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The papers and extended abstracts included in these proceedings summarize over 110 presentations scheduled for this sixth biennial event at Iowa State University. Short abstracts are also included that summarize presentations for which manuscripts were not available at the time of publication of these proceedings. The topics cover a broad spectrum of transportation issues in the following areas: asphalt pavement, asset management, bridges and structures, pavements, planning, safety, traffic engineering and operations, intelligent transportation systems, human factors, environment, geotechnical engineering, and weather.

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Adding Spatial Data to Existing Information Systems

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ABSTRACT

In this session, we will take a look at how spatial data was added to the Iowa Department of Transportation's Five-Year Program, Aviation Information Management System, and Geographical Information Management System, among others. These systems had a wealth of business data but lacked spatial data for GIS analysis. Details will be given on how Iowa used Oracle Spatial and its Linear Referencing System to store and generate locations for existing systems. Issues and successes for each project will be shared.

Key words: Iowa—information management—spatial data

Accelerated Design and Construction for the 24th Street Bridge in Council Bluffs, Iowa

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ABSTRACT

With the help of Highways for Life initiative and the Innovative Bridge Research and Construction program, Iowa Department of Transportation (Iowa DOT) is getting the opportunity to use proven innovations to meet the needs of the traveling public during and after construction of the 24th Street Bridge in Council Bluffs, Iowa. Specifically, construction acceleration, durable high-performance materials, innovative contracting methods, intelligent transportation systems, and a health monitoring system are some of the innovations being used.

The 24th Street Bridge over I-80/I-29 in Council Bluffs, Iowa, will feature full-depth post-tensioned concrete deck panels supported on high-performance steel girders. The two-span steel bridge will serve as the primary access to some of the most popular attractions in Western Iowa and will highlight the aesthetic theme for the I-80/I-29 corridor.

This paper focuses on the innovations associated with the design, fabrication, and construction of the 354 ft. by 82 ft. steel bridge. The information presented is of use to bridge owners, designers, and other industry professionals as they strive to make the best use of high-performance materials and the latest construction innovations.

Key words: accelerated construction—precast concrete decks

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

BACKGROUND

Lower revenues and higher costs of construction projects have had a dramatic impact on the transportation infrastructure budget across the nation. Thus, it is important for state departments of transportation to use cost-effective and economically sound methods in dealing with the aging infrastructure that is in dire need of rehabilitation.

The Highways for Life (HfL) initiative and the Innovative Bridge Research and Deployment (IBRD) program is giving the Iowa Department of Transportation (Iowa DOT) the opportunity to use proven innovations that are new to Iowa to meet the needs of the traveling public. Some of the innovations that will be used on this project include accelerated construction techniques in the form of precast components, high-performance materials, innovative contracting methods, intelligent transportation systems, and a structural health monitoring system.

The 24th Street Bridge over I-80/I-29 in Council Bluffs, Iowa, features full-depth post-tensioned deck panels supported on high-performance steel (HPS) girders. The two-span steel bridge will serve as the primary access to some of the most popular attractions in Western Iowa and will highlight the aesthetic theme for the I-80/I-29 corridor.

Several meetings were held in conjunction with this effort, ranging from constructability review meetings with the local industry to technology transfer meetings involving national experts. After meeting with local contractors to explore the feasibility of several accelerated construction concepts, a technology transfer meeting was organized and held in Omaha with participation from the Federal Highway Administration (FHWA), Associated General Contractors, fabricators, Iowa State University, and the University of Nebraska, along with the project team. The purpose of the meeting was generally to foster a transfer of technology among Iowa DOT, the Nebraska Department of Roads, and the FHWA in the area of precast concrete bridge components, with the 24th Street Bridge being of particular focus.

This paper focuses on some of the innovations associated with the design, fabrication, and construction of the 354 ft. by 105 ft. steel bridge. The information presented will help bridge owners, designers, and other industry professionals make the best use of high-performance materials and the latest innovations on future projects.

The 24th Street interchange project is the first part of a multistate (Iowa and Nebraska) effort to improve and upgrade the capacity of the Council Bluffs Interstate System (CBIS). The CBIS is composed of three highly congested corridors: I-80, I-29, and I-480. The 24th Street interchange serves major attractions and businesses such as casinos, a conference/event center, hotels, and major shopping outlets. Therefore, three lanes of traffic, one in each direction plus a turning lane, need to be maintained on 24th Street during construction.

Typically, a project of this magnitude is constructed over two consecutive construction seasons, but due to the critical location of this interchange traffic restriction duration on 24th Street needed to be limited to a single season (April–October). With this in mind, accelerated construction techniques along with innovative methods were primary features for this project, which made it a very attractive candidate for both the HfL and IBRD programs.

DESIGN CONCEPT

Design concept development for the 24th Street Bridge involved many considerations. Several bridge types and construction phasing options were considered to find the best solution to meet design and safety standards, facilitate traffic, and minimize right-of-way impacts. The existing four-span 216 ft. by 64 ft. prestressed concrete beam bridge spans five interstate traffic lanes. The proposed bridge needs to accommodate the future interstate expansion to a 12-lane dual-divided roadway section, with a I-80/I-29 centerline shift of approximately 42 ft. at the bridge. The project concept required 24th Street and I-80/I-29 to remain open during the phased construction of the new bridge.

A key constraint for the 24th Street Bridge was the shifted design location of the I-29/I-80 centerline, as it defined the location of the proposed bridge center pier. The solution for the 24th Street Bridge required that during the first phase of construction traffic be maintained between existing piers and the proposed center pier. Construction of additional piers to reduce span lengths was not feasible when considering existing, staged, and proposed roadway configurations.

The desired solution was the proposed two-span 354 ft. by 105 ft. welded plate girder bridge. Steel girders made this the most feasible option, as the required span lengths of 178.5 ft. and 175 ft. exceed Iowa's prestressed concrete beam standards. Longer spans worked well for the interstate final lane configurations and allowed the flexibility to stage the I-29/I-80 traffic without reducing the number of traffic lanes during the phased construction of the bridge. The steel girder solution offers the contractor the flexibility of installing shear connectors in the field after the placement of deck panels, which provides more tolerance and an opportunity to make any needed adjustments.

Another advantage to the use of steel girders was from an aesthetic point of view. The use of steel girders complemented the use of other aesthetic features on this structure, such as the aesthetic terrace wall features designed to add interest to each bridge berm, as illustrated in Figure 1.

Figure 1. Artist rendition of proposed 24th Street Bridge

The proposed bridge cross section consists of six lanes (two lanes in each direction plus two turn lanes) along with a raised median, raised sidewalk, and raised multiuse trail. While the additional bridge length will accommodate the widening of I-80/I-29, the wider bridge roadway will improve traffic flow on 24th Street.

As discussed previously, the intent of this project is to stage-construct the new bridge in two phases while maintaining traffic on both 24th Street and I-80/I-29 at all times, with the exception of limited night closures. Figure 2 provides a good description of the staging sequence for this construction project. Both phases of construction will be completed in one construction season, with a spring start and fall completion. This aggressive schedule will require the use of innovative design, contracting, and construction techniques.

