30% of the total vehicle miles traveled in 2019 were in rural areas. However, rural areas accounted for 45% of all traffic fatalities in 2019.

Numerous challenges face local highway and public works agencies. A considerable lack of resources has plagued local agencies for decades while, at the same time, national infrastructure has crumbled, and input costs have risen faster than inflation. There are also severe workforce challenges for local transportation. The past decade

has seen sharp declines in the overall number of people employed in local transportation, and the average age of the workforce has increased (*22*). Many potential workers also lack the training necessary to do the job. For instance, there is currently a significant gap in the number of Commercial Driver's License truck drivers available compared to the number of job vacancies. According to the American Trucking Association (ATA), in 2021, there was a shortage of 80,000 truck drivers, which the ATA estimates could increase to 160,000 by 2030 (*23*).

The importance of local roads and the challenges local agencies face may seem obvious; however, it is not always clear how the people who maintain this system get the support they need to do the job. Beyond equipment, funding, and staffing pressures, local road agencies also do not have access to quality, affordable training and education. Local roads agencies, especially in rural areas, are seldom overseen by workers with formal engineering, management, or government administration training. Many local highway department leaders are elected; most rely on experience, familiarity, tradition, and creativity to get the job done.

There are other sources of training and education beyond the LTAP system, but very few are affordable or applicable to the unique work of local highway agencies. It is also important to note that each state is unique, and within each state, there are a variety of different regions, each with its own specific set of characteristics. The uniqueness of each state necessitates a customized local roads support program that considers everything from topography and climate to local agency funding and regulation. An in-state program also helps facilitate trust and familiarity between the state LTAP center and the agencies being served.

Of course, the challenges facing local highway and public works agencies predate the formation of today's LTAP system. Looking back at the history of local roads support in the United States can give us a window into why today's LTAP program is so important, how these programs achieve success, and where the program is headed in the future.

The National System of LTAP Centers Today

Today, LTAP centers provide training, education, and technical assistance to meet the needs of local transportation agencies. LTAP centers' work includes workforce development, infrastructure maintenance, highway safety, worker safety, infrastructure design, asset management, and more. Overseen by FHWA with contracts through the state highway agencies, LTAP centers meet the needs of their own state. The TTAP

centers are directly contracted by FHWA but serve the same role to support the tribal highway community.

As an example of the scope of the work being done by the LTAP community, according to the FHWA, in 2020, LTAP and TTAP centers provided over 1,200 individual training sessions for nearly 150,000 participants totaling 3,900,000 h of training provided across the nation (*2*). LTAP funding comes from a variety of sources depending on the state and individual program, however most LTAP centers receive the majority of their funding from the FHWA and state transportation agencies.

Despite the unique nature of every LTAP center, there are some fundamental similarities between programs across the country. For instance, all LTAP centers attempt to make their training and education accessible for local agencies regardless of location, size, or proximity to an urban center. The entire national program was originally called the Rural Technical Assistance Program because congress recognized the need to provide resources, training, and assistance to the sprawling road networks of American rural areas. Many LTAP services are free or delivered at the lowest cost possible to ensure that communities can connect with the resources they need.

Another common feature of LTAP center programming is always an effort to create resources, training, and education that is practical and relevant to the people in the field. For instance, LTAP centers will often focus on teaching low-cost solutions to common problems that affect a wide variety of local agencies. LTAP centers are always looking to innovate and better connect with their audiences. For example, many LTAP centers recently added webinars and other online or digital training offerings developed to meet the needs of a modern local transportation workforce in the internet age. Overall, the goal of every LTAP is to meet a need that is not being met audience of local roads agencies.

LTAP centers also deliver services to communities by establishing meaningful national, state, and local partnerships. At the federal government level, the FHWA's Center for Local Aid Support is an essential partner for all LTAP centers nationwide and provides overall program oversight and management. Other critical national partners include:

- The National Association of County Engineers (NACE),
- The American Public Works Association, and
- American Association of State Highway and Transportation Officials (AASHTO).

Finally, as a family of programs spread out across the nation, one of the most significant sources of strength LTAP centers has come from each other. To that end,

the NLTAPA exists to help states share knowledge and resources to benefit all. Under this association, LTAP centers have a central organization to coordinate, exchange best practices, share training expertise, and cooperate on developing new ways to improve local highways (*24*). Many states benefit from the expertise of other LTAP centers in the form of guest instructors providing training that would otherwise have been unavailable.

The LTAP Model for Providing Local Transportation Support

Many qualities make the LTAP system unique and effective, but perhaps the most important is that every LTAP program reflects the communities they serve. LTAP programs are not standardized "cookie-cutter" bureaucracies but are dynamic and constantly evolving. LTAP centers work in and around the local highway agencies they serve. LTAP staff listen to their communities needs, solicit feedback, and formulate training and education that meets the real-world demands of local highway agencies' jobs. Often, the leadership and staff of LTAP programs come from the local highway management and engineering sectors within the states they serve.

   Another way LTAP programs uniquely serve their communities is by being as accessible as possible. LTAP programs travel across their states to provide training, offering all LTAP services for free or at the lowest cost possible. LTAP centers go to great lengths to identify means to provide for those in the local highway community with the greatest need. Many centers today have established technology lending programs and secured grants for equipment, and all centers provide free technical assistance in some form.

   LTAP centers also provide workforce development that is accompanied by recognition. Many LTAP programs fill a critical need for training by establishing curriculums that often culminate in certification and recognition. Programs referred to as "Road Master" or "Road Scholar" programs establish critical pathways for education, certification, and recognition that may otherwise be denied to local highway personnel either due to the prohibitive cost of other educational channels or because of geographic isolation.

   How would one go about creating a system of support like the modern LTAP program? One of the essential pillars of a successful LTAP-like program would be to create community buy-in by working with local, well-established organizations already present in a particular state or locality. Another vital element in the success of any LTAP-like program would come through taking the time to listen and be responsive to the community being served. Understanding how local agencies do their jobs, the

limitations of their resources, and the challenges they face daily helps LTAP centers craft programming that agencies can utilize.

Another important aspect that any program developed to be similar to the LTAP system would want to incorporate would be a confederation-support model of organization. While every state and center is unique, LTAP programs themselves need new ideas and resources. NLTAPA is the national organization that fulfills that role, along with a strong partnership between LTAP centers and the FHWA Center for Local Aid Support. LTAP centers today share resources, neighboring states exchange trainers for limited course offerings, and national conferences allow LTAP centers to share ideas and brainstorm solutions to common challenges. The overall system is strengthened by making sure that no LTAP program operates in isolation.

Finally, LTAP programs serve as a central hub or clearinghouse to bring together ideas and assets gathered from multiple sources from widely dispersed areas of states and from across the nation. LTAP centers understand that they alone cannot fulfill the entire needs of the communities they serve, but they can become a gathering place to organize and better distribute resources others develop to the people who need them. Many good ideas have spread across the country thanks to LTAP programs collecting and disseminating these resources. Examples include safety circuit rider programs, better ways to conduct work zone and flagger training, asset management programs for local agencies, and many more.

Today's LTAP System Is More Important Than Ever

Today there is a greater need than ever to provide quality training, education, and technical assistance to local transportation agencies. Workforce pressures like increasing retirements within local agencies, difficulty recruiting workers, critical skills gaps among new workers, and rural flight are all rising. On the regulatory side, state and federal mandates strain local departments without always providing the related resources to facilitate compliance.

Of course, the nation was given its own education on the importance of local transportation workers during the COVID-19 outbreak. Regardless of viral transmission rates or public health mandates, these essential workers continued to perform their duties while often assisting neighboring municipalities when workers fell ill. LTAP programs rose to the challenge by pivoting their training to online formats, creating COVID-specific guides, and working with partners to connect agencies with even more resources to assist them as the pandemic raged.

There are many reasons to be optimistic about local roads and LTAP programs' role in supporting them. Many LTAP centers are bringing to light the innovative culture of the local highway and public works community through programs like the national Build a Better Mousetrap Competition. Local agencies submit and share their unique solutions to everyday problems in this innovation competition (*25*). LTAP programs and NLTAPA are also consistently cultivating new local, state, and federal partnerships. By working with in-state associations, partnering with universities, and collaborating with state transportation agencies, LTAPs are magnifying their efforts to reach even more of their audience.

In recent years, the LTAP system has grown by leaps and bounds to open new sources of digital training to reach even more local agencies nationwide. LTAPs have played a role in channeling online training resources like the AASHTO TC3 modules (*26*) and working with FHWA to offer those courses to local agencies for free (*2*). Many LTAP programs have developed their own online courses by incorporating webinars and learning management systems into their regular service offerings (*27*). Also, within the LTAP system itself, new generations of training leaders are joining the ranks and becoming more involved, thanks to the growing efforts of NLTAPA.

Moving into the future, LTAP centers will also be on hand to help local agencies navigate the challenges and opportunities posed by a nation facing a crisis of aging infrastructure. Recently passed state and federal legislation will infuse communities with much-needed resources to upgrade, replace, or better maintain their infrastructure. Educating local agencies about the availability of these resources and how to access them will be an essential part of future LTAP work. Also, LTAP centers are helping communities connect to information and education about preparing for a future where infrastructure must be more resilient to global climate and weather changes.

Local agencies across the nation will always face headwinds when it comes to ensuring our roads remain safe and reliable for all. Local highway and public works agencies must be supported and provided with the necessary resources to get the job done. As our nation's transportation system grows and evolves, it will be important to ensure that local agencies are not left behind. LTAP programs will continue to lift up the agencies they serve by providing quality training, education, and technical assistance now and for many years to come.

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Author Contributions

The authors confirm contribution to the paper as follows: study conception and design: David Orr and Adam Howell; data collection: Adam Howell and David Orr. Author; analysis and interpretation of results: David Orr and Adam Howell; draft manuscript preparation: First draft by Adam Howell with major edits and review by David Orr. All authors reviewed the results and approved the final version of the manuscript. The authors do not have any conflicts of interest to declare.

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Development of Sustainable Research Capacity Building of the Research and Development Unit for Myanmar Rural Road Sector

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The Research and Development Unit (RDU) was set up by Research for Community Access Partnership, funded by the United Kingdom's Department for International Development (DFID-UK) and established in Myanmar in November 2021. The laboratory director and manager were trained by the Council for Scientific and Industrial Research (CSIR, South Africa). The purchase and installation of laboratory equipment was supported by the World Bank program.

RDU conducts key activities to succeed in the operation and management of RDU, such as identifying and implementing research projects, material testing, and capacity-building training to contribute to the sustainability of the rural road system in Myanmar. The establishment of RDU is to serve both the public and private sectors' rural road engineering and transportation needs through the development, implementation, and distribution of best practices by means of new knowledge, capacity development, and economy in the rural road sector.

Myanmar is an agricultural country where organic soil is found in most parts. The soft soil, which is abundant in Myanmar because of the alluvial minerals deposits, is a critical problem for rural road implementation. Myanmar is located in a tropical region; thus, the rainfall intensity is high in the delta, coastal, and hilly regions and low in the dry zone in the middle of Myanmar. To be a sustainable RDU, it occasionally holds workshops to train the research staff with hands-on experiments using locally available materials and stabilization measures for the soft soil ground on rural roads.

Methodology

A reliable rural road network is instrumental in developing and improving the quality of life in rural communities. Based on the urgent need of the country, RDU has identified and prioritized the projects and undertaken the research work. With a limited budget, RDU started with only 11 staff, including the RDU director in 2021. The number was increased to 20 in 2022 so that more staff could be delegated with more responsibilities based on the research priorities. Now, the RDU staff are divided into two or more groups or teams, each addressing specific fields of material testing such as bitumen testing, soil testing, aggregate testing, concrete testing, and field testing. The RDU laboratory is led by the trained director of RDU and assisted by the trained senior laboratory manager. A deputy director leads two laboratory researchers to provide initial analyses of the results and makes substantive suggestions for improvements in techniques and interpretations. A senior laboratory manager ensures that the RDU laboratory operates smoothly by assisting laboratory engineers and technicians, monitoring equipment, and maintaining protocols and standards. Six laboratory engineers and seven technicians operate laboratory testing for the aggregate and cement room, the bitumen rooms 1 and 2, the concrete room, the soil rooms 1 and 2, the tensile machine room, and the sample receiving and preparation area. Two staff members take respective responsibilities in financing and the equipment store. For field testing, the equipment for pavement roughness, pavement strength, and stability of pavement are available in the RDU laboratory.

RDU was the very first research unit established in Myanmar. Capacity-building training on research work, laboratory testing, and quality assurance and quality control work are essential to foster the development of key research skills, including the implementation of research projects. Each RDU staff member has been trained on how to formulate the research problems appropriately. At the same time, this will add value to sustain existing professional engineering knowledge within the Myanmar research community, and it is sure to support the growth of RDU with well-trained staff. The trained RDU staff from the Department of Rural Road Development (DRRD) lead the contractors from private firms to the development of their knowledge and skills, which are required to conduct technical audits, inspection checks, and performance evaluations regarding rural road projects effectively and efficiently. RDU has held various workshops focusing on topics: research methods and design, fundamentals of research development, short courses on laboratory testing and field testing of road construction materials, and low-volume rural road pavement design. One of the research works by using locally available materials on low-volume rural road construction is the application

of soil stabilization measures in the delta region and flat areas, after which pavement performance work was monitored on these projects.

The *Design Manual for Low Volume Rural Roads in Myanmar*, published in 2020, was funded by DFID-UK in close cooperation with RDU, DRRD. This manual has been serving as guidance on the rational, appropriate, and affordable design for low-volume rural roads in Myanmar.

Results and Findings

Two soil stabilization works as research projects of low-volume rural roads, where the weak subgrade soil was dominant, have been successfully conducted under the RDU laboratory: research trial I Okkan–Thakyarsat road project, which is 10 km long, situated in the Yangon division, and research trial II Ma Yan–Kyunthanaung village road project, which is 1 km long, situated in the Ayeyarwady division. These research works evaluated the localized materials' capability and the agents' effectiveness to improve the existing ground in order to support the pavement structure and sustain its road quality. To obtain better results, the necessary lab tests for two trial stabilization works using different proportions of the locally available soils were conducted to satisfy the required strength of the subbase layer. The suitable soil samples along the project roads, to mix with the stabilized agent for investigation, were collected from borrowed pits and transported to the RDU laboratory for required properties tests.

The collected soil sample for the trial I project was tested in the RDU laboratory, and the results showed the soil type as sandy silt and gravel with the value of CBR (4 days soaked) 5.12% only. In this case, the soil with CBR (4 days soaked) value of less than 10 was not suitable to construct the subbase layer. Therefore, three trial mix ratios were carried out by using lime as a stabilizing agent mixing with different ratios of borrowed soil, which has to be gravelly and lateritic soil. Among those three mix ratios, the mix of 35% borrowed soil, 60% of gravel and lateritic soil with lime 5%, which met the design strength value of soaked CBR 30%, was determined to construct the subbase layers. Similarly, the results of the collected soil samples (trial test II) showed silty clay trace sand, and the value of 4 days soaked CBR was only less than 3%. In order to meet the requirement of sub-base layers, there were three trial mixes by using lime with different ratios of borrowed soil and shingles. Among these mixes, the mix of 34% borrowed soil and 60% shingle in Ayeyarwady with 6% lime, which got the value of soaked CBR 25% to construct the subbase layer.

After completion of construction, monitoring performance was established on these two projects by carrying out the visual assessment of the road, drainage factor measurement, and traffic count. These trial sections will be measured every 6 months to quantify the extent and rate of deterioration. Consequently, it was found that the monitoring result of these two project roads had reasonable performance after 3 years in service. Although there is a lack of maintenance of the drains, the performance of the drainage system of both project roads is adequate. Both roads, low-volume roads, were displaying minor cracks.

The workshops were conducted in RDU, as well as in these regions and other regions, to demonstrate the effectiveness of using localized materials with the stabilization measure to meet the requirement of a rural road sub- base quality. The RDU laboratory has performed material testing of road projects from DRRD and the private sectors. It supports low-cost solutions for rural roads by understanding the effectiveness of local materials and natural resources. The financial support for laboratory testing is received from DRRD road projects and also earned from private sector road projects, one of the main funding sources for RDU. The RDU laboratory is operated with limited equipment to conduct soil testing, aggregate testing, bitumen testing, and concrete testing. For instance, only the basic properties tests of bitumen can be done at RDU due to the lack of advanced bitumen testing equipment. In the future, more equipment is required to deal with increasing workloads and to qualify the research projects.

Challenges of RDU

Although RDU is operational, there are numerous challenges, such as a lack of adequate vehicles for field studies and operations, as well as a center for information and communication technology. In the future, an information center will be established to meet the technical information requirements of RDU researchers, the DRRD, and the entire Myanmar road sector. The RDU still needs long-term guaranteed funding to support its operations to ensure the sustainability of its capacity building efforts. The role of all relevant stakeholders is also important in order to receive long-term financial support of the RDU's continuous development in fully addressing Myanmar rural road sector challenges.

Conclusion

The properties of soil differ depending on the terrain and region. Weak soils in the Yangon and Ayeyarwady regions have been successfully stabilized for the construction of subgrade and subbase layers. In spite of that, salt intrusion has been a problem in these areas. Some rural roads failed before their design life due to this problem. In some cases, the road failure occurred shortly after the road construction was completed. It was caused by a number of variables including soil conditions, adverse moisture conditions, effect of climate, quality of materials, and uncertainty in traffic forecasting and construction error. Therefore, further research is essential in the implementation of rural road construction so that road management can be made systematically within a limited budget and while receiving technical cooperation and support from other international organizations. The undertakings of RDU in providing capacity-building training, sharing research knowledge through workshops, and conducting innovative research projects, will benefit not only the RDU staff but also Myanmar's rural road engineers.

Furthermore, the RDU should have strong ties with international organizations such as research institutes (the CSIR in South Africa and the joint industry program) and international universities to promote long-term sustainability of rural road research capacity.

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Overview of Applications Used in MnROAD Low-Volume Roadways from Past to Present
What Do They Give to Future Engineers?

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A task force composed of Minnesota Department of Transportation (DOT) engineers and officials, Federal Highway Administration (FHWA), the Strategic Highway Research Program (SHRP) administrators, industry representatives, and university experts was created in the 1980s to explore the idea of building a Cold Regions Pavement Research Test Facility. This led to the construction of the Minnesota Road Research Facility (MnROAD) to test pavement concepts in a real-life scenario with the understanding that this would help Minnesota DOT and its partners to better understand how to build the best roads for the state's budget.

Since the planning stages of MnROAD, Minnesota DOT has recognized the need to conduct experiments and collect data on low-volume roads. The Low-Volume Road (LVR) section addressed these needs by filling knowledge gaps within low traffic volume mainline systems of Minnesota for local municipalities and counties while complimenting research gained through the high-volume Mainline. The Local Road Research Board (LRRB) has been a valuable partner to MnROAD since its opening, both as a consistent source of funding and as a basis for project initiatives. All projects below received funds from LRRB or derived their origins from LRRB research initiatives.

To simulate low-volume traffic MnROAD uses a five-axle, 18-wheel tractor-trailer semi to provide the loadings with a gross vehicle weight of 80 kips, 5 days a week on the inside lane for 8 h a day (80 laps a day approximately), respectively. The outside lane is left without traffic to examine how only environmental effects impact pavement performance over time.

MnROAD relies on pavement instrumentation and field pavement performance to collect important pavement design parameters data under traffic load. Sensor types include vibrating wire strain gauges, moisture sensors, thermocouple trees with

watermarks and thermocouples, portland concrete cement (PCC) joint opening gauges, maturity data loggers, and fiber-optic sensors. Given the large number of sensors collecting data and the regular assessments of the pavement structural and environmental conditions, MnROAD's depth of knowledge exceeds that of any other full-scale facility.

While this familiarity is useful for all test sections, for the low-volume road test sections it is especially significant, as very little full-scale, closely controlled observations have been done on LVRs up until MnROAD's opening for operations. The MnROAD LVR is in Albertville, 40 mi Northwest of St. Paul, north of the MnROAD I-94 WB Mainline as shown in Figure 1.

Methodology

Since finishing its construction in 1993, MnROAD has led research in areas that include seasonal load policies (winter and spring), mechanistic-empirical design (asphalt and concrete), asphalt binder grading, low temperature cracking reduction, improved pavement maintenance operations, development and calibration of a mechanistic-empirical design guide, implementation of innovative construction technology, improved preventive maintenance techniques, effective use of recycled materials, development and refinement of techniques for cost-effective pavement rehabilitation, understanding of pavement surface characteristics, and continued support of many non-pavement research areas.
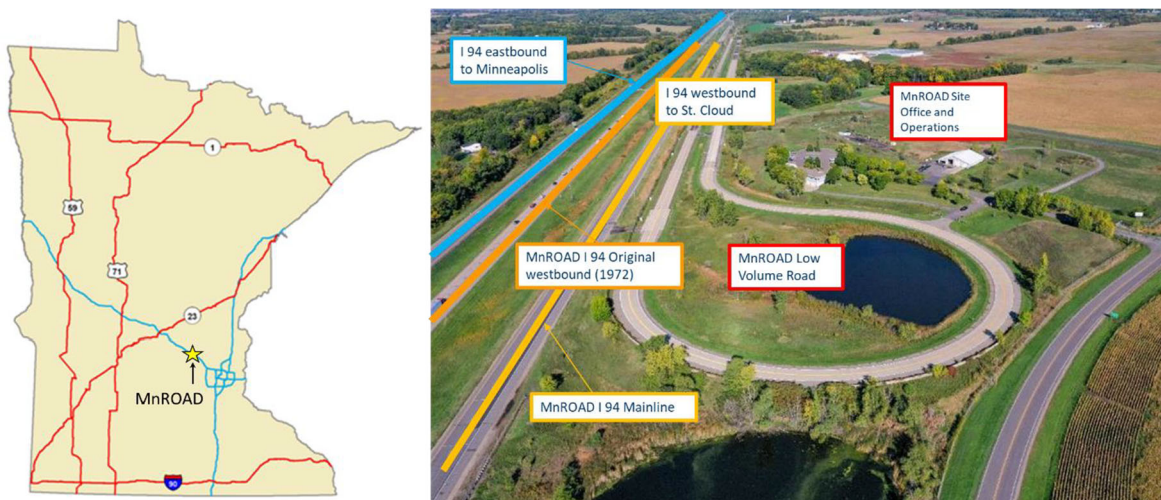
FIGURE 1 Minnesota Road Research Location and Facility.

The research done at MnROAD on LVRs, has been extremely valuable in its influence on local design and maintenance practices, knowledge of materials, and ability to minimize costs in improving local roadways.


Findings

These nearly 30 years of extensive research which are shown in timeline format within Figure 2 have critically benefited Minnesota and its partners in quantifiable and invaluable ways for both drivers and the pavement engineering industry. MnROAD has consistently shown a benefit-to-cost ratio greater than 1.0. These numbers do not consider the additional benefits of educating future pavement engineers, learning what not to do, demonstrating and highlighting technologies road owners can use today.

One of the first set of LVR materials properties data that were used in the mechanistic–empirical design program was collected at MnROAD LVR. This helped pave the way for the development of MnPAVE which determined that current at the time concrete and asphalt designs were structurally over designed (1–2).

Properly applying seasonal load restrictions was a major source of pavement deterioration. Data collected at the MnROAD LVR was used in the creation of Minnesota spring load restrictions and winter load increase guidelines (3).

Temperature profile data collected at MnROAD showed that lower non-wear layers do not receive the extreme high–low temperatures experienced by surface wear layers. This led to the use of different asphalt binder grades between wear surface and non-wear base materials and translated into asphalt binder grade cost savings because non-wear layers need a less expensive asphalt binder. Related to cold weather states and asphalt binder grade recommendations, data collected at MnROAD demonstrated that performance grade (PG) 58-28, PG 58-34, and PG 58-34 asphalt binders do not perform the same. PG 58-28 performed as expected while PG 58-34 performed better than expected and PG 58-40 experienced significant distress and, therefore, opposite to what is expected at low temperature perfomed the worst. This is the reason why PG 58-34 is recommended for wider use in the state of Minnesota.

MnROAD experience with aggregate roads found a strong relationship between washboarding and number of truck passes and forensic cross-sections revealed that rutting primarily occcurred in the aggregates and not the sugrade. It also proves that aggregate gradations are not reliable predictors of performance for aggregate roads (4). The research of aggregate roads also led to the understanding that the rate of freezing and thawing under these types of roads was much different than under
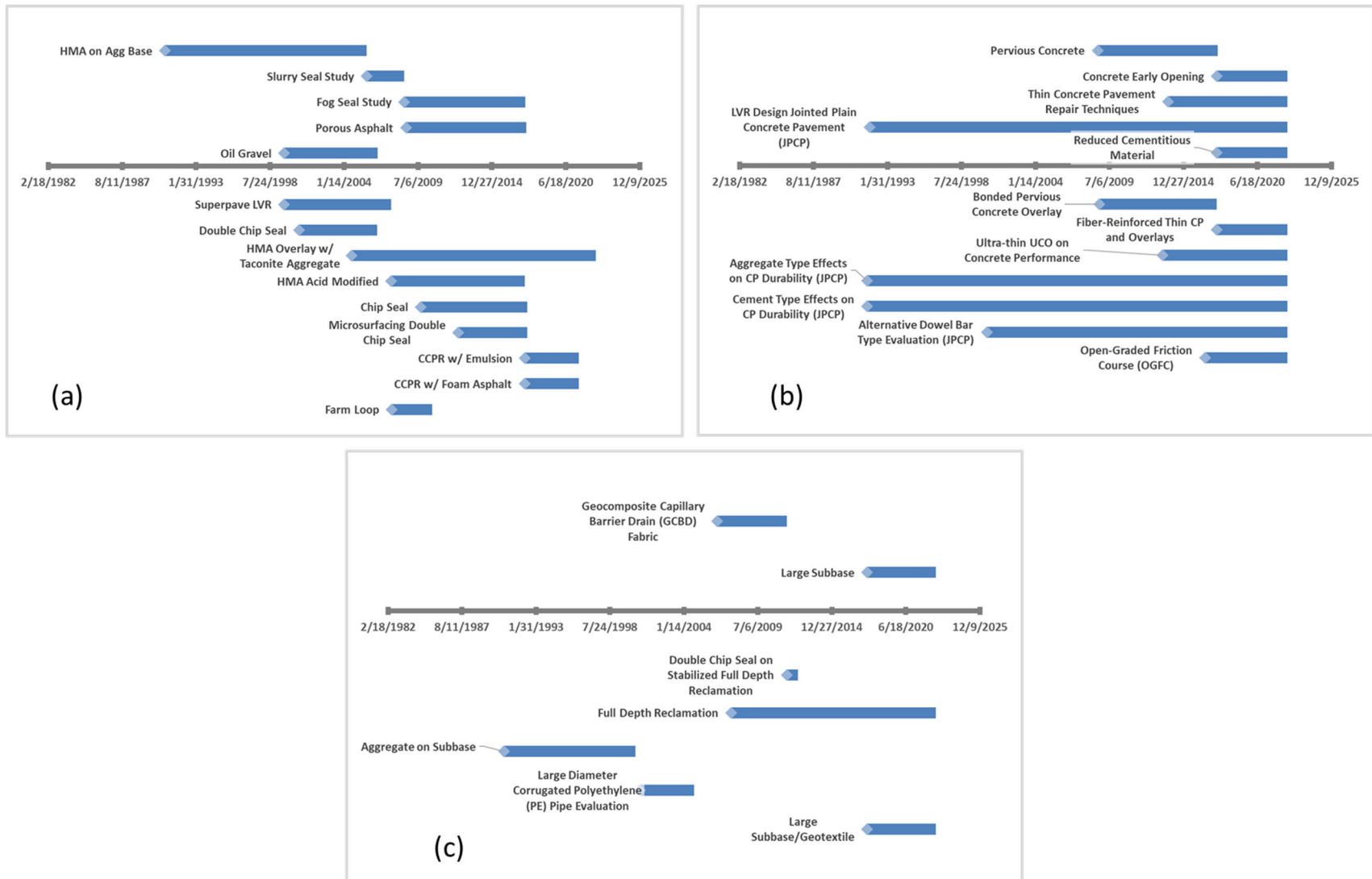
**FIGURE 2 Timeline of applications used on MnROAD LVR within
(*a*) flexible, (*b*) rigid, and (*c*) pavement foundation.**

hot-mix asphalt (HMA) sections. The subgrade under aggregate sections froze approximately 4 to 5 days sooner than the subgrade below the HMA. Furthermore, the subgrade under the aggregate sections took between 11 and 35 days longer to thaw than the subgrade under the HMA (*5*).

Emulsified oil gravel research was conducted using 3 test sections at MnROAD. At the time of this study, it was a unique experience to test tracks and it came through fruition through a partnership with the Finnish National Road Administration (FINRA) which has been building roads with oil gravel with long life and low amount of cracking since the late 1950s. Two of the three test sections performed very well and did not experience as much thermal distress as their HMA counterparts (*6*–*8*).

MnROAD has been studying thin and ultrathin concrete layer overlay of asphalt pavements, known as whitetopping, since 1993. Using MnROAD wide array of load response sensors to monitor the performance of these thin concrete slabs in real-time, the results of this effort helped expand the experience engineers had on whitetopping nationally (*9*–*11*).

Other research areas where MnROAD LVR test track has provided a lot of information include innovative contruction, preventive maintenance, recycled materials, and rehabilitation. Implementation of innovative construction technologies and methods have helped reduce construction cost and time. Research on preventive maintenance techniques at MnROAD has been shown to help maintain our current pavement investments. Projects using recycled materials at MnROAD, including taconite aggregates, investigated the performance of pavements. With most states looking into future construction costs versus needs, MnROAD experience with rehabilitation techniques has contributed to develop and refine these techniques providing information on rehabilitation for cost effective pavement.