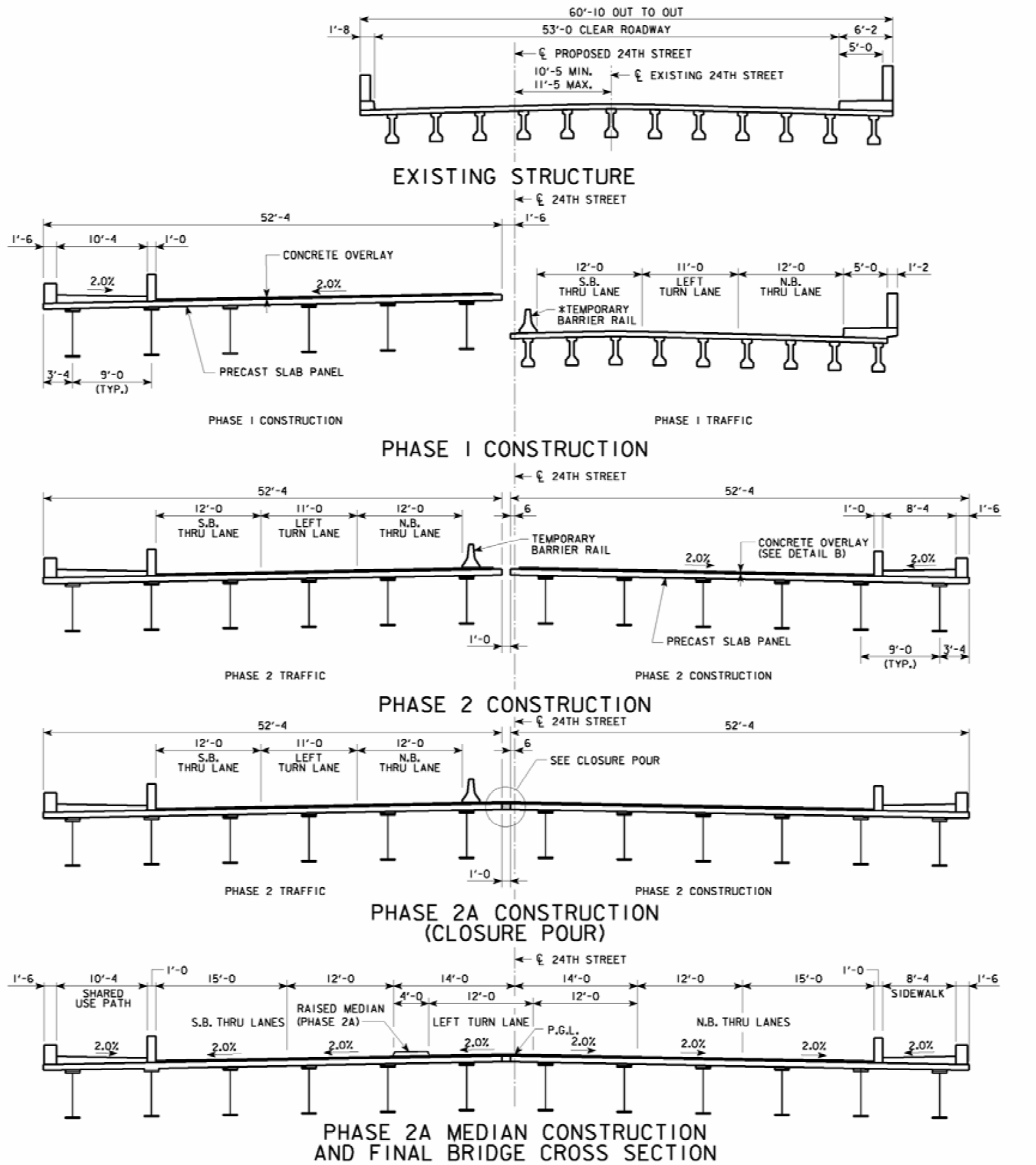

Figure 2. Bridge construction phasing

The two major components for the acceleration of the bridge construction involve (1) the use of full-depth, full-width precast deck panels, in lieu of a traditional cast–in-place concrete bridge deck application, and (2) accelerated contracting techniques (A+B bidding with incentives) along with a delayed construction start to allow lead time for steel fabrication.

The bridge deck uses full-depth precast deck panels with panel widths comprising roughly half of the new bridge width in each of two primary phases. A longitudinal closure pour will be utilized at the juncture between each half of the bridge deck. To improve ridability and provide an additional level of protection for the post-tensioned deck system, the panels will be topped with a high-density concrete overlay.

To ensure that the construction of this bridge will be completed in one construction season, bidders will be required to establish the number of calendar days to be used to complete construction. The product of the number of calendar days multiplied by a predefined daily user cost plus the contract sum will determine the winning bidder (A+B). The maximum allowable number of calendar days will be stated on the proposal. Bids showing time in excess of this maximum will be rejected. With this contracting method, it will be imperative for the contractor to complete construction within the specified days and avoid the cost of additional days.

STEEL FRAMING

The superstructure is made out of precast/prestressed concrete deck panels supported on steel girders. Steel girders were chosen for the main load carrying system due to two main factors: the span lengths and the vertical clearance. Steel also offered other advantages, such as better compatibility with the precast deck concept and the aesthetic theme. The span lengths were above the Iowa DOT limit for the use of their standard prestressed concrete bulb tee beam, and the constraints on vertical profiles required a shallower than optimum girder depth. The use of steel girders enabled greater flexibility to achieve the most feasible bridge design while maintaining the goal of accelerating construction.

The steel girders were designed to act compositely with the deck. This required that the deck be connected to the girders through the use of shear connectors. Composite action was achieved by the use of shear studs grouped together to maximize on the economy of deck panel fabrication and were evenly spaced at 2 ft. along the length of each girder. In addition, the plans allowed the contractor to install the shear studs in the field rather than having the studs installed in the shop, as is traditionally done with cast-in-place deck application. This shear stud installation method will allow the contractor greater tolerance for the erection of the deck panels in an expeditious manner. This is because the deck panels are not required to be set over already installed shear studs, and the contractor would be able to move the panels in all directions to maintain geometry control without being restricted by the location of shop-installed studs. Also, the installation of the shear studs in the field would not fall within the critical path for completion of this project.

The deck slab is supported by 12 lines of steel girders that are spaced at 9 ft. 0 in. on center with a maximum girder length between field splices of 121.75 ft. Because of the restriction to the girder depth, the design required the use of higher strength material than the traditional design. The designer determined the use of higher strength material for the bottom flange and the top flange between the two field splices of the pier section to be the most economical.

HPS 70W steel was used in these areas to take advantage of higher strength coupled with improved toughness and durability. All other steel, including the web, were specified to be A709, grade 50 steel. The web thickness of 1/2 in. required no intermediate stiffeners in the positive moment regions and a minimal amount in the negative moment zones. Cross frame diaphragms were generally spaced at 22 ft. The diaphragms consisted of two angle cross braces between two WT top and bottom struts. A plate diaphragm is specified between the two phases of construction, with one set of girder holes for phase one to be drilled and connected after the phase two superstructure is completed and most of the dead load has been applied.

DECK SYSTEM

After weighing all the factors, it was determined that each deck panel would be 10 ft. long by 52 ft.4 in. wide by 8 in. thick. Each panel will be pretensioned in the transverse direction with (20) 1/2 in. diameter, 270 ksi, low-relaxation strands. There will also be a total of 28 flat ducts embedded in each panel to house the longitudinal post-tensioning. Four 0.6 in. diameter, low-relaxation, 270 ksi strands will be installed in each of the embedded ducts. Pockets will be formed in the panels to accommodate headed shear studs to tie the deck to the girders to provide fully composite action with the deck in the positive moment regions. To provide economy of fabrication, phase one panels are geometrically similar to phase two panels. Each phase of the project will utilize 35 precast deck panels, for a total of 70 panels. The deck panels were designed in accordance with the latest edition of the AASHTO LRFD specifications. A plan view of a typical phase one panel is shown in Figure 3.

The deck panels will be installed after the steel framing (girders and cross frames) have been erected and the slab buildup below the deck panels has been formed. Slab build-up forming methods and the leveling of the panels to the correct elevation are left up to the contractor. However, the plans include optional leveling bolts embedded in the deck panels that could be used by the contractor to aid in setting the panels to the correct elevations (see Figure 4). After all the deck panels for a phase are erected, the transverse joints will be filled with high-strength, non-shrink grout.

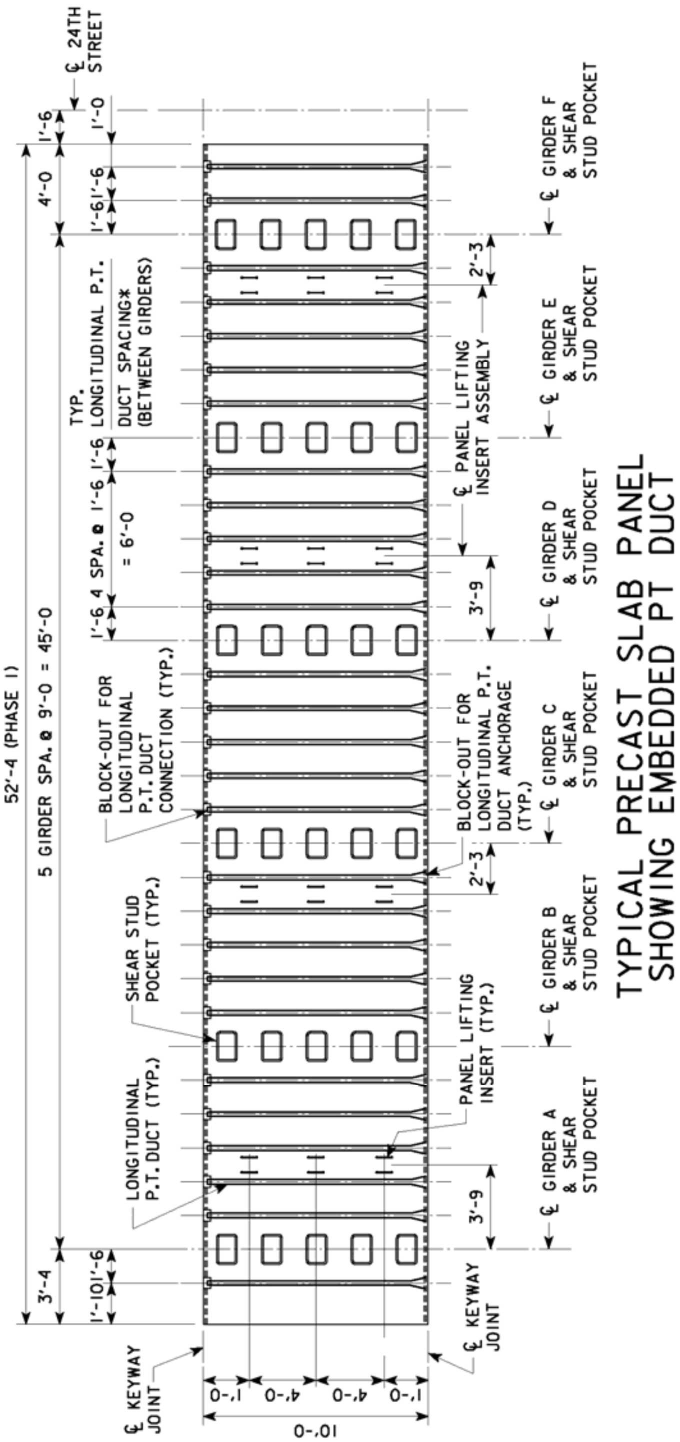

Figure 3. Typical precast slab panel, showing embedded PT duct

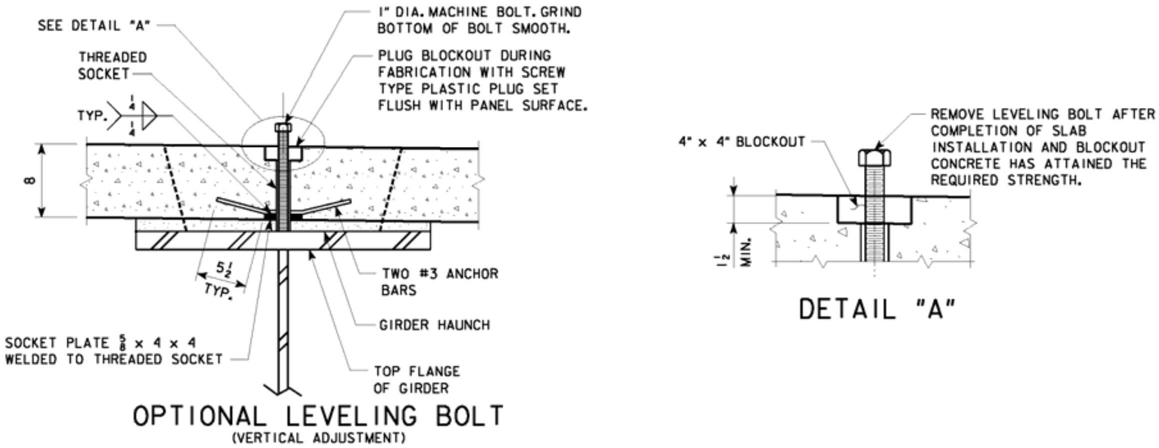

Figure 4. Leveling bolt details

This project utilized a female-to-female transverse joint between panels, eliminating the need for match casting and reducing the risk for damaging panel edges during erection and post-tensioning. The decision to use this type of joint was largely based on Iowa DOT experience with other projects that showed that this type of joint tended to perform better than other type of joints, especially where longitudinal post-tensioning has been utilized. The transverse joint configuration in the panels is a very important aspect to the design and successful service life of the structure. A poor detail of the transverse joint could result in leakage of the joint material and spalling adjacent to the joint. The transverse joint detail along with the blockout for splicing the post-tensioning ducts are shown in Figure 5.

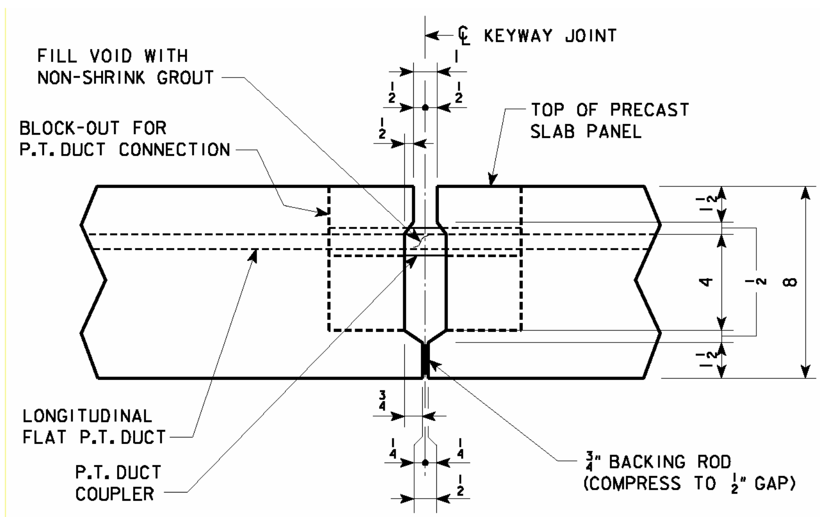

Figure 5. Transverse joint detail

After the grout in the transverse joints attains required strength, the longitudinal post-tensioning force will be applied and locked off. Several factors influenced the amount of post-tensioning force that panels were designed to accommodate. LRFD specifications do not allow any tension in areas where auxiliary reinforcement is not provided. Tension in the panels was caused by the composite dead load, live load, and impact in the negative moment zone near the pier. This criterion controlled the post-tensioning design at the transverse joints between the panels. In addition, the depth of the panel can only accommodate anchorages for four standard strand tendons.