MnROAD LVR research has also contributed to develop techniques for smooth, quiet, durable, and skid-resistant pavements. This has helped increase road safety and reduced the necessity of building costly noise walls due to the reduction in noise from the pavement and tire interaction.

Conclusions

The research done at MnROAD on LVRs has been extremely valuable in its influence on local design and maintenance practices, knowledge of materials, and ability to minimize costs in improving local roadways.

One major benefit of the LVR test cells at MnROAD is the fact that no other test track investigates LVRs to the extent of MnROAD. Along with its experience in areas such as cold-regions research (including low-temperature cracking) and whitetopping, MnROAD has established itself as an important facility in furthering LVR research and construction techniques.

Much of the near future research is focused on the current high priority topics of sustainability, alternative materials, and intelligent construction technologies for both asphalt and concrete pavement construction and maintenance. As the global sustainability landscape shifts to the emphasis of study and precautions for reduced environmental impact, MnROAD seeks to lead the industry in that direction.

Proposed projects in line with this initiative will test ideas for new materials in pavements and see its impact on the surrounding ecology. Revised pavement material recycling techniques and use of technology hope to reduce dependence of virgin materials throughout the pavement structure and limit waste products.

Minnesota DOT and MnROAD staff are currently in the process of developing benefits summary of Phase III results and send out a call for research construction ideas for a new reconstruction phase of MnROAD LVR.

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Outcomes of the 2019 12th International Conference on Low-Volume Roads

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The 12th International Conference on Low-Volume Roads was held in Kalispell, Montana, September 15–19, 2019, and was sponsored by the Transportation Research Board and co-sponsored by the US Forest Service and US Department of Agriculture with local hosts the US Forest Service and the Western Transportation Institute at Montana State University. The conference welcomed 250 participants from 22 countries who experienced low-volume roads (LVRs) in northwest Montana firsthand. The conference provided 27 sessions covering 104 presentations, six hands-on workshops, and a field tour highlighting demonstrations of a variety of LVR management tools.

The purpose of this abstract is to highlight identified research ideas, themes and needs that were spawned at the conference and outcomes that were later produced by the LVR Committee.

Methodology

Information presented here was capture at the 12th International Conference on Low-Volume Roads by the Committee Research Coordinator and Conference Planning Committee. Information sources include direct feedback from conference attendees and presenters, on sight poll responses, and on sight and post-conference surveys sent out via email.

Findings

The following section summarizes findings and key outcomes from the conference and summarizes activities conducted by the LVR Committee to support the conference outcomes.

Research Ideas, Themes, and Needs

The following research ideas and needs were provided by conference attendees:

- Create simple format lessons learned guide on gravel road design and maintenance.
- Highlight the Transportation Research Board (TRB) LVR conference and research to encourage attendance at future LVR events.
- Create synthesis' of LVR research to avoid reinventing the wheel.
- Expand the focus to include bikes, pedestrians, transit, and other transportation modes.
- Foster discussion on climate changes mitigation on LVRs, including resilience and mitigation techniques.

Key Outcomes

The conference was focused on providing relevant training and usable tools to attendees. Conference attendees described the event as informative, educational, and inspiring. Figure 1 highlights key words that were used by conference attendees to describe the event.

Outcomes from the Conference

The following publications were developed as a part of the 12th International Conference on Low-Volume Roads.

- *Transportation Research Circular E-C248: 12th International Conference on Low-Volume Roads* (2019) is the compendium of papers and extended abstracts developed for and presented at the conference. A key modification to the conference was the

FIGURE 1 Key words used by conference attendees to describe the event.

acceptance of extended abstracts to encourage more practitioner-based sharing and participation and to encourage presentation of current findings.

- *Transportation Research Record: Journal of the Transportation Research Board, Vol. 2673, Issue 12.* Transportation Research Record 2673 (2019) published the highest-quality papers from the conference.

- The Low-Volume Road Pooled Funded [TPF-5(379)], administered by Iowa Department of Transportation, developed, with participation from the transportation departments from Alaska, Illinois, Iowa, Kansas, Louisiana, Michigan, Utah, Virginia, Wisconsin, and Ohio and Ohio County Engineers, *Transportation Research Circular E-C272: Technology Exchange on Local Roads Bridge Programs* (Veneziano and Goetz, 2020), which is based on two sessions at the conference on local bridge programs in the United States.

LVR Committee Activities to Support Themes Identified at the Conference

Outcomes from the 2019 conference were used to focus LVR Committee efforts including developed research needs statement, webinars, and TRB Annual Meeting sessions which are summarized below.

The following Research Needs Statements (RNS) developed or in development by AKD30.

- Best practices for non-vehicular safety on rural roadways.
- Best practices for low-volume but heavy loads (such as agriculture, logging, mining).
- Role of LVRs on road network resilience.
- *Guide to Manual of Uniform Traffic Control Devices* compliance for LVRs. What is a must versus what is optional.

The following webinars were held through TRB and sponsored or co-sponsored by AKD30.

- Specifying Clay and Gravel for Road Surfacing, December 2020.
- Chemical Treatments on Low-Volume Roads, March 2021.
- Evaluating and Rating Unsurfaced Roads, August 2021.
- Giving Low-Volume Roads a Longer Life with Geosynthetics, November 2021.
- Progress Toward More Resilient Pavements, November 2021.
- On the Edge—New Applications and Safety Outcomes of Edge Lane Roads, February 2022.
- Using Buried Bridge Techniques to Accelerate Bridge Construction Processes, March 2022.
- Optimizing Unpaved Road Design with a Materials Blending Tool, July 2022.

The following activities were organized by the LVR Committee and held at the 101st Annual Meeting of the Transportation Research Board.

- Low-Volume Road Improvements under the Great American Outdoors Act Workshop
- Trending Issues in Low-Volume Roads Poster Session
- Safety Studies on Low-Volume Roads Lectern Session

The following activities have been organized by the LVR Committee and held at the 102nd Annual Meeting of the Transportation Research Board.

- Low-Volume Roads Sustainable Pavement Design and Rehabilitation Methods Workshop.

- Current Topics on Low-Volume Roads Lectern Session.

Achievements

In 2022, the TRB Technical Activities Council awarded the LVR Committee the Blue Ribbon Award in the category of Implementation: Moving Research Ideas into Transportation Practice for the effort put forth in the 12th International Low-Volume Roads Conference and Workshops. Specifically, the conference focused on providing workshops that gave attendees a tangible tool to take back to the office.

Conclusions

Input from the 2019 12th International Conference on Low-Volume Roads participants has been used to support the direction and action taken by the LVR Committee. This is demonstrated in the RNS, the webinars proposed and hosted by the committee through TRB, and events hosted and planned for the TRB Annual Meeting including workshops, lectern, and poster sessions. Future events, such as the 2023 13th International Conference on Low-Volume Roads, a partnership between TRB and Iowa Local Roads Research Boards, will work to continue these LVR Committee efforts and identify new research ideas, themes, and needs.

Acknowledgments

The authors would like to acknowledge the 12th International Conference on Low-Volume Roads planning committee – Shawna Ballay, Steve Bloser, Alex Campbell, Andrew Ceifetz, Gary Danczyk, Asif Faiz, Maureen Kessler, Keith Knapp, Bethany Kunz, Glen Legere, Ricky Mitchell, and David Orr; and conference attendees and presenters. The authors would also like to recognize Dana May from WTI.

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PAVEMENT TESTING AND ANALYSIS

Revisiting Field Performance of Cold Central Plant Recycled Test Sections at MnROAD's Low-Volume Road

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In 2017 four test sections containing a Cold Central Plant Recycled (CCPR) layer (Test sections 133, 233, 135, and 235) underneath a surface treatment were constructed on the MnROAD Low-Volume Road (LVR). Two types of CCPR mix designs were constructed: Test sections 133 and 235 contained an emulsified mix, and test sections 135 and 233 contained a foamed mix. Each CCPR mix design (foam and emulsion) had two types of wearing surface layers [double chip seal and 1.5-in. hot-mix asphalt (HMA) overlay, Thinlay] to create the four test sections. The rest of the pavement structure beneath the newly constructed CCPR was left in place and consistent throughout all four test sections with the remaining granular base over the existing clay subgrade. This paper presents a revisiting of the field performance of the CCPR test sections at the MnROAD LVR. To do this, extensive field data were collected over approximately 3 years that included visual distress surveys, ride quality, rutting, and falling weight deflectometer testing. Field study determined that both CCPR mix designs (foam and emulsion) performed similarly and that the Thinlay HMA layer outperformed the double chip seal related to rutting, but both surfaces are considered valid alternatives for future applications depending on the traffic volumes expected.

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

Light Weight Deflectometer Evaluation of Low-Volume Road Structural Deterioration Under Rapidly Increased Traffic Patterns

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When evaluating low-volume road (LVR) condition, there is not a tremendous emphasis on structural capacity since deterioration is typically due to environmental factors. However, in cases where LVRs experience rapidly increased traffic loading, structural condition becomes important. The research objective was to determine whether the light weight deflectometer (LWD) can be used as a structural evaluation tool for LVRs. Specifically, this study explored the capability of LWD equipment in evaluating the three major factors that influence flexible pavement structural capacity: load-induced deterioration (e.g., rutting and cracking), soil moisture, and asphalt temperature. Four full-scale flexible pavement test items were constructed with varying base course materials and layer thickness typical of LVR. Instrumentation was installed during construction to monitor moisture content, temperature, and ambient weather conditions. Accelerated traffic was applied with a four-axle military truck. A total of 10,000 cumulative vehicle passes were completed. Falling weight deflectometer (FWD) and LWD testing equipment was utilized to measure pavement structural condition at selected traffic intervals. The LWD was shown applicable for LVRs experiencing accelerated traffic. The LWD is portable and is more efficient for LVRs when high rut depths are permitted. The LWD tracked trends of the FWD; therefore, the LWD based on these experiments can be recommended for use in lieu of FWD to assess structural condition of LVRs.

Alternative Installation Procedure for Asphalt Strain Gauges in Low-Volume Road Pavements

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Asphalt strain gauges (ASGs) are a type of instrumentation used to monitor dynamic strains induced by traffic loading of flexible pavements. ASGs have widespread use in accelerated pavement testing (APT) experiments (*1–3*), instrumentation of roadways (*4–6*), instrumentation of runways (*7*), and structural health monitoring of pavements (*8*). The most common type of ASG used is an H-bar gauge, which consists of a semi-rigid frame with strain gauges installed in either a full Wheatstone bridge or a quarter Wheatstone bridge configuration (*9, 10*).

ASGs are typically installed at the bottom of a hot-mix asphalt (HMA) layer during construction so that the HMA is compacted around the ASG, thus ensuring full embedment of the ASG into the HMA. This is important to ensure that the ASG is fully engaged with the HMA layer and will accurately measure dynamic strains during traffic loading.

The most common ASG installation procedure (sometimes referred to as the mound method) involves five main steps: (1) adhering the ASG to the top of the base course, (2) covering the ASG with a mound of loose HMA, (3) applying light hand compaction, (4) placing the HMA mat with an asphalt paver, and (5) roller compacting the HMA mat. For successful installation of ASGs by this method, it is critical to consider the paving sequence such that the asphalt paver is aligned not to impact the ASG directly with its tracks or wheels (*7, 11, 12*). Note that other ASG installation procedures exist, including block-out or trench-cut methods (*13*) and even coring of HMA after construction to epoxy strain gauges to the reinstalled core (*14*).

For most typical ASG installations by the mound method, there is adequate width of paving to allow for keeping paver tracks or wheels from directly driving on the ASG. However, for a single narrow lane paving, such as is typical for a low-volume road (LVR), the wheel-path position where the ASG must be installed is precisely aligned with the location where the paver wheels or tracks must be installed. This problem was recently encountered during construction of an LVR test section APT experiment.

To overcome this challenge, an alternative ASG installation procedure was trialed. The objective of this paper is to describe the alternative procedure developed and

demonstrate its successful implementation. After ASG installation, light and heavy truck traffic was applied to the pavement while ASG signals were monitored; ASG response was satisfactory. Post-APT forensic extraction of the ASG showed good embedment into the HMA layer. Density measurements next to the ASG showed that density was somewhat lower compared to the overall pavement.

Methodology

Pavement Cross Section and Materials

A segment of roadway representative of a typical LVR was constructed as part of an APT experiment. This roadway segment was 152.4 m long, and the HMA paving was 3.35 m wide. A gravel base course 100 mm thick was placed and compacted. Above this, an HMA layer with target thickness of 38 mm was placed and compacted.

The native subgrade was classified as a low plasticity clay, which was compacted prior to placing the base course. An unprocessed, naturally occurring local source of gravel was used as the base course, which was classified as sand with clay and gravel. This material does not meet normal specifications for a roadway base course and would be normally classified as a subbase material. However, because many existing LVRs have been constructed with this type of material, it was used for this LVR roadway segment.

The HMA used for construction consisted of a blend of 55% crushed gravel, 25% natural sand, and 20% recycled asphalt pavement (RAP). The fine-graded, dense aggregate blend had a nominal maximum aggregate size of 9.5 mm, 46% passing the 2.36-mm sieve and 4.9% passing the 0.075-mm sieve. The unmodified asphalt binder was performance grade 67-22. As produced, the mixture total binder content was 5.6%, including a 1% contribution from the RAP.

The ASGs used for the project were model KM-100HAS, obtained from Tokyo Measuring Instruments Laboratory (Tokyo, Japan). Their measurement range was ±5,000 microstrain using a 100-mm gauge length and a 350 Ω full bridge design; the rated operating temperature range was –20°C to 180°C (*15*). These ASGs have been successfully used in other roadway projects, where the instrument survivability was found to be greater than 97% after 2 years and about 85% after 5 years (*13*).

In each wheel path, an ASG was installed parallel to traffic (longitudinal), and another was installed perpendicular to traffic (transverse). In total, four ASGs were installed in the pavement (two in each wheel path).

Asphalt Strain Gauge Installation

The wheel-driven asphalt paver (CAT® Model AP1000F) used for construction had a minimum paving width of 3.0 m, leaving only 0.35 m of potential side-to-side adjustment of the paver to accommodate placement of the ASG. However, the main drive tires were so wide that there was no way to install ASGs at the desired wheel-path locations (±1.22 m from the centerline) without the paver tires directly impacting them during construction if using the typical mound installation method.

   To overcome this challenge, an alternative ASG installation procedure was used. The first step was to identify and mark the locations where ASGs were to be installed relative to the area to be paved, as shown in Figure 1*a*. Next, external reference points were established outside the paving area such that horizontal alignment and distance from the reference point to the desired ASG location could be readily found once the HMA was placed (Figure 1*b*). HMA was then placed by the paver. Immediately after placement and prior to any rolling of the hot HMA mat, a string line was used to determine horizontal alignment of the ASGs, and measurements from the external reference points located the ASGs to the desired distance from centerline of the lane. A square, flat-bottomed shovel was used to dig up fresh HMA about 200 mm square to the depth of the base course surface. A narrow trenching shovel was then used to dig a narrow slot in the HMA for the ASG cabling (Figure 1*c*). For this pavement experiment, all cables had to be routed to the right-hand shoulder; therefore, ASG cables from the left wheel path had to cross the paved lane. Figure 1*d* shows the left wheel-path ASG and cabling installed into the HMA mat before backfilling.