In order to determine the effects of long term losses, mainly creep and shrinkage, a computer model was created to estimate the losses at the end of the service life for the structure. This computer model was checked with hand calculation, utilizing loss formulae in accordance with the latest LRFD specifications. Analysis showed that the age of the panels along with the strength of the panels at the time of post-tensioning have a significant effect on the amount of losses due to long-term creep and shrinkage. In this case, and in order to maintain zero tension at the transverse joints, no more than 23% maximum loss of the post-tensioning force can be accommodated when considering all the factors mentioned previously. For example, a 6,000 psi strength panel would be required to be 100 days old before the post-tensioning force can be transferred to the concrete, while a 12,000 psi panel would need to be 28 days old before post-tensioning. With an October letting date and an expected June erection of the panels, this would require an accelerated winter fabrication schedule and storage of the panels. This could have resulted in an economic disadvantage to the project. To avoid this situation and in order to provide as much flexibility as possible during the construction stage, the contractor was given the option to design a concrete mix that would yield the required design strength while accommodating the contractor's accelerated schedule and minimizing the fabrication costs.

Concurrent with the concreting of the haunch and the shear studs, the sidewalk and barriers could be constructed. To connect the barrier and the sidewalk to the deck panels, threaded inserts were included in the panels and capped so that no reinforcing steel extends out of the panels, thus simplifying, finishing, storing, and shipping.

After the shear stud pockets and the haunch concreting have been completed, the driving surface of the deck will be topped with low-slump, high-density concrete overlay. The same process is then repeated for the second phase of construction.

To tie the two phases of construction together, a longitudinal closure pour near the centerline of the bridge will be cast in place. The longitudinal closure pour is located in the middle of the center girder bay and is designed as a moment connection carrying the full positive moment. The location of the closure pour was dictated by the number of temporary lanes and the location of the girder lines. Another advantage of locating the longitudinal joint in the area of positive moment between the girders is that the top of the joint is always under compression, reducing the possibility of stress cracks and joint leakage. See Figure 6 for the details of the longitudinal joint.

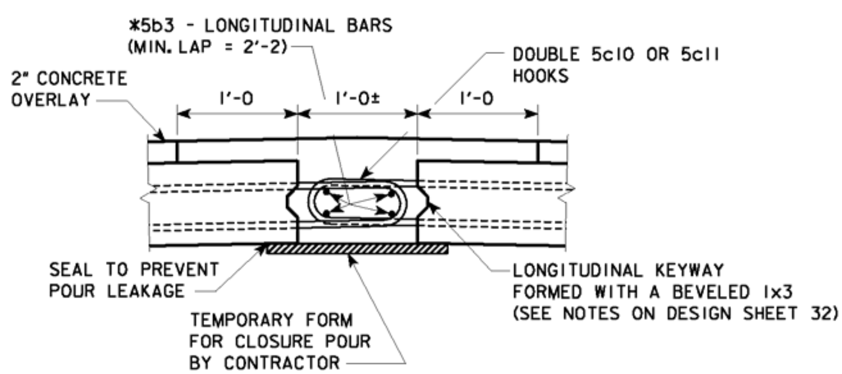

Figure 6. Longitudinal joint closure pour detail

The final step of the bridge construction will be to construct the median with a similar construction method as that used for the barriers and sidewalks.

CONCLUSION

In conclusion, to achieve the goals set for this project the design incorporated details from past projects, as well as the latest in research in the areas of deck panels. Coordination among the designer, the owner, local contractors, and fabricators was key to developing an economical design that could be constructed under an accelerated time frame. Some of the primary goals of this project are to minimize disruption to the traveling public during the reconstruction of the 24th Street interchange and enhance safety during and after construction.

Steel girder framing offered the best compatibility for use with a post-tensioned precast deck system for which construction acceleration and long-term durability were the key objectives. Furthermore, horizontal and vertical clearance requirements in terms of maximizing span length and minimizing superstructure depth, respectively, were best served with the use of steel girders. The natural look of weathering steel blended in with the overall aesthetic features.

As this project is scheduled for letting in October 2007 and construction in 2008, the performance of the innovative components will be closely monitored and evaluated. The success of these innovations, along with lessons learned, will be the basis for implementation on future projects.

Turnberry's Town Square, Las Vegas: Elevated Left Turn Access

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ABSTRACT

The Las Vegas Resort Corridor is the economic center of the Las Vegas Valley. As resident and tourist populations of the Valley increase, traffic to and from the Resort Corridor is also expected to increase. To minimize traffic conflicts and optimize signal operations between Sunset Road and the I-215 interchange on Las Vegas Boulevard, Kimley-Horn and Associates, Inc. proposed and designed a grade-separated left turn to serve the new Town Square regional retail development.

Key words: elevated—flyover—grade—left turn—separation

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PROJECT LOCATION

The proposed Town Square development is located west of Las Vegas Boulevard, east of I-15, north of the 215 Beltway, and south of Sunset Road on approximately 97 acres within Clark County, Nevada. The proposed development is expected to contain approximately 1.5 million square feet of commercial uses. The $750 million retail/commercial center is expected to contain an open air mall and entertainment center along Las Vegas Boulevard, “the Strip.” Since the development’s location is within the northeast quadrant of a system-to-system interchange, access to the site is primarily limited to two arterial streets, Las Vegas Boulevard and Sunset Road.

ENGINEERING PRINCIPLES

The Town Square elevated left turn access project utilized engineering ingenuity to solve the unique challenges associated with providing access to the development site while maintaining critical traffic progression along Las Vegas Boulevard.

When the project was in the development phase, access concerns were raised due to the project’s location on Las Vegas Boulevard just north of the 215 Beltway interchange. Existing traffic volumes in the vicinity of the project were expected to increase significantly prior to completion. As the project was being conceptualized by the owner and the consulting team, there were plans for three new signals along the project’s Las Vegas Boulevard frontage. With the development of Synchro simulation models by Kimley-Horn and Associates, Inc., it was quickly shown to the developers of the project that the installation of the three signals could not adequately move traffic along Las Vegas Boulevard as well as into and out of the site. In consultation with the multiple agencies involved with the project (Clark County, Nevada Department of Transportation, and Regional Transportation Commission), three different access alternatives were considered for the southern access into the property: full signalization, a half-signal, and a grade separation (over or under Las Vegas Boulevard). After review of the three alternatives, a grade-separated access into the site was agreed upon by both the developer and reviewing agencies.

The application of grade separation on arterial streets is not necessarily a new concept. In other major cities, grade-separated accesses exist; however, these accesses typically occur below grade. The Town Square elevated left turn concept is unique because the overhead bridge structure elevates out of the left-turn pocket and travels over a major arterial. This is the first of its kind in Nevada and provides a unique alternative to half-signal design (as right-in/right-out movements are allowed below the structure). This concept can be added to the traffic engineering toolbox to aid in direct site access over busy arterial corridors.

SOCIAL SIGNIFICANCE

The Town Square elevated left turn access is taking an existing level of service (LOS) F left-turn movement into an existing Fry’s Electronics Store, adding traffic from a major retail/entertainment district, and creating an unobstructed LOS A left-turn movement into a major regional shopping and entertainment facility. The addition of the elevated left turn is expected to ease existing driver frustration and aggression by minimizing delay on Las Vegas Boulevard. The decrease in delay is also expected to reduce air quality impacts. Overall, the elevated left turn facility will provide improved access into a major traffic generator of a retail/entertainment district.

PROJECT COMPLEXITY

Traffic Volumes

In 2005, the annual average daily traffic (AADT) volumes on Las Vegas Boulevard were 42,000 vehicles per day. Without the project, Las Vegas Boulevard is anticipated to have approximately 57,000 vehicles per day. The project is estimated to generate approximately 63,000 daily trips, of which 2,250 are anticipated to occur in the a.m. peak hour and 6,000 are anticipated to occur in the p.m. peak hour, in addition to the existing Fry's Electronics Store trips. The combined Town Square and Fry's Electronics Store left-turn access through the grade-separated left turn is expected to be approximately 575 a.m. peak hour trips and 950 p.m. peak hour trips. This elevated left turn is expected to carry these left-turn movements out of conflict with 1,000 a.m. and 2,500 p.m. peak hour trips traveling southbound along Las Vegas Boulevard.

Structure

The elevated left turn is being constructed as a three-span structure with retaining walls on both sides. The three-span structure was chosen for structural efficiency due to the horizontal curvature (R=205 ft.), stability, and superelevation of the left turn (4%) for a 25 miles per hour (mph) design speed. The structure was designed as a post-tensioned concrete box girder as opposed to steel girders or precast girders. The span for the structure over the southbound lanes on Las Vegas Boulevard is 200 ft. to accommodate a future regional fixed guideway system proposed within the Las Vegas Boulevard center median.

Design Considerations

The design speed for the elevated left turn was a topic of concern for the parties involved. Several different design speeds were explored; however, a final design speed of 25 mph was used for vertical and horizontal design. The design speed was based on the idea that vehicles are entering a left-turn pocket, and if a signal were present the vehicles would travel at a slow speed. Also, the elevated left turn structure terminates in a shopping center ring road. Designing the structure to a high design speed would cause vehicles to enter the site at significantly faster speeds than desired, thus requiring additional onsite traffic calming measures.

The main challenge in the design of the elevated left turn was to allow sufficient space in the left-turn bay for vehicles to slow down before entering the elevated left turn structure while still providing a traversable grade on the approach to achieve a 17 ft. vertical clearance over Las Vegas Boulevard. Rumble strips will be located within the left-turn bay to encourage the slowing of vehicles prior to the approach of the elevated left turn. The maximum grade on the approaches is 8%, with a left-turn bay of approximately 250 ft.

Construction

Throughout construction of the elevated left turn, the developer was required to maintain two lanes of travel in each direction along Las Vegas Boulevard, as well as continue to provide left-turn access for the Fry's Electronic Store customers. At the time this summary was prepared, construction of the elevated left turn was underway (construction started in January 2007), and the travel lanes were being maintained. The construction period is compressed to four months, with a scheduled completion of June 2007. This date is months ahead of the opening of Town Square, which is scheduled to open in October 2007. The advance completion is to aid in reducing the existing Fry' Electronic Store traffic access congestion. The following figures show progress of the construction of the elevated left turn.

Figure 1. View looking at abutment #1 from top of falsework

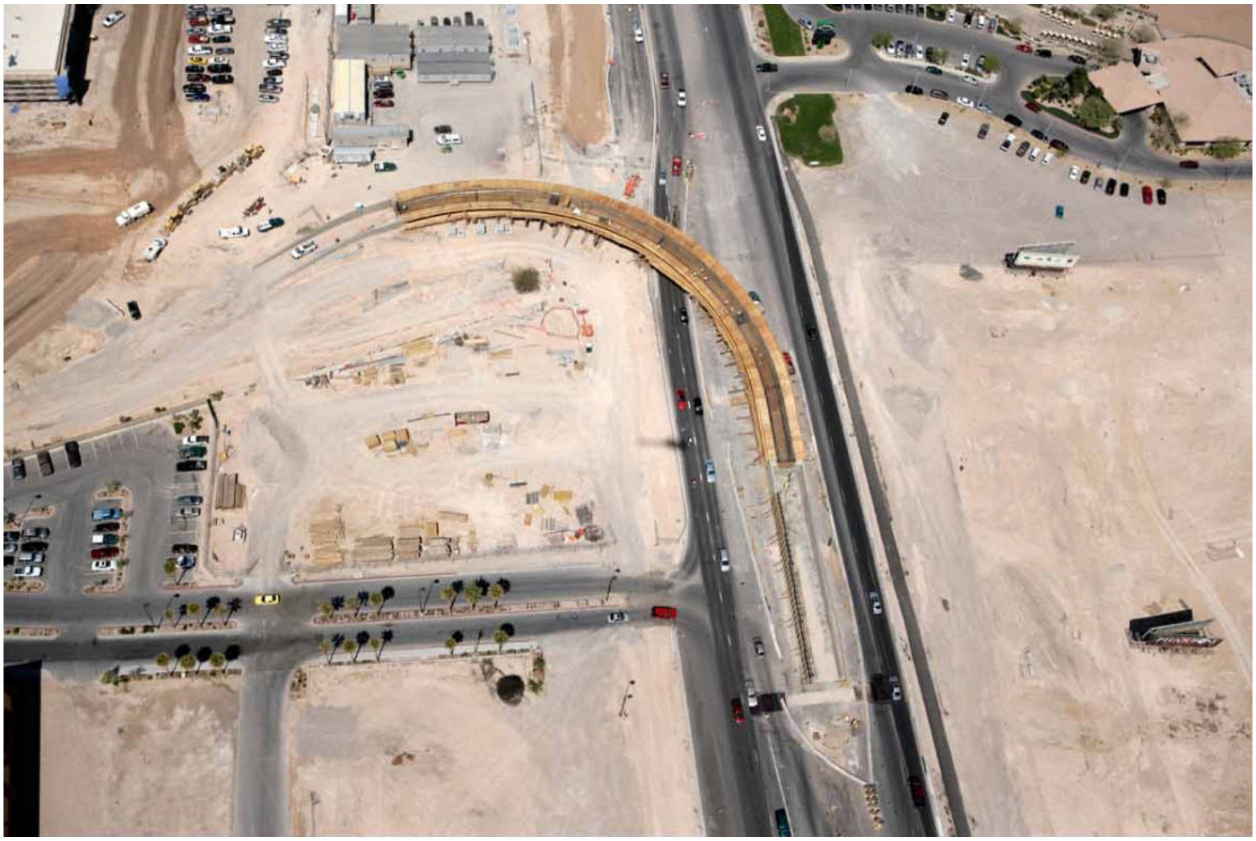

Figure 2. Aerial view of elevated left turn

Figure 3. Aerial view of elevated left turn looking northbound on Las Vegas Boulevard

Figure 4. Looking east at the elevated left turn

SUMMARY

A first of its kind in Nevada, this proposed median structure is replacing the existing at-grade left turn at Fry's Electronics Store and eliminates the conflict with southbound traffic on Las Vegas Boulevard. The project clearly demonstrates sound engineering principles by using engineering ingenuity to solve the unique challenges associated with providing access to the development site while maintaining vehicle progression along Las Vegas Boulevard. The addition of the elevated left turn is expected to ease existing driver frustration and aggression by minimizing delay on Las Vegas Boulevard while maintaining a free-flow access into a new regional shopping and entertainment district. Since this concept is the first of its kind in Nevada, special consideration had to be used in designing the design speeds, slopes, and construction traffic plans. In the future, this concept can be included in the traffic engineering toolbox to aid future designs in providing direct site access over busy arterial corridors.