   Figure 1*e* shows the same process for the right-hand wheel-path ASG. Due to the variable nature of the low-quality base course being used, the paver wheel caused an indentation in the base course, and the HMA was thicker on the right-hand wheel path. To ensure placement of the ASG at the correct depth from the surface, the excavation was made to the same depth as the other side, and a thin layer of HMA filled the base course indentation under the ASG (Figure 1*f*). Once ASG and cabling were in place, the HMA was used to backfill the areas. A hand tamp was used to provide some compaction and ensure a smooth surface (Figure 1*g*). The roller then passed over the entire area with its normal compaction pattern. By carefully pre-staging the process, the delay in compaction for the entire ASG installation did not exceed 10 minutes. Figure 1*h* shows the final compacted surface; there are no joints, and only a few sandy booth tracks are visible.

FIGURE 1 ASG installation: (*a*) ASG layout before paving; (*b*) establishing reference outside paving lane; (*c*) excavating HMA for ASG cabling; (*d*) ASG in place before backfilling, full depth; (*e*) excavating HMA for ASG; (*f*) ASG in place before backfilling, partial depth; (*g*) tamping backfilled HMA; and (*h*) final compacted HMA surface.

Findings

Asphalt Strain Gauge Response to Traffic Loading

Two types of military vehicles were used to accelerate traffic testing of the LVR pavement experiment. A four-wheel, two-axle utility truck with a gross weight of 47.6 kN (Figure 2*a*) was used. Figure 2*b* is a representative set of ASG response data from this vehicle type. ASG response in both the longitudinal (blue line) and transverse (orange line) orientations are shown. The ASGs provided a nice, clean signal response to the vehicle loading. Note that the longitudinal ASG returns to a zero value after the truck passes; however, the transverse ASG does not. This is possibly due to slippage between the HMA and the base layer (*16*).

An eight-wheel, four-axle tactical cargo truck with gross weight of 258 kN (Figure 2*c*) was also used for trafficking. Figure 2*d* is a representative set of ASG response data from this vehicle type. The ASGs also provided a nice, clean signal response to this vehicle. Both transverse and longitudinal ASGs show permanent, nonrecoverable strains due to the heavier traffic loading. The clean signals observed under traffic loading indicate that the ASGs were well embedded into the surrounding HMA and responding to strains within the HMA layer. After trafficking was completed, the ASGs were excavated to verify their placement depth and evaluate the quality of embedment into the HMA, which was deemed to be excellent (Figures 2*e* and 2*f*). Note the bottom-up fatigue cracks in Figure 2*e*.

Hot-Mix Asphalt Density

To further assess quality of the ASG installation, the excavated HMA into which the ASGs were embedded was trimmed into blocks, tested for density, and compared to that of the entire pavement. The vacuum sealing method to measure HMA bulk density (AASHTO T 331) was used because at the anticipated relatively low level of compaction for this pavement, the traditional saturated surface dry method (AASHTO T 166) would result in water absorption errors. The consequence of this is that the reported average in-place density from cores of 89.6% of theoretical maximum density (TMD) would be about 90.3% if measured by T 166 (*17*). Table 1 summarizes the density results. Average in-place density based on 10 nuclear gauge readings was 89.9% of TMD. The average density of three cores was 89.6% of TMD. This level of compaction is rather low compared to most highway specification requirements; however, it is not uncommon for LVR pavements.

(a)

(b)

(c)

(d)

(e)

(f)
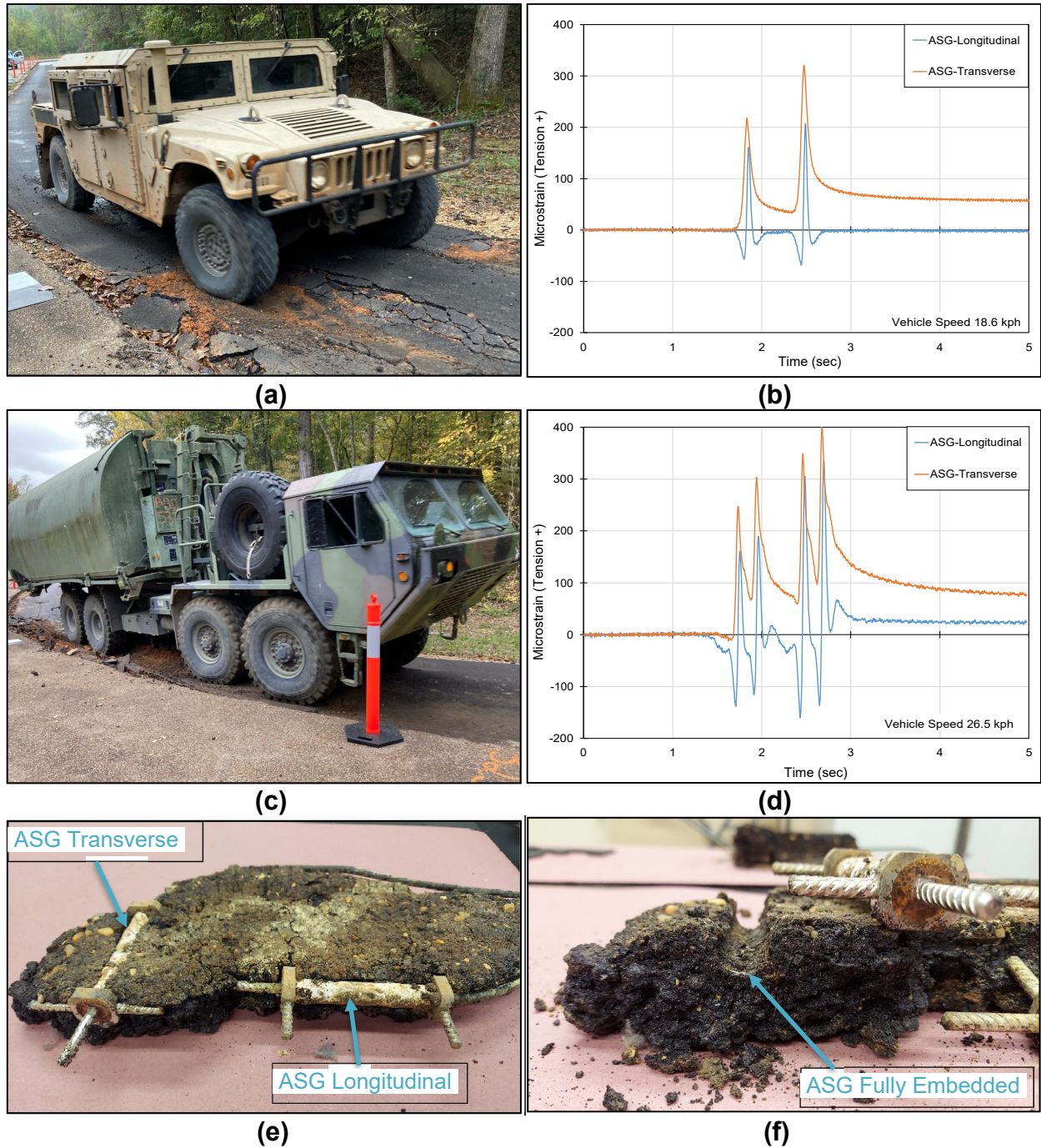
FIGURE 2 ASG performance: (*a*) M1114 truck trafficking; (*b*) ASG response to M1114 truck; (*c*) M1977 truck trafficking; (*d*) ASG response to M1977 truck; (*e*) bottom view of full-depth ASG in HMA; and (*f*) HMA after transverse ASG removed.

TABLE 1 Asphalt Layer Density Results

Location Station (m)	Offset (m)	Nuclear Density[a] Percent of TMD[c]	Cores[b] Percent of TMD[c]	Around ASG[b] Percent of TMD[c]
21.3	1.22	89.2	87.3	—
30.5	0.91	92.6	—	—
35.7	-0.91	91.7	—	—
43.6	-0.76	89.7	91.6	—
57.0	-0.46	90.1	—	—
57.9	1.22	—	—	85.7
57.9	−1.22	—	—	85.6
64.6	0.76	92.0	—	—
90.2	1.22	88.8	—	—
91.8	−1.22	84.3	—	—
119.8	−0.76	91.1	—	—
126.8	−1.07	90.0	90.0	—
	Mean	89.9	89.6	85.6
	COV[d] (%)	2.6	2.4	0.1

Note: — indicates no data.
[a] AASHTO T355 method with offset correction made to core densities.
[b] AASHTO T331 method.
[c] TMD: theoretical maximum density.
[d] COV: coefficient of variation = standard deviation/mean * 100.

Block samples surrounding the ASGs had an average density of 85.6% of TMD. Density values of the block samples were less than most density values observed for the overall pavement. This may be partially attributed to difficulties with loose aggregates when cutting the blocks; however, in general their density is noticeably lower than that of the overall pavement. Lower HMA density adjacent to instrumentation is expected.

This indicates that the ASG instruments and their installation procedure have some impact on the localized HMA properties at the measurement location. The degree to which this affects measured HMA strains is unknown. Data are not available for HMA density adjacent to ASG installed with the conventional mound method; therefore, comparisons cannot be drawn between methods.

Conclusion

An alternative ASG installation procedure was successfully implemented during construction of an LVR thin pavement experiment. The installed ASGs performed well during APT traffic loading.

Forensic examination confirmed that the ASGs were well embedded into the HMA; however, compacted HMA density around the ASG was somewhat lower than that for the overall construction. The new alternative ASG installation procedure is suggested for use when narrow paving width constraints do not allow for traditional ASG installation methods to be utilized.

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Using Bender Elements to Investigate the Effect of a Multi-Axial Geogrid on Aggregate Layer Stiffness

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Geogrid inclusion in low-volume road applications has been shown to improve rutting performance. While rutting performance and traditional instrumentation response data provide meaningful insight of pavement behavior, they do not provide a direct measurement of material behavior near the geogrid. Recently, the University of Illinois at Urbana–Champaign has developed field deployable shear wave transducers, i.e., bender elements, to measure localized stiffness enhancement attributed to geogrid inclusion in unbound aggregate layers that have been verified in laboratory and limited field applications. Bender element sensors were installed in an accelerated pavement test experiment at the US Army Engineer Research and Development Center that included a recently developed multi-axial hexagonal geogrid. The sensors were installed directly on the geogrid and 10 cm above the geogrid in the unbound aggregate layer to measure the geogrid zone of influence. The test item was trafficked with a dual-wheel truck gear, and bender element measurements were made at select intervals throughout traffic application. The data were analyzed using two techniques, first arrival time and peak to peak arrival time, to estimate unbound aggregate moduli. It was found that the bender element sensors were sufficiently robust to survive installation, construction, and trafficking. Calculated modulus values 10 cm above the geogrid were found to be higher than those directly

on the geogrid, which was an unexpected finding. However, a forensic investigation revealed moisture and fines migration into the bottom of the unbound aggregate layer, which could have dampened the shear wave signal, resulting in lower calculated modulus values.

Unpaved Road Issues

A Gravel Loss Prediction Model Using Beta Regression

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Unpaved roads consist of considerable portions of the road network in many countries. These roads are crucial to development of infrastructure systems, advancements in socioeconomic activities, and improvements in agricultural and production sectors. Unpaved roads benefit the underdeveloped, rural, and remote neighborhoods and act as lifelines for geographically disadvantaged communities. Frequent and regular maintenance activities keep the roadway system operational at a desired level of service. Resurfacing is one of the major maintenance treatments for unpaved roads. A gravel loss prediction model (GLPM) can evaluate the impacts of varying magnitude of resurfacing treatments on the roadway performance. A GLPM can provide valuable insights for roadway maintenance budget scheduling and decision-making tasks. In this paper the backgrounds, input requirements, and output results of three popular GLPMs were reviewed. These models were Highway Development and Management Model 4; South African Technical Recommendation for Highways Model 20; and Australian Road Research Board Model. Also, the practicality of roadway resurfacing frequency charts, which were developed based on these models, was evaluated. This study determined that the existing GLPMs and the corresponding roadway resurfacing frequency charts were often unreliable and impractical. In this study, a beta regression (BR) analyses methodology was utilized to develop and calibrate a GLPM for Iowa. Simple yet effective, the BR model

outperformed the popular GLPMs and offered a practical approach to quantify annual roadway gravel loss.

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

Development of a Tool for Determining Optimal Material Blends for Unpaved Road Wearing Courses

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Experience over more than 100 years has shown that appropriate choice of materials used on unpaved road wearing courses is critical for ensuring good and safe performance. Despite the availability of internationally developed performance-based specifications for unpaved road wearing course materials that will meet most requirements, many road agencies continue to use empirical specifications with grading envelopes and plasticity index ranges that do not provide an indication of expected performance. This complicates choosing optimal blends of more than one material to get a better performing unpaved road wearing course or determining quantities of supplemental materials and a recycling depth when converting distressed low-volume paved roads to an engineered unpaved road standard. This paper introduces a performance-based approach to selecting unpaved road wearing course materials and a tool for optimizing blends of materials to meet the specification. The tool can be used in manual calculations, but rapid results can be obtained by using a web-based tool. The tool can be used for new unpaved road construction, regraveling existing unpaved roads, or converting distressed paved roads to an engineered unpaved road standard. The tool is a companion to another tool that can be used for selecting chemical treatments for unpaved roads. It should be noted that results from the tool are dependent on accurate input values for materials and road layer thickness, and consequently project investigations and testing of materials are encouraged. Estimated values are likely to give misleading outputs.

Rehabilitation Options and Guidance for Severely Distressed Low-Volume Paved Roads with a Focus on Conversion to Engineered Unpaved Surfaces

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A large percentage of low-volume paved rural roads in the United States is in a severely distressed state, primarily due to delayed maintenance because of limited funding. Road agencies are now faced with decisions on what to do with these roads under constrained budgets. Options include closing selected roads completely if more than one road can be used to access properties; converting the distressed paved road to an engineered unpaved road, pulverizing the existing materials to form a new base, and applying a surface treatment (e.g., chip seal); or doing a full-depth recycle, potentially with supplemental aggregates, using bituminous or cementitious recycling agents (choice dependent on the properties of the recycled material) and then applying a surface treatment or thin asphalt overlay. Choice will depend on traffic volume, safety concerns, available materials, and available funds. Regardless of the choice, sound engineering procedures should be followed throughout the conversion or rehabilitation process to ensure that funds are used optimally, and that the road provides satisfactory performance over its new design life. This paper provides guidance on converting roads to an engineered unpaved standard.