Local Calibration for Fatigue Cracking Models Used in the Mechanistic-Empirical Pavement Design Guide

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ABSTRACT

This paper identified two important calibration factors for a Midwest implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG). The calibration factors are for the fatigue damage model in flexible pavements. The gathering of the data required for calibration is labor intensive because the data resides in various and incongruent data sets. Spreadsheet templates specifically designed to manage the calibration data were used to collect pavement performance data from state transportation agencies in Michigan, Ohio, and Wisconsin. Calibration factors were then derived by minimizing the differences between observed and predicted pavement performance. The pavement performance field data in Wisconsin were employed for calibration initially, and the distresses predicted with these calibration factors were compared to pavement field performance in the other states. These calibrated models in the MEPDG assure the reliable prediction of pavement distress, such as longitudinal and alligator cracks.

Key words: fatigue cracking prediction—local calibration—Mechanistic-Empirical Pavement Design Guide

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INTRODUCTION

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is a new product that was developed to enhance and improve existing pavement design procedures. It will result in transitioning from the existing empirically based pavement design procedure to a mechanistic-empirically based procedure that will combine the strengths of advanced analytical modeling capacity and observed field performance. The model parameters are based on data collected from a few pavement test sites and full-scale testing facilities. These performance models are key elements in the accuracy of the design results and thus warrant detailed validation and calibration, particularly with regard to the effect of local climate and pavement structure conditions.

This paper presents the results of a regional pooling effort for the purpose of calibrating the MEPDG models for the Midwest. Regional pooling of performance data is not an easy task because it requires coordination among participating states, uniformity in data collection, similarity of database structures, and a centralized approach for data analysis and reporting. To collect the pavement data from the states in the Midwest region, a uniform database format was developed, and state highway agencies were asked to fill out the form. The collected pavement date from Michigan, Ohio and Wisconsin were applied to evaluate the calibration factors in the fatigue cracking models for the Midwest region. The collected data was fed in to the MEPDG software, and a comparison of the output from the program to actual pavement performance enabled validation and calibration.

BACKGROUND

The most widely used procedure for pavement design is the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (AASHTO 1986; AASHTO 1993). A few states use the 1986 or 1972 AASHTO guidelines. Some states have developed their own design procedures, some based on mechanistic-empirical procedures (Khanum et al. 2005). For example, the Wisconsin Department of Transportation (WisDOT) developed its own pavement design procedure based on the 1972 AASHTO Interim Guide.

The design methodologies in all versions of the AASHTO guide are based on the empirical performance equations developed using AASHO road test data from the late 1950s (Khanum et al. 2005). Those equations are based on the obsolete construction methods, materials, and loads of the 1950s. The limitations of earlier versions motivated the development of the MEPDG through NCHRP project 37-1(A).

MEPDG Procedure

The MEPDG combines empirical and mechanistic procedures. Mechanistic methods are used to predict pavement responses, and pavement performance is predicted based on performance data collected from real-world pavements. Figure 1 illustrates the design procedure in the MEPDG. The designer first considers the pavement construction (structure) and site conditions (material, traffic, climate, and existing pavement condition, in the case of rehabilitation). The designer then selects a trial design, including the number of total layers, thickness of each layer, and choice of material. From these inputs, the design procedure mechanistically calculates structural responses: stress (σ), strain (ε), and deformation (δ). From these calculated responses, damages are projected during design life and accumulated monthly by pavement performance models. The MEPDG allows the designer to calibrate pavement performance models depending on environmental factors such as traffic and climate. Well-calibrated prediction models result in reliable pavement designs and enable precise maintenance plans for state highway agencies

(Carvalho and Schwartz 2006). Local pavement performance data can be used to validate and adjust calibration factors integrated in the MEPDG. The procedure empirically relates damage over time to pavement distress and smoothness level, as chosen by the designer. The key damage features and smoothness problems are surface cracking, fracture, fatigue, rutting, and roughness. With selection of calibration and design reliability levels, the trial design is then evaluated against some predetermined failure criteria. If the trial design does not meet desired performance criteria at a predetermined level of reliability, it is modified and the evaluation process is repeated as necessary (NCHRP 2004).

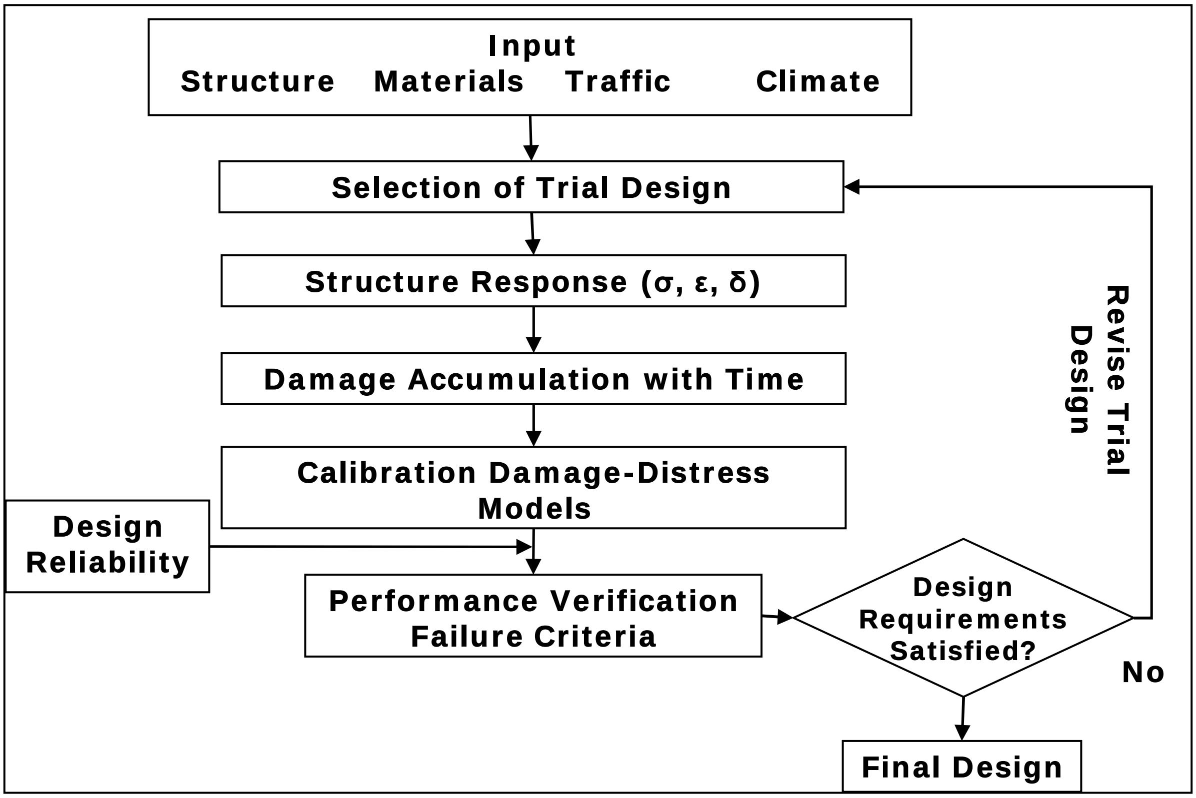

Figure 1. MEPDG procedure (NCHRP 2004)

PAVEMENT DATA COLLECTION

The pooling of pavement material, structure, and performance data from multiple states requires coordination with the participating states, a common data collection format, and similar levels of data availability. Thus, for this project, a uniform data collection format was established and delivered to the participating state agencies. The uniform format was created based on Appendix EE of the MEPDG. In considering data gathering familiarity for the participating state agencies, Microsoft Excel spreadsheets were determined to be the best format for gathering the pavement data, since they have functions that help the agencies complete the forms. The Excel file consists of five different work sheets: general project information, traffic, climate, pavement structure/material, and pavement performance. The first four sheets are for the input data required for the MEPDG program, and the last sheet, pavement performance, is for comparing output from the software to measured field data. Comparison of the output from the software to field data allowed the reviewing and adjusting, if necessary, of calibration factors in the MEPDG distress models.

Data Quality and Assumptions for Wisconsin Pavement Sections

To select sections from which to gather data in Wisconsin, WisDOT's primary pavement performance database, the Pavement Information Files, was used. When the representative sections were selected, three criteria were considered: (1) sections with severe distresses, (2) sections with no rehabilitation and no overlay, and (3) sections more than five years old. Sections with severe distresses were defined as having total rutting greater than 0.25 in., an International Roughness Index (IRI) value greater than 172 in./mile, and a Pavement Distress Index (PDI) value greater than 65. PDI is a mathematical expression for pavement condition rating that is keyed to observable surface distresses in Wisconsin. The PDI number (0 for best condition and 100 for worst) is used to summarize the level of distress and is used primarily for network-level evaluation (in WisDOT's PDI Survey Manual). Table 1 shows the specific values of criteria for selecting the sections in Wisconsin.

Table 1. Initial selection criteria in Wisconsin

Distress	Criteria value	Mark
Rutting	0.25"	Default limitation value for failure in MEPDG
IRI	172 in/mile	Default limitation value for failure in MEPDG
PDI	65	Level when WisDOT recommends maintenance on Principal Arterials

It was not an easy task to collect the available pavement information required to run MEPDG. Due to the lack of available pavement information, only nine sections were employed for flexible pavement. Table 2 lists the representatives sections with pavement performance and the required data available. The sequence number shown is the primary key in the PIF database used to identify each section.

Table 2. Wisconsin sections with significant distress in 2006 (flexible pavement)

			Pavement Performance					
Sequence #	County	High way #	Rutting (in.)	IRI (in./mi)	Alligator cracking (%)	Transverse cracking (number/sta*)	Longitudinal cracking (ft./sta*)	PDI**
23010	DANE	19	0.25-0.5	214	50-74	1-5	1-100	70
34230	PIERCE	29	0.25-0.5	234	25-49	6-10	201-300	92
98490	GRANT	80	0.25-0.5	259	25-49	6-10	1-100	83
133580	OUTAGAMIE	187	0.25-0.5	274	50-74	1-5	101-200	93
33620	SHEBOYGAN	28	0	198	50-74	0	1-100	95
34240	PIERCE	29	0.25-0.5	192	25-49	6-10	201-300	83
113040	BROWN	96	0.5-1	46	1-24	6-10	101-200	88
133510	OUTAGAMIE	187	0	227	25-49	6-10	101-200	81
136706	WAUKESHA	164	0	56	0	6-10	1-100	22

* sta: station (100ft = 0.21 mile per station on average)
** PDI: Pavement Distress Index

As-built plans were the major source for obtaining pavement profile, such as number of layers and material properties, and PIF provided WisDOT with the pavement performance history. After obtaining as-built plans, however, the research team discovered that resurfacing had been done, and overlays had been applied, to some of the sections. An irregularity in the distress measures was also recognized. Occasionally, distress quantities appeared to increase then drop back down without explanation. After discussion with WisDOT's pavement design experts, two explanations were possible: First, minor maintenance may have been applied. Minor maintenance activities are not considered restoration or reconstruction that can be designed by the MEPDG. Such operations usually focus on the ride quality, rather than structural improvement. The distresses thus seem to disappear for awhile, but they rise a few years later. Second, the irregularity may be due to human factors when distresses are observed. Prior to 1999, the pavement performance data (except IRI) was collected manually by pavement crews in each region and then sent to the central office. This, by itself, induces variability. In 1999, WisDOT purchased new equipment to collect both IRI and pavement distress data. Both the use of new equipment and the removal of regional variability caused adjustments to the PIF data. Thus, it was decided that only the data collected after 2000 were to be applied for calibration study.

Michigan and Ohio Pavement Sections

The pavement database structures were delivered to the states as Excel spreadsheet files, and the state agencies were asked to complete the spreadsheets for at least five flexible pavement sections. Michigan and Ohio delivered five sections for flexible pavement. Even though the states made efforts to provide the pavement data, critical obstacles were encountered in conducting the calibration. Required data items for running MEPDG were missing, and some of the data collected by the state transportation agencies were not applicable to MEPDG. For these missing data, the default values in MEPDG were used.

The pavement performance data from Michigan and Ohio show trends in irregularities similar to those observed in Wisconsin. Most of the representative sections were constructed in the late 1980s and early 1990s. It is not likely these sections have been rehabilitated. Given the unresolved irregularities in the Ohio and Michigan data, Wisconsin's data was used for calibration. After determining calibration values with Wisconsin's data, the field data from Ohio and Michigan were compared to the prediction models using default calibration values in the MEPDG and to the prediction models using calibration values for Wisconsin data. The comparisons will show whether other states best fit the Wisconsin or default model. The comparison will also show the deviations between actual field data and the prediction models.

CALIBRATION OF THE FATIGUE CRACKING MODEL

The calibration factors in the MEPDG prediction models can be determined by analyzing corresponding field performance data. The calibration factors are adjustable and known to depend upon conditions such as climate, loads, and pavement structure. Climatic and material sources vary regionally, and thus there is some logic to calibrating the models on a regional basis. The calibrations are done by comparing the collected pavement performance with the predicted pavement performance. The default values in the MEPDG were applied initially, and then the calibration factors were adjusted to reduce the difference between collected/observed and predicted pavement performance. The best fit minimizes the difference between MEPDG prediction and observed performance.