Rapid Ground Stabilization of Low-Volume Roads for Military Vehicle Traffic

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This paper describes the assessment of rapid soil stabilization for repairing and upgrading damaged low-volume roads in contested military environments. The research objective was to identify and evaluate techniques for rapid soil stabilization to support military ground vehicle maneuver. Various types of stabilizers mixed with silty sand were assessed in the laboratory for their compressive strength at various soil moisture contents and in the field for their rutting performance. Laboratory data were analyzed to downselect products for field evaluation. Field data were analyzed for the ability to withstand trafficking from a military vehicle by evaluating the rut depth. The field placement process was also refined so it can be used in remote locations with minimal equipment. The field testing showed that the rapid soil stabilization techniques evaluated were able to withstand the required traffic without a traditional pavement surface material.

POLICY, PLANNING, AND ECONOMICS

Low-Volume Track Paths in Austria
Theory and Application

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A new guideline regarding track paths in Austria has been elaborated (RVS 03.03.82). This paper shows its practical applicability in a case study of a recently built track path.

The obvious advantage of this tracked paving approach is to reduce the overall impact of impervious surface types on the environment. Low-volume rural track paths provide an alternative to full-width construction in terms of nature and landscape conservation as well as animal ecology, as they reduce the ecological separation, resources used and sealing of the landscape. With regard to economic efficiency, considered over the entire service lifetime, low-volume rural track paths have produced positive experiences with respect to the construction and maintenance costs, as well as concerning technical, economic, ecological, resource saving, and user-related aspects (Haslehner, 2019).

Case Study: Rust

Out of the many track paths built in Austria since 2018 and according to the new guideline (Haslehner, 2019) the track path project in the community of Rust has been selected as a prime example. This track path is situated in a tourist hotspot and at the same time in a nature preserve. This example also approves the ecological advantages of the presented construction method.

General Aspects: Location and Boundary Conditions

The example presented in this paper is situated in the "Ruster Hügelland", which is a slightly hilly area close to the biggest lake of Austria and the Hungarian border (Figure 1). The municipality of Rust is inhabited by 1,984 people and is a very famous tourist
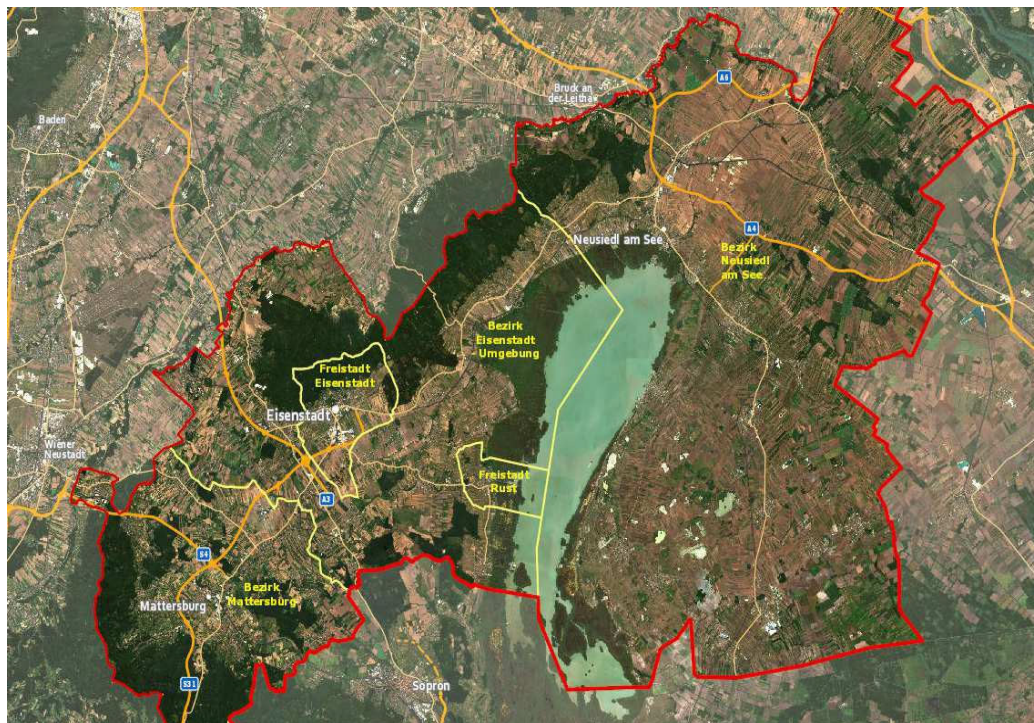
FIGURE 1 Municipality of the Free City of Rust.

destination not only in the state of Burgenland but the whole of Austria. It is renowned as the city of Storks, owing to the dozens of storks making their homes on the rooftops of the houses, and its fine viticulture. In addition, Rust boasts a history dating back to the year 1681, when it was declared a free city in the Kingdom of Hungary.

Funding

After long negotiations with the municipality and the road-building community responsible for this project it was decided that the existing unbound path should be paved as a track path. The project was funded by the state of Burgenland under the program for construction of rural roads and paths. Within this program the state subsidises 50% of the construction cost, while the other half has to be carried by the municipality and the responsible road-building community. This project allows 96,000 m² (approx. 24 acres) of agricultural land to be developed, which is partitioned among 21 landowners. Some of the land along the track path is also owned by municipality of Rust, the development of this land was not funded by the state of Burgenland. The funding and construction of this project was carried out according to the state law for road management in Burgenland (2005). Figure 2 gives the location of this low-volume

FIGURE 2 Map of the project site.

track path project. The map shows the Free City of Rust on top with its already existing rural path network in blue, while the track path project is shown in red.

Technical Characteristics

The track path with a length of 440 m (1,450 ft) has been approved for building by the state government.

    The cross-section of the track path was designed with a concrete track width of 95 cm (3.1 ft) and a central strip width of 80 cm (2.6 ft). These measurements are again given in the schematic cross-section shown in Figure 3.

    For the dimensioning of the rural track paths in Rust (Figure 4) the elaborated construction design standard (Table 1) has been used based on traffic load and bearing capacity (Haslehner, 2019).

    According to Table 1 the bearing capacity of the base layer was to be 35 MN/m^2 (5,100 psi) while the bearing capacity of the unbound layer had to be 60 MN/m^2 (8,700 psi). It was checked that these minimum requirements on the bearing capacities were met, as outlined with the test results in chapter 2.5.1. The expected lifetime traffic load is less than 2,000 standard axle loads. Regarding these criteria it was designed with an unbound layer thickness of 20 cm (0.66 ft) and a concrete path thickness of 14 cm (0.5 ft) according to the elaborated construction design standard.
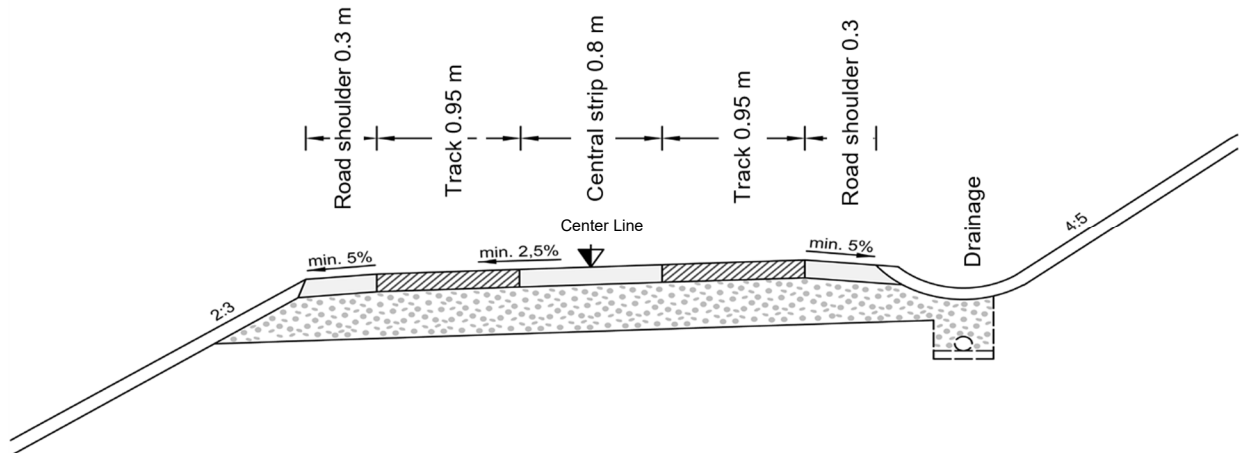
FIGURE 3 Cross-section, – Case Study: Rust.

FIGURE 4 Case study: Rust.

TABLE 1 Design Standard for Rural Track Paths (Haslehner, 2019)

Load class		LK-L I			LK-L II			LK-L III		
A - ESAL		≤ 5,0 ·10⁴			≤ 1,0 ·10⁴			≤ 0,2 ·10⁴		
$E_{V1,UP}$ [MN/m²]		≥ 25	≥ 35	-	≥ 25	≥ 35	-	≥ 25	≥ 35	-
$E_{V1,BP}$ [MN/m²]		-	-	≥ 60	-	-	≥ 60	-	-	≥ 60

Building type S1 — Surface Treatment / Unbound layer, 0.8 ≤ B ≤ 1.1

Building type S2 — Asphalt / Unbound layer, 0.8 ≤ B ≤ 1.1

Building type S3 — Block paved / Gravel bed / Unbound layer, 0.8 ≤ B ≤ 1.1

Building type S4 — Concrete / Unbound layer, 0.8 ≤ B < 0.9; Concrete / Unbound layer, 0.9 ≤ B < 1.0; Concrete / Unbound layer, 1.0 ≤ B ≤ 1.1

Legend:

- Unbound Layers — RVS 08.15.01 [2010]
- Block Pavement — RVS 08.18.01 [2009]
- Requirements on Asphalt Layers — RVS 08.16.01 [2010]
- Concrete Pavement — RVS 08.17.02 [2011]
- Gravel Bed — RVS 08.18.01 [2009]
- Surface Treatment — RVS 08.16.04 [2012]
- / improvement of bearing capacity recommended
- ▽ Substructure surface
- S thin unbound layer to ensure drainage and evenness

B   Width of the track

The resulting schematic road structure with the specific dimensions for this case study is shown in Figure 5.

ÖNORM B 4710 part 1 [2018] defines the requirements on the concrete layer for the concrete type C30/37 B7 GK22 F45, which was to be used in the construction (Figure 6). Again, it was checked that these demands were met, as we show in chapter 2.5.2.
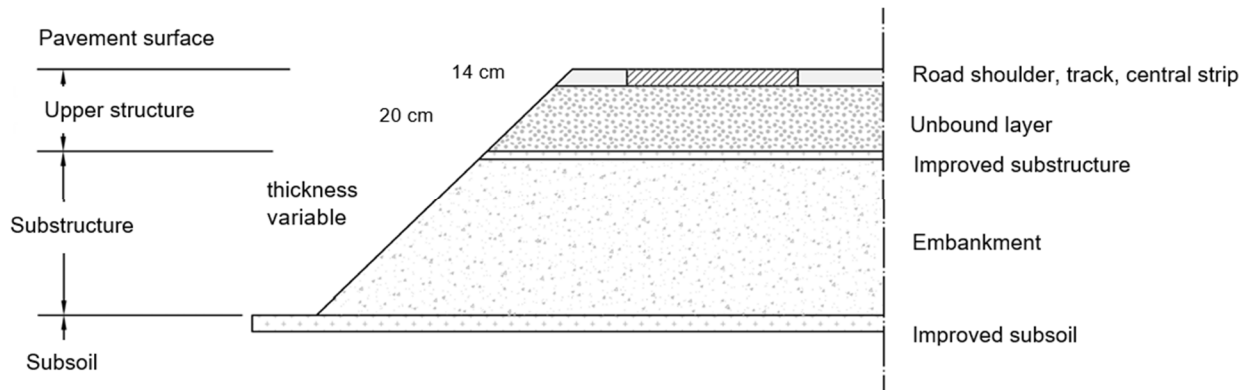
FIGURE 5 Road structure: rural track path in Rust.

**FIGURE 6 Schematic road structure with the specific dimensions
for Rust case study.**

Call for Tenders

The call for tenders had to include the federal and also state law.

Details of the Call for Tenders

The tender validity date was set for 12 months after submission, until which the tender committed to keeping their price unchanged, this was done so in accordance with the federal law. After the submission deadline ended in February of 2022 the tender remained open for acceptance by the road-building community for 6 months. Within this period the road-building community decided upon whom to award the construction of the track path. The call for tenders also included legal matters regarding labor, salary and social matters, as well as outlining the legal boundary conditions of the construction regarding the building material. As the construction site was situated in a nature preserve special attention had to be paid to the laws regarding landscape and water preservation, which were also highlighted in the call.

The call for tenders also specified the beginning and end dates for the construction work. The technical standard for the construction of the track path was set according the guideline RVS 03.03.81 rural roads and paths (2011) and RVS 03.03.82 track paths (2017), which were developed and published by the "Austrian Research Association for Roads, Railways and Transport." The concrete type was set to be C30/37 B7 GK22 F45, according to ÖNORM B4710 part 1 (2018).

Result of the Call for Tenders

In accordance with the federal law, three companies who were known to meet the required technological and economical standards were invited to send in their offers. After examining and verifying the contents of the offers, the results were analyzed and arranged in Table 2 by offered price from lowest to highest.

TABLE 2 Call for Tenders: Result

Tenderers	Range	Price	Total Price
A	1	27.170,00 €	32.604,00 €
B	2	50.270,00 €	60.324,00 €
C	3	62.264,40 €	74.717,28 €

Table 2 shows that the offers are very scattered and that the estimated price far overreached the lowest price offer. The tenderer with the cheapest price was selected due to economic considerations.

Construction and Testing

During the construction tests to determine the thickness and bearing capacity of the unbound layers, various tests concerning the concrete layer have been carried out. The unbound layer was constructed in May of 2022 and shortly thereafter the concrete layer was added to avoid damage or impurities in the unbound layer due to the agricultural traffic in the area.

Testing of the Unbound Layer

The bearing capacity of the unbound layer was determined by two static load plate tests, the results of which are given in Table 3.

Table 3 clearly shows that in both tests the minimally required bearing capacity of the unbound layers, which in accordance to the guideline for track paths is set to be 60 MN/m^2, is easily met (Table 1). The compaction factor EV_2/EV_1 shows that the unbound layers have been sufficiently compactified. The required thickness of the unbound layer has been tested over the whole length of the track path and was found to be in line with the 20 cm requirement.

Testing of the Concrete Layer

The presented suitability and conformity test of the concrete type C30/37 B7 GK22 F45 are in accordance with the values demanded in the call for tenders and ÖNORM B 4710 part 1 [2018]. The compressive strength of 36 N/mm^2 was reached and the test results after 28 days of the construction are given in Table 4.

TABLE 3 Results of the Static Load Plate Testing

Test No.	EV$_1$ [MN/m^2]	EV$_2$ [MN/m^2]	EV$_2$/EV$_1$
1	111,6	187,5	1,68
2	86,5	142,1	1,64

TABLE 4 Compressive Strength after 28 Days

Test	Bulk Density [kg/m^3]	Max. Load [kN]	Compressive Strength [N/mm^2]
1	2380	1151	51,1
2	2360	1113	49,1
3	2380	1067	47,2
Average	2370	1110	49,1

The total air content, bulk density and temperature of the concrete layer have been continuously monitored and documented during the construction. The results of the acceptance inspection of the concrete layer (of concrete type C30/37 B7 GK22 F45) were in accordance with the values specified in the call for tenders. The minimally required thickness of the concrete layer of 14 cm was found to be achieved over the entire length of the track path.

Concluding Remarks

This paper demonstrates the successful application of the new guideline (RVS 03.03.82, 2017) for low-volume track paths in Austria. The case study in Rust, which was selected as an example for the presented paper, has clearly proven the guideline to be an invaluable tool in helping to transfer theoretical knowledge and expertise into practical application. With its structured layout the guidelines are easy to handle and streamline the process of planning and executing the project. The example of Rust has also shown that low-volume rural track paths provide a very valuable approach regarding nature, wildlife and landscape preservation and result in a very positive ecological impact. The purpose of this paper is to showcase the general idea of economically and ecologically conscious construction in sensitive areas (e.g., national parks or nature preserves). This is a general approach that can be applied in every region around the world with the advantage and possibility of regarding specific local conditions.

There is no geographical or regional limitation to the idea of building track paths, to reduce the ecological impact of human built tracks on the environment.

Literature, Standards, and Guidelines

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Mechanistic Basis for Permit Fee Decision of Superloads Traveling on Low-Volume Roads Using Structural Damage Evaluation

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Superloads, including implements of husbandry and superheavy loads transporting significant amounts of heavy agricultural and industrial products, frequently travel on county or municipal roads designed for low-volume traffic so as not to interrupt highway traffic flow by their slow-moving behavior and wide vehicle widths. Such nonstandardized loading configurations and high gross vehicle and axle weights of superloads have significant potential for causing unexpectedly greater distress on low-volume roads than general vehicle classes categorized by the Federal Highway Administration. To evaluate the impact of superloads on paved and granular roads designed for low-volume traffic, a mechanistic road structural and damage-associated cost analysis is needed for predicting unexpected damage induced by various types of superloads, and to provide a logical basis for superload permit fee decisions. In this study, a layered elastic theory-based road and pavement analysis was performed to quantify damages to flexible pavements and granular roads caused by superloads. Road-damage-associated costs of flexible pavements and granular roads subjected to a single pass of various superload types were also derived and compared through multivariate life-cycle cost analysis. Suggested potential permit fees for each type of superload traveling under different road conditions were developed by calculating damage-associated costs related to traffic, road structure, material, and treatment types.

Bureau of Land Management's National Long-Range Transportation Plan

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Public lands managed by the US Department of Interior's (DOI's) Bureau of Land Management (BLM) are recognized as America's Great Outdoors, and a "Backyard to Backcountry" treasure. More than 120 urban centers and thousands of tribal and rural communities are located within 25 mi of BLM-managed public lands. The BLM manages over 90,000 mi of roads, primitive roads, trails, bridges, and culverts across BLM public lands in the United States. The BLM transportation system is essential to fulfilling the agency's multiple-use mission of sustaining the health, diversity, and productivity of public lands for the use and enjoyment of present and future generations. The BLM transportation system supports economic generation, recreation access, conservation, disaster response and evacuation, and tribal and rural community connections. It is highly connected to the transportation networks of its partners, including those facilities owned and maintained by other Federal Land Management Agencies (FLMAs), state departments of transportation (DOTs), county governments, tribal governments, and private landowners.

Because the BLM's transportation system is in many ways an "unsung hero" in supporting the agency's mission, the BLM saw a need to develop a long-term vision for managing its transportation program.

To address this need, the BLM published its first-ever National Long Range Transportation Plan (the Plan), *Transportation Connections 2040* on October 5, 2021. The Plan is a practical tool to convey the BLM's strategic goals and vision for travel and transportation planning and transportation asset management. It is intended to advance

the Biden-Harris Administration's national conservation and restoration priorities established in the 2021 *Conserving and Restoring America the Beautiful* multi-agency report. The BLM aims to work collaboratively to manage the multimodal transportation system that supports the equitable access, connectivity, and safety needs of multiple uses across public lands, while protecting natural, cultural, and historic resources.

Methodology

The BLM's Plan is the culmination of a collaborative, interdisciplinary effort to create a common vision for managing the BLM's transportation network. The BLM Division of Business, Engineering, and Evaluations and the Division of Recreation and Visitor Services worked collaboratively to develop the Plan, with facilitation and technical support from the US DOT's Volpe National Transportation Systems Center. Between 2018 and 2019, an interdisciplinary team of members involved in various program areas and representing the BLM headquarters, state, district, and field offices developed the content within the Plan. The Federal Highway Administration Office of Federal Lands Highway also participated in the Plan's development and review. The BLM engaged in outreach with transportation officials from numerous western states and local jurisdictions, including those attending the 2019 Western Association of State Highway and Transportation Officials Annual Meeting. These agencies provide critical connectivity and access for the public to enjoy BLM-managed public lands.

The plan identifies actions to help the BLM reach the following transportation goal areas, which the BLM developed as part of the plan development process:

- Access, Connectivity, and Experience. Manage the BLM's transportation system to provide seamless public access to support the BLM's multiple-use mission.
- Transportation Asset Management. Strategically invest funding to sustainably maintain BLM transportation assets.
- Collaborative Partnerships. Develop and maintain collaborative partnerships for a transportation system that connects communities to public lands.
- Natural, Cultural, and Historical Resources. Manage the BLM's transportation system to protect resources while providing appropriate access.
- Safety. Provide safe and appropriate multimodal transportation access for all users of BLM-managed lands.

To ensure accountability and link the Plan to future investment decisions, the Plan establishes a performance-based framework for implementation and monitoring. As

such, the Plan includes objectives, strategies, and performance measures for each goal area. For each strategy, the Plan identifies the current status (e.g., existing activities, expanded activities, or new activities), responsible BLM division, and the time horizon for implementation. In the monitoring plan, the Plan identifies existing data sources and responsible divisions for eac h performance measure.

The BLM developed this plan to benefit multiple different audiences, including BLM staff; BLM and DOI leadership; BLM's partners, including other FLMAs, the US DOT, state DOTs, counties, tribes, and other partners; and the traveling public. BLM staff at headquarters, state office, district office, and field office levels can use this plan to guide their transportation decision-making. BLM's partners can use this plan to identify opportunities to collaborate with the BLM on projects of mutual benefit. The general public can use this plan to better understand the BLM's transportation program and how the BLM's transportation planning process supports the agency's mission and improves public access to BLM-managed public lands and resources.

Since publishing this Plan, the BLM has developed a Performance Management Plan that identifies baseline data, trends, and targets for the performance measures in the Plan and establishes procedures for ongoing monitoring. Ongoing monitoring of performance will help the BLM evaluate how well the agency is meeting its transportation goals and will identify opportunities to adjust management practices to improve transportation conditions. The BLM intends to use the performance-based framework in this plan to inform future investments over the next 20+ years.

Findings

The BLM Plan provides a data-driven analysis of baseline conditions for the BLM's transportation system; a practical, strategically prioritized list of actions the BLM can take to implement the plan and achieve its objectives; and a set of meaningful and feasible performance measures to monitor progress. Given the uncertainty of future conditions over the Plan's future planning horizon of 20 years, the Plan's implementation and monitoring framework is flexible. For example, the plan provides that the performance measures should be reviewed and updated as the BLM transportation program matures over time to ensure effective performance monitoring.

The BLM Plan was titled *Transportation Connections 2040* to emphasize that transportation is about making connections to BLM-managed public lands for the wide range of travelers that use those lands for work, enjoyment, or intercommunity travel. The BLM's transportation system includes particularly important connectors for many

rural communities throughout the United States; the Plan provides the strategic direction to help the BLM improve these physical connections on the land. This plan also aspires to build and strengthen the virtual connections that the BLM's transportation program relies on, both between program areas within the BLM and with the many local, state, and national partners that allow the BLM to achieve more than it can on its own.

Conclusion

Congress tasked the BLM with a mandate to manage public lands for a variety of uses such as energy development, including renewable energy, livestock grazing, recreation, and timber harvesting while ensuring natural, cultural, and historic resources are conserved for present and future use. Achieving this mission is not possible without a transportation system that can effectively and sustainably move people and equipment to and through BLM-managed lands. The routes on these lands are often a complex network of unpaved legacy roads that originated for purposes that have shifted over time, no longer meeting the safety, access or other needs of current users. Portions of these roads are owned and maintained by the BLM while others are owned or maintained by county, state, other government, or private entities. As such, the BLM depends on partnerships to plan and implement transportation improvements and leverage limited funding.

The BLM developed *Transportation Connections 2040* to be a springboard for collaboration among the BLM transportation program and its partners. The BLM established its national transportation goals, objectives, strategies, and performance measures in this plan to respond to the BLM's unique challenges, needs, and opportunities. Yet many of the actions proposed in this plan are mutually beneficial to the Bureau's partners. For example, each of the BLM's five priority goal areas are aligned with the US DOT's national goals for the Federal-Aid Highway Program at 23 USC 150(b). Therefore, many of the strategies in this plan are intended to address the same goals that state DOTs, metropolitan planning organizations, and others are working toward achieving.

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Local Government Wheel Tax for Road Infrastructure Improvements

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While labor, fringe, and material costs have increased substantially in the past two decades, state aid and local government budget allocations for road infrastructure have not been commensurate with these cost trends. Consequently, deferral and backlog of maintenance are becoming common practice with adverse impacts on road infrastructure performance and on road users. These challenges have compelled some local governments to seek additional funding sources to address the maintenance backlog and needed capital improvements. One funding source that is gaining attention in the state of Wisconsin is the wheel tax. It allows governmental entities, per Wisconsin Statute, to impose a flat annual registration fee on certain vehicle types normally kept under the governmental entity's jurisdiction. The county of Iowa in Wisconsin implemented a wheel tax of $20 in 2015. Within 5 years of its implementation, the county collected $2.1 million. This provided the required matching funds for larger state and federal grant money for the construction and rehabilitation of 14.13 mi of roads and three bridges at a cost of $6.93 million. The Iowa County experience appears to suggest that the wheel tax is a sustainable option to raise additional funds for clearing backlog of road improvements and meet matching fund requirements for other external grants.

Introduction

A major concern of all governmental entities including counties, cities, towns, and villages, is the maintenance and improvement of their road infrastructure within the financial limitations of their budgets. Road maintenance and improvement needs,

however, have become increasingly difficult to meet due to competing needs from other sectors of development coupled with constraints on state and local tax revenues. While labor, fringe, and material costs have increased substantially in the past two decades, budget allocations for road infrastructure in Wisconsin have not been commensurate with these cost trends leading to deferral and backlog of needed capital improvements and maintenance.

The consequences of maintenance deferral are not new and continue to be highlighted. Besides cost, maintenance deferral can lead to increased safety risk to road users and reduced service life of the pavement. Wisconsin's 2020 Infrastructure Report Card revealed that more than 33% of Wisconsin's roads are in fair or lower condition and that without adequate intervention, the deterioration is likely to continue over the next 10 years. Furthermore, there is a cost of $547 per year to each driver for driving on roads that need maintenance. Roads received an overall grade of D+, while bridges earned a C+ grade on the report card (1).

Although it has also been reported that a $1 timely investment made in pavement maintenance can save $4 to $10 in future costs (2), and that user operating costs on the average represent 4 to 10 times agencies' out-of-pocket costs (3), governmental entities continue the practice of maintenance deferral due to insufficient funding. These challenges have compelled some governmental entities to seek additional funding sources to address the problem. Hough, Smadi, and Bitzan (4) examined traditional and innovative financing methods used by midwest and mountain plains states for local roads. They concluded that two innovative financing methods with potential for more use in the future included the wheel tax and special assessment. The special assessment is a tax levied on private developers to meet the cost of local government improvements that augment the value of the developer's property. The wheel tax is a tax imposed by a governmental entity per codified law on vehicles with a prescribed weight limit or weight and wheel combination. South Dakota, for example, per its Codified Law Ch. 32-5A, allows counties to impose a wheel tax by county ordinance on vehicles with gross weight of over 6,000 lbs. The tax rate for vehicles meeting this criterion is further assessed on a per-wheel basis, and ranges from $0 to $5 per wheel (5). In 2008, 38 of South Dakota's 66 counties imposed a wheel tax in the range of $2-$4 per wheel (6). In 2013, 46 counties collected wheel tax.

Furthermore, it was proposed to increase the wheel tax by $1 per wheel and provide additional wheels to be taxed to a maximum of 12 wheels (7).

According to DeBoer and Yadavalli (8), governmental entities in Indiana have motor vehicle excise surtax and wheel tax as county option taxes for use in road maintenance and construction purposes. These taxes are commonly referred to as Local Option

Highway User Taxes (LOHUT). As of 2012, about 50% of the counties in Indiana had adopted LOHUT since 1982. The surtax generally applies to passenger cars, motorcycles and light trucks weighing less than 11,000 lbs. It is usually adopted as a flat rate per vehicle or as a percentage of the state motor vehicle excise tax. The wheel tax on the other hand, applies to larger vehicles such as trucks weighing more than 11,000 lbs, tractors, semi-trailers, and recreational vehicles. The county council is responsible for considering surtax and wheel tax adoption. The two taxes must be adopted together. In 2011, 47 of the 92 counties in Indiana collected surtax and wheel tax in the amount of $71.8 million (*8*).

The state of Wisconsin's Statute 341.35 on the other hand, allows governmental entities to enact an ordinance that imposes an annual flat wheel tax on all motor vehicles with gross vehicle weight up to 8,000 lbs that are registered in the state, and customarily kept in the governmental entity's jurisdiction (*9*). The wheel tax has in recent years, gained attention in Wisconsin. Wheel tax collection by governmental entities in Wisconsin steadily increased from a value of $9.4 million in 2015 to $62. 8 million in 2021 (*10*).

According to the 2017 Vehicle Registration Fees by States (*11*), although only Indiana and Wisconsin collected "wheel tax," counties in the following states were allowed to collect "additional fees" besides base vehicle registration fee: California, Colorado, Hawaii, Kansas, Montana, Nevada, New York, Pennsylvania. For example, Alameda County and San Francisco County in California impose a "County Transportation Projects Fee" of $10 (*12*). In Pennsylvania, a county may pass an ordinance to implement an annual fee of $5 for each vehicle registered to an address located in that county. As of January 2022, 27 of the 67 counties in Pennsylvania had passed ordinances to implement the $5 fee (*13*).

In summary, the practice for counties to impose additional fees on vehicle registration is common but varies significantly. The collection of wheel tax, however, is only reported for South Dakota, Indiana, and Wisconsin in the literature.

The purposes of this paper are to (1) examine Wisconsin local government road infrastructure funding mechanism, and (2) share the experience of the implementation of a wheel tax in Iowa County in 2015 and its impact on road improvements.

Iowa County Funding Sources for Road Improvements

Iowa County is one of 72 counties and among 1,924 governmental entities in Wisconsin. It is in the Southwest region of the state (Figure 1). Its highway department

FIGURE 1 Location of Iowa County in Wisconsin.

currently manages a wide array of road infrastructure including 371 centerline miles of county trunk highways, 52 bridges, and 1,897 culverts. Like every other governmental entity in Wisconsin, Iowa County has the responsibility of constructing and maintaining its road infrastructure with a mix of state aids and local revenues. Currently, there are 18 different state and federal programs available for funding local government transportation projects in Wisconsin. Each program has a different set of criteria that determines the eligibility of a project for funding under that program. Local revenues for road improvements mostly come from tax levy allocations. Notable funding sources are briefly presented in the following sections in relation to Iowa County.

State Aids to Local Governments

Iowa County roads are eligible for 15 of 18 funding programs per Wisconsin Statutes. The notable programs that the county has successfully tapped into include the General Transportation Aids (GTA), Local Bridge Improvement Assistance (STB-Bridge), Surface Transportation Program-Rural (STP-Rural), Flood Damage Aids Program, and Emergency Relief (ER) for major collectors and routes with higher functional classifications. Due to functional classification criteria requirements, not all routes are eligible for participation in every program. Likewise, not all programs provide adequate funding to award all eligible projects across the state. Hence, the viability of a project at the county level is subject to a comparative review analysis against other project applications from across the state.

 The GTA is the largest category of local aid distributed to governmental entities by the state. It represents approximately 25% of all state-collected transportation revenues from fuel taxes, vehicle registration fees, and other transportation-related taxes and

fees. It is designated for portions of the cost pertaining to activities such as road construction, filling of potholes, snow plowing, grading of shoulders, pavement markings, and repairing curbs and gutters. Fund allocation is based on a 6-year spending average or a statutorily set rate-per-mile. A governmental entity's share of the GTA is based on the road mileage under its jurisdiction and the amount of the governmental entity's own resources it spends on its mileage over a rolling average of 6 years. This is used to calculate rate-per-mile (RPM) payments and further used to determine share of costs (SOC) payments. The SOC percentage is determined on a yearly basis and considers the total costs reported as well as the balance of remaining funding within the appropriation. Governmental entities cannot receive more than 85% of their 3-year costs average regardless of whether they are RPM or SOC eligible (*14*). GTA assistance to governmental entities has generally declined for almost two decades (Figure 2*a*). GTA assistance to governmental entities in 2019 was $442.69 million, a reduction of approximately 9.3% from its peak of $487.99 million in 2003. The declining trend can be attributed to a couple of factors: (a) Wisconsin's gas tax has not been raised since 2006 and overall fuel consumption has declined as more fuel-efficient vehicles are being produced by car manufacturers and (b) state-collected vehicle registration fees also have not been raised since 2008, except for electric and hybrid vehicles. With less money being generated by the state from these sources, less aid is expected to go to governmental entities. The trend is exhibited by Iowa County, which experienced a significant reduction of nearly 31.3% from its peak of $1,264,217 in 2004 compared with $868,221 in 2019 (Figure 2*b*).

The STB-Bridge program allows local governments to rehabilitate and replace significantly deteriorated or deficient bridges on a cost-shared basis between the county and the state. Bridges with a sufficiency rating (SR) of 80 or less are eligible for rehabilitation funding while replacement funding is applicable to bridges with a SR less than 50 on a 100-point scale. The SR is calculated based on standard bridge inspection reports submitted by local governments to the Wisconsin Department of Transportation (DOT) Structures Bureau (*15*). STP-Rural allows funding to be used on roads and streets in rural areas functionally classified as principal arterial, minor arterial or major collectors and are not part of the State Trunk Highway network (*16*). The ER program allows local governments to receive funding for the replacement or structural repair of federal- aid highways functionally classified as major collectors or higher, which have experienced catastrophic failures or natural disaster (*17*).
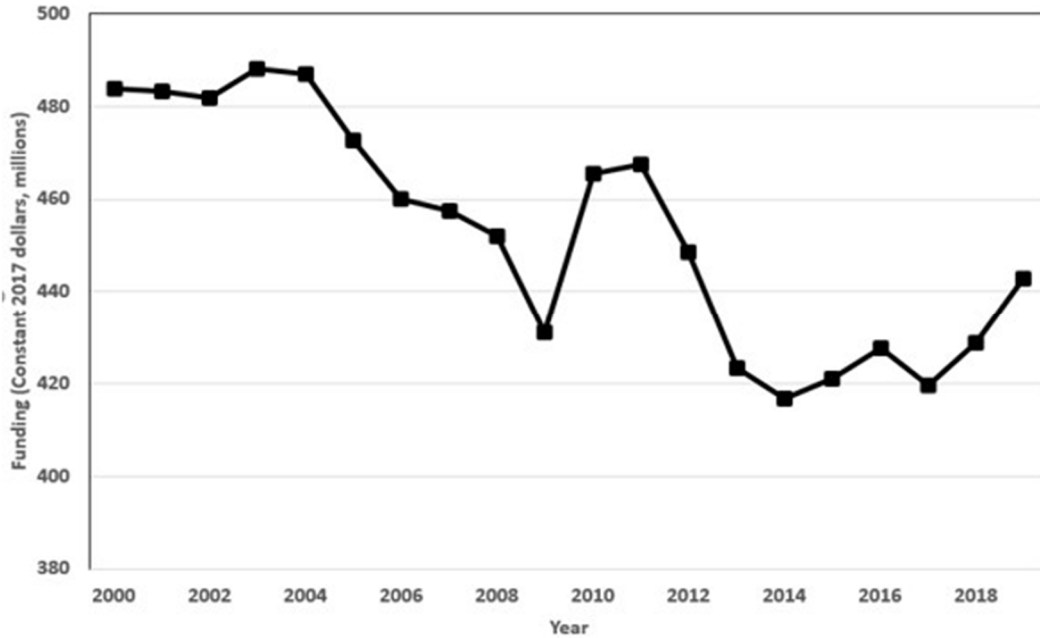
(a)
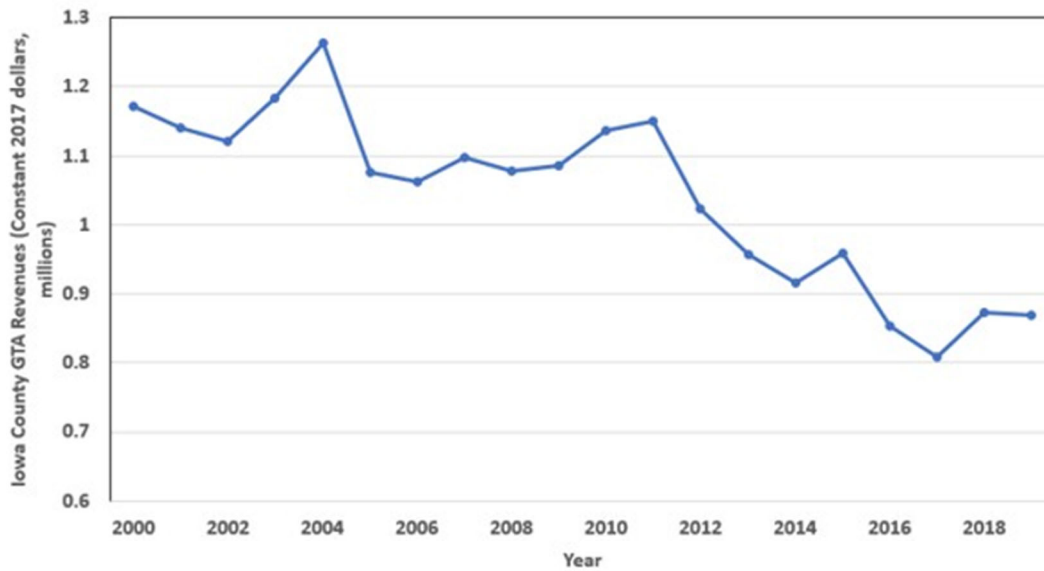
(b)

FIGURE 2 State GTA assistance and Iowa County GTA revenue: (*a*) state GTA assistance to government entities and (*b*) Iowa County GTA revenues.

Iowa County Road Improvement Funds from Tax Levy

Besides state aids, Iowa County depends on property tax and other few tax options authorized by the Legislature to fund road improvements. These options include (*a*) a county sale and use tax of 0.5%; (*b*) room tax; and (*c*) a county registration fee for certain motor vehicles, i.e., the wheel tax. Tax levy allocated from property tax receipts is the largest source of county revenue for funding road infrastructure in Iowa County. A comparison of Iowa County GTA, tax levy, and wage and fringe cost (Figure 3) show that while GTA assistance trend remains relatively flat, the tax levy has seen a slow but steady increase since 2010. However, both the tax levy and GTA have been consistently outpaced by wage and fringe costs for more than 16 years. Additionally, material cost for construction and maintenance operations continue to increase (Figure 4), compounding the challenge of having enough resources to address the backlog of road infrastructure projects in the county.
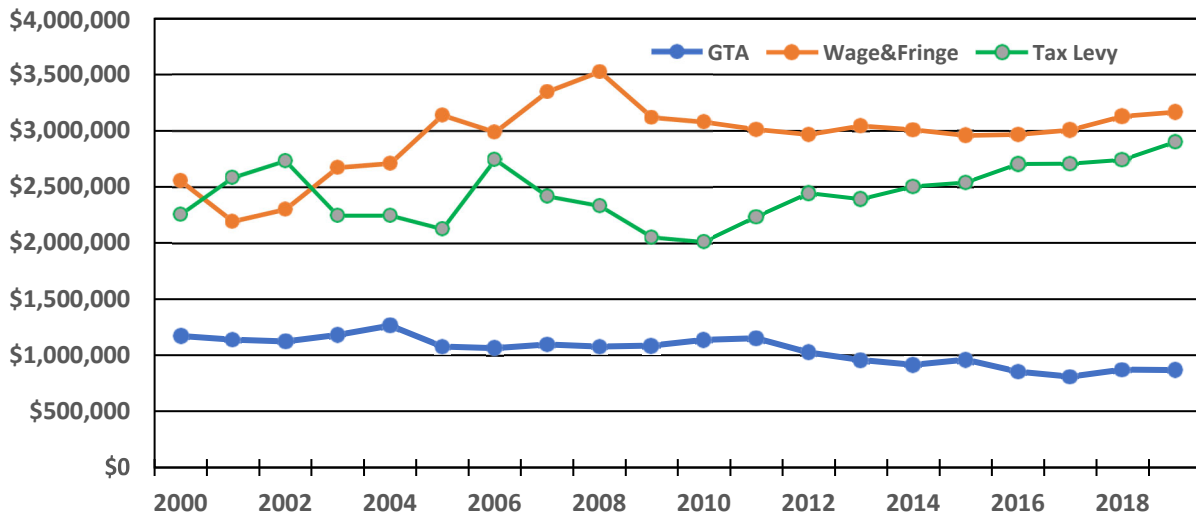
FIGURE 3 GTA and tax levy comparison with wage and fringe cost.
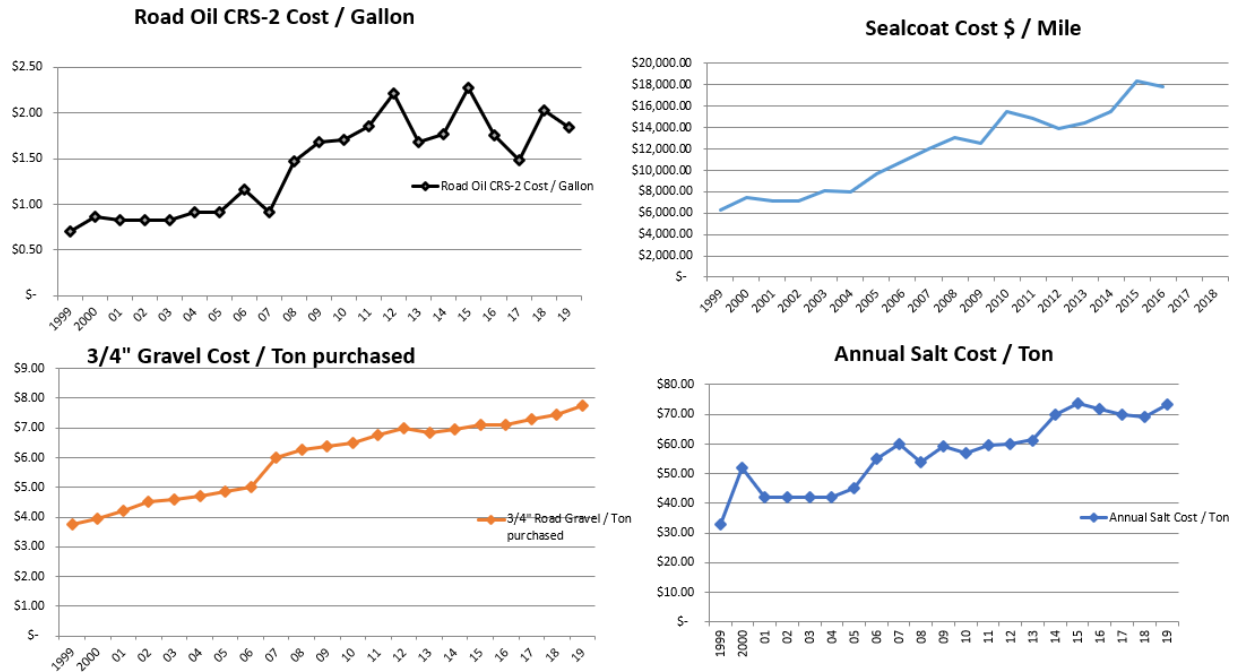
FIGURE 4 Iowa County material cost trends.

State and County Cost Share Options

The major options for sharing the cost of road infrastructure between the state and Iowa County include the following (*18*):

1.   100% local cost borne by the county through the tax levy or other sources to be expended on any classification of road the county desires for roadway and bridge

2.   50%–50% cost share between the county and Wisconsin DOT via the Local Roads Improvement Program (LRIP). The LRIP program consists of two primary components for determining cost share amounts provided by Wisconsin DOT.

   a.   County Highway Improvement Program (CHIP). The county's biennial CHIP entitlement amount is ±$136,000 per funding cycle

   b.   County Highway Improvement Program-Discretionary (CHIP-D). The county receives CHIP-D monies via a competitive process with nine counties within the South- central district of the Wisconsin County Highway Association. The county receives an amount equivalent to $430,000 once every 4 years.

3.   20%–80% cost share (county–Wisconsin DOT) via the STP-Rural. Only the routes within the county labeled as major collectors and above qualify for the program. In addition, projects must be designed and let to private industry through the Wisconsin

DOT monthly letting process.

    4.   There is also a 20%–80% cost share between the county and state or federal via the STP-Bridge Program. Bridges compete for funding based on their eligibility, county entitlement formula, and SR based on routine bridge inspection history in a biennial cycle.

    To maintain county roads in a suitable condition and extend the service life of the network, the county periodically develops capital improvement plans that focus on resurfacing, reconditioning, reconstruction, and bridges to determine its share of associated costs. *Resurfacing* requires no major changes to the roadway's horizontal or vertical alignment or to its horizontal cross-sectional elements. Resurfacing seeks to remove and replace or remove a portion of the pavement surface and increase its load carrying capacity. *Reconditioning* involves making some modifications to the roadway's geometry including widening travel lanes and shoulders. *Rehabilitation/Reconstruction* involves activities such as replacement of the driving surface and restoration of the aggregate base, improvements to enhance safety, geometrics, drainage, or intersections with minimal or no improvement to improve capacity.

    For bridges, the common capital improvement involves replacement when their SR falls below 50. The primary factor for replacement is when the bridge is structurally deficient and cannot carry expected loads or when it is functionally obsolete and cannot accommodate wider vehicle and other equipment configurations. Bridge replacement projects are subject to review by the Department of Natural Resource and compliance to FEMA guidelines and regulations regarding hydraulic efficiency analysis. Culvert replacements may occur as part of a road construction project, or when it has reached its expected service life, or when it experiences catastrophic failure. The treatment types commonly performed by the county for each capital improvement type are summarized in Table 1.

    In 2011 the county estimated that 9.90 mi of capital improvements must be constructed in any given year to keep up with the age and distress deterioration of the system. This would include 5.08 mi of major routes, 2.2 mi of minor routes, and 2.62 mi of local routes. For bridges, one bridge replacement was targeted per year. For culverts, it was estimated that two large (>60 in. diameter), 12 medium (diameter between 36 in. and 60 in.), and 40 small (diameter<36 in.) culverts would be replaced on a yearly basis. The associated county share of the cost is as shown (Table 2). The total yearly amount required was projected to be $1, 888,920, with pavement related capital improvements representing nearly 88% of the county cost.

TABLE 1 Iowa County Capital Improvement Treatment Types

Capital Improvement Type	Typical Treatment Types
Resurfacing	HMA wedge or rut filling structural overlay for distressed pavement areas. HMA nonstructural surface overlay of <2 in. Surface milling and HMA overlay: both structural (>2 in.) and nonstructural (<2 in.) Cold-in-place pulverization with a new HMA structural layer (>2 in.)
Reconditioning	Pavement Restoration: Cold-in-place pulverization and HMA structural mat Full-depth reclamation and HMA structural mat Reconstruction by shoulder widening, combined with an HMA nonstructural overlay Reconstruction by shoulder widening, combined with HMA structural overlay
Rehabilitation/ Reconstruction	Earthwork reconstruction of the horizontal alignment, vertical profile, side road intersections, and/or a combination of the same
Bridges and Culverts	Replacement

HMA = hot-mix asphalt

TABLE 2 Iowa County Expected Yearly Improvement Cost

Infrastructure Category	Funding Option Type	County Cost
Major Routes	20%–80%	$144,520
Minor Routes	50%–50% CHIP or CHIP-D	$385,000
Local Routes	100% (county)	$393,000
Major Routes not qualified for STP-Rural	50%–50% CHIP or CHIP-D	$728,000
Bridge	20%–80%	$41,400
Large culvert	100% (county)	$96,000
Medium culvert	100% (county)	$21,400
Small culvert	100% (county)	$73,600
Total Yearly Cost		**$1,882,920**

Historically, the county had struggled to come up with cost share funds for some projects and returned money to the state that benefited other counties. Consequently, the inability to generate additional funding partly resulted in significant backlog of road infrastructure improvements for the county. Hardy (*19)* reported that in 2003, the county was awarded $350,000 for an improvement project, but by the program's deadline in 2009, the county could not utilize the funds due in part to a lack of matching funds. The money was returned to Wisconsin DOT to be redistributed to another local county, which could garner the minimum matching funds of $350,000. In 2014, the county carried out another capital improvement study and estimated that approximately $7.1 million was needed for road improvement projects from 2015–2018. To complete these projects would require the county to raise a qualifying match of $2.6 million over the 4-year period or lose the money to another county for the improvement of its road

infrastructure. The County Board of Supervisors decided to pursue a wheel tax as a funding alternative to help meet its cost obligations.

Wheel Tax

The imposition of wheel tax on motor vehicles by governmental entities (cities, villages, and towns) in Wisconsin has been around since 1967 as authorized by the Legislature. In 1979, the authority was extended to counties. Since 1983, however, state law has allowed governmental entities to adopt an ordinance that imposes a flat, annual registration fee on automobiles and trucks weighing up to 8,000 lbs that are typically kept within the governmental entities' jurisdiction (20). Vehicles above the 8,000-lb weight are exempted from the wheel tax. The rationale behind the exemption is that such vehicles are more likely to use state roads than use and cause damage to low volume roads. Instead, they pay a sliding scale for vehicle registration taxes to the state based on their license type. The wheel tax is collected by Wisconsin DOT at the time of first registration and at each registration renewal. Wisconsin DOT retains 17 cents per registration for administrative costs. Wheel tax use was sporadic until recently, as governmental entities increasingly look for ways to make up for state cuts. Prior to 2011, only four governmental entities had a wheel tax in place. As of July 2021, however, the list had grown to 45 consisting of 14 counties and 31 municipalities. The distribution of wheel tax rates imposed by the 45 governmental entities in Wisconsin is shown (Figure 5). Wheel tax rates vary, but $20 wheel tax appears popular among governmental entities, with city government leading the $20 tax group.
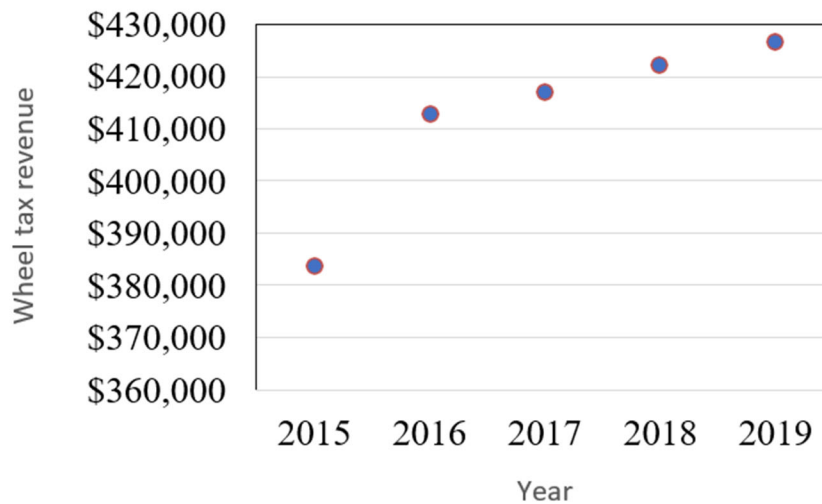
Iowa County Wheel Tax Impacts

In 2014 the Iowa County Board of Supervisors adopted a $20 per vehicle wheel tax in accordance with state law. The law does not specify the wheel tax amount but requires governmental entities to impose a flat tax and use all generated revenues for transportation-related purposes. At the time of adoption of the wheel tax, the County Board estimated that the county had approximately $7.1 million in road improvement projects scheduled for state funding from 2015 to 2018. This required the county to raise a qualifying match of $2.6 million over the 4-year period. The implementation of a
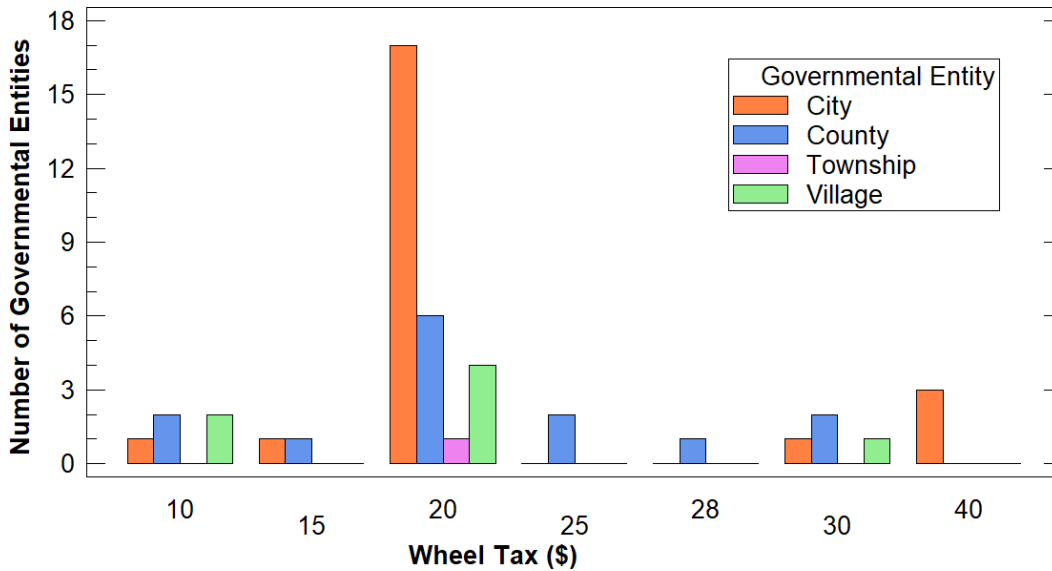
FIGURE 5 Wheel tax amounts imposed by Wisconsin governmental entities.

wheel tax was estimated to generate $430,000 on an annual basis, which could be leveraged by the county against other state or federal funding to complete improvements for five bridge projects. Since the adoption of the wheel tax, a cumulative amount of $2,062,400 had been realized as of 2019 with the yearly distribution shown in Figure 6. Due to statutory requirements related to the date of adoption, the first year of implementation (February of 2015) shows the lowest amount, but the subsequent years show a small but gradual increasing trend.

FIGURE 6 Iowa County wheel tax revenues.

Prior to the wheel tax ordinance, yearly budgets were prepared based on what projects had been planned and in contract with Wisconsin DOT for the following budget year. Sometimes a project could be dropped in the schedule or rescheduled for letting because the County Board did not allocate the funds necessary for the county to meet its local share of cost for a project already in a contract agreement for completion. With the wheel tax funding source and a dedication of those funds for improvements, 5-year capital improvement plans can now be prepared, and wheel tax revenues can be generated to help meet any contractual obligations the county has already agreed to. With the wheel tax implemented and secured, the county is more certain about what projects can be funded for the next 4 to 5 years, making it easier to apply for grants and schedule projects. Without the wheel tax revenue, project improvements were subject to the final budget and tax levy approvals of the County Board, which varied significantly from year to year. The wheel tax ordinance has allowed for the allocation of a specific funding source for contract agreements and removed the annual fluctuations.

A summary of projects completed since the implementation of the wheel tax is shown in Table 3. Roadway improvements were completed as part of projects on CTH QQ, E, K, and G. Bridge improvements were completed as part of projects on CTH E, G, and HK. Between 2015 and 2019, the county performed approximately $6,928,933 in road and bridge improvements. The local (county) share of cost was $3,342,812, of which $2,062,400 (i.e. 62%) was achieved using wheel tax generated revenue. The remaining

TABLE 3 Summary of Iowa County Projects Completed with Wheel Tax-Supported Funds

Year	Road Name	Segment Start	Segment End	Segment Length (mi)	Improvement Type	Total Cost	County Share
2015	*CTH QQ	STH 39	STH 39	3.38	Pulverize	$746,885	$628,193
2016	CTH E	Mifflin	Highway Shop	0.63	Reconstruct Bridge	$1,392,788 $924,711	$278,577 $184,942
	CTH HK	CTH K	CTH H	0.10	Bridge	$646,601	$147,680
2017	CTH K	Mounds View Rd	STH 14	7.89	Pulverize	$1,462,669	$996,689
2018	CTH E	Highway Shop	CTH G (Rewey)	1.92	Recondition	$626,507	$490,507
2019	CTH G	Pecatonica Bridge	Approaches	0.21	Reconstruct Bridge	$645,402 $483,370	$519,550 $96,674
Total Cost						$6,928,933	$3,342,812

*CTH = County Trunk Highway

$1,280,412 of local cost share funding was from a combination of short-term project borrowing, fund transfers, and tax levy allocations. Over this timeframe, 32% of all project funding was derived from the wheel tax option.

Summary and Conclusions

This paper examined governmental entity funding options for road infrastructure improvements in Wisconsin, but with attention to the wheel tax and its overall impact on Iowa County road infrastructure. Based on the examination, the following observations are made:

1. Since the implementation of the wheel tax in 2015, nearly $2.1 million was collected by 2019.
2. The $2.1 million wheel tax allowed 14.13 miles of county trunk highways and three bridges to be rehabilitated at a total project cost of $6.9 million.
3. The use of wheel tax generated funds provides a sustainable source for ongoing future revenues for the county when pursuing Federal and/or State grants. The county now has an attainable capital improvement plan/schedule due to having a reserved source for local cost share funds.
4. As state aid and local government budget allocations for road improvements continue to decline, other funding sources are inevitable. Although the Iowa County experience appears to suggest that the wheel tax is a viable option to raise additional funds for road improvements, only 2.3 % (45/1924) of governmental entities in Wisconsin currently use it. This may be due to a few factors:

    a. wheel tax applies to vehicles weighing up to 8000 lb. and not to trucks, trailers, motorcycles, and farm vehicles; communities may feel that large trucks and farm vehicles cause more damage and should pay more compared to the average car owner,

    b. others believe that any tax should be indexed on the gas tax so people can be charged according to use,

    c. if more communities enact a wheel tax, certain legislators may be inclined to require communities to reduce their allowable levy by the amount of revenue raised by the wheel tax, and

    d. if more communities enact a wheel tax, legislators may be less inclined to increase GTA or shared revenue. These factors coupled with the appearance of most communities' reluctance to adopt a wheel tax warrant further exploration.

Author Contributions

The authors confirm contribution to the paper as follows: Study conception and design: Craig Hardy and Owusu-Ababio; Data Collection: Craig Hardy Draft; Manuscript preparation: Sam Owusu-Ababio; Analysis and Interpretation of Results: Owusu-Ababio, Danny Xiao, and Craig Hardy. All authors reviewed the results and approved the final version of the manuscript

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Identifying Colonial Roads in Southern New Jersey

Application of Remote Sensing and Field and Archival Methods to Document the Locations, Condition, and Routes of Critical American Revolutionary War Supply Chains

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This paper explores the identification, use, and preservation of historic roads in the state of New Jersey that primarily travel through public lands. The authors examine in detail the historical significance of several unpaved routes that continue to exist in Burlington County, New Jersey, as well as discuss various methods that can be used to identify routes with historical significance and document their physical characteristics. Field research was conducted to establish the current location of these historic routes, using lidar, global positioning satellite, and geographic information system methods to estimate their likely date of construction. Further examination and mapping of these routes was undertaken, followed by documentation of the historical events linked to their use, thus establishing historical context. We have identified likely routes used as critical Revolutionary War supply routes. The paper concludes with a discussion of appropriate actions that should be considered in the preservation of these routes and offers planners some options in terms of public policy.

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