Calibration of the fatigue cracking model in the MEPDG was conducted based on the model presented in Appendix II-1 of the MEPDG (NCHRP 2004) and in a Transportation Research Board conference paper by El-Basyouny and Witczak (2005). Accordingly, fatigue cracking prediction is based on the cumulative damage concept. The damage is calculated as the ratio of cumulative predicted load repetitions from traffic to the allowable number of load repetitions. The damage for fatigue cracking is expressed as a percentage. Theoretically, fatigue cracking occurs when accumulated damage is 100%. The equation for calculating the damage for fatigue cracking is as follows:

$$D=\sum_{i=1}^{T}\frac{n_i}{N_i} \tag{1}$$

where

$D = \text{Damage}$
$T = \text{total number of periods}$
$n_i = \text{actual traffic for periods } i$
$N_i = \text{allowable failure repetitions under conditions prevailing in period } i$

The general mathematical form for the number of load repetitions is also shown in the MEPDG. The form of the model is a function of the tensile strains at a given location and the modulus of the asphalt layer (El-Basyouny and Witczak 2005; NCHRP 2004).

$$N_f = \beta_{f1} k_1 (\varepsilon_t)^{-\beta_{f2} k_2} (E)^{-\beta_{f3} k_3} \quad (2)$$

where

N_f = Number of repetitions to fatigue cracking
ε_t = Tensile strain at the critical location
E = Stiffness of the material (psi)
$\beta_{f1}, \beta_{f2}, \beta_{f3}$ = calibration parameters.
k_1, k_2, k_3 = material constants from laboratory testing

Here, β_{f1} , β_{f2} , β_{f3} are the calibration parameters to be determined. According to the literature, β_{f1} is assumed to be 1 unless the asphalt concrete layer thickness is less than 3 in. In this study, because the total thickness of the asphalt layer is more than 3 in., β_{f1} is assumed to be 1 for all sections. As recommended in the literature, the calibration should be done by running the software for combinations of calibration factors β_{f2} , β_{f3} Following the MEPDG, three values of β_{f2} and three values of β_{f3} were applied for calibration. Hence, total runs were nine times per section. The runs were conducted for values of 0.8, 1.0, and 1.2 for the calibration factor on the strain (β_{f2}) and values of 0.8, 1.5, and 2.5 for the modulus calibration factor (β_{f3}) for the MS-1 model (NCHRP 2004). Table 3 lists the possible combinations of calibration values.

Table 3. All combinations of calibration values for fatigue cracking model

Number	β_{f2}	β_{f3}
1		0.8
2	0.8	1.5
3		2.5
4		0.8
5	1.0	1.5
6		2.5
7		0.8
8	1.2	1.5
9		2.5

Comparison of predicted to actual percent damage in the pavement should deliver the appropriate calibration values. However, field data on percent damage is not available. State highway agencies monitor fatigue damage through visible distresses in the pavement, such as longitudinal and alligator cracks. Thus, fatigue calibration values must be related to those visual distresses.

Calibration of the Longitudinal Fatigue Cracking Model

The damage transfer function used in the MEPDG for longitudinal (surface-down) fatigue cracking is in the form shown as follows.

$$F.C. = \left(\frac{1000}{1 + e^{C_1 - C_2 * LogD}} \right) * (10.56) \tag{3}$$

where

$F.C.$ = fatigue cracking (ft/mile)
D = Damage in percentage
C_1, C_2 = regression coefficients

In the MEPDG, the regression coefficients, C_1 and C_2, were evaluated using a Microsoft Solver numerical with more than 100 sections nationwide, and the default values of each C (C_1=7.0, C_2=3.5) were decided to be used for this study. Damage in percentage, D, can be calculated by the fatigue cracking model. All combinations of calibration values were applied to discover the best ones. Nine runs for each section were conducted, and the outputs were evaluated by the sum of squares (SS) for each plot. SS is defined as follows:

$$Sum\ of\ Square\ (SS) = \sum_{i=1}^{n} (Output\ from\ ME\ PDG - Observed\ Field\ Value\ in\ PIF)^2 \tag{4}$$

where n = number of data points.

Nine trials for each section resulted in two sets of betas, one for each section, that minimized the SS values for longitudinal cracking (β_{f1}=1.0 β_{f2} =0.8 β_{f3}=0.8 and β_{f1}=1.0 β_{f2} =1.2 β_{f3}=1.5). Figure 2 and Figure 3 are the plots of the output from the MEPDG for various combinations of the calibration factors. The numbers in parentheses are beta values β_{f1}, β_{f2}, and β_{f3}, respectively. The default beta values are (1.0, 1.0, 1.0). For reference, "PIF" denotes the field-observed pavement performance.

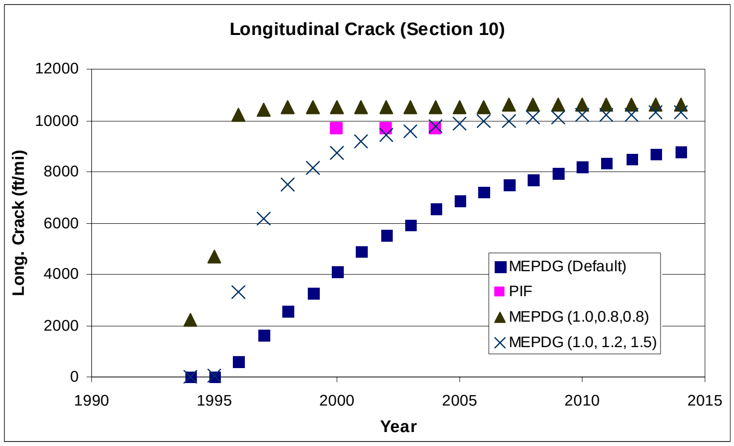

Figure 2. Predicted longitudinal cracking in Wisconsin Section 10 for various calibration values

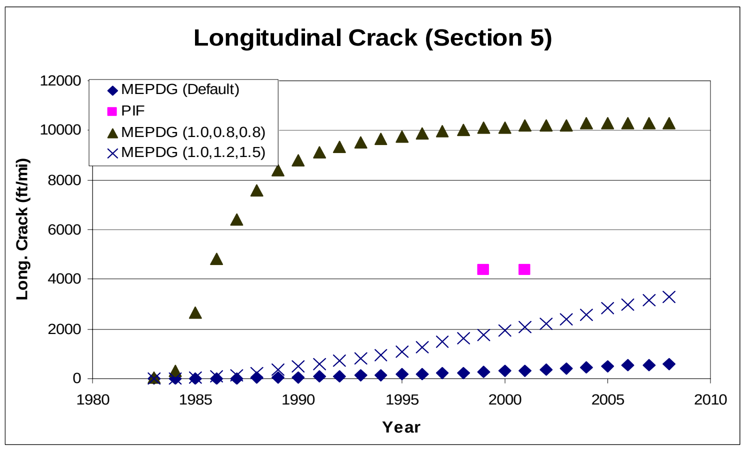

Figure 3. Predicted longitudinal cracking in Wisconsin Section 5 for various calibration values

SS values for three beta sets, including default values, were calculated and compared. The results indicate that a prediction model with β_{f1}=1.0, β_{f2}=1.2, and β_{f3} =1.5 has the smallest SS value and is thus the best fit. To investigate further, the possible combinations of calibration factors were applied for all sections. Figure 4 though 6 show plot comparisons of actual pavement performance versus predicted pavement performance for each calibration set. Ideally, if there is no difference between actual and predicted performance, the data points should fall on the 45 degree line (y=x in graphs). From the figures, the plot in which β_{f2}=1.2 and β_{f3} =1.5 shows the best fit for longitudinal cracking, which is consistent with the SS analysis.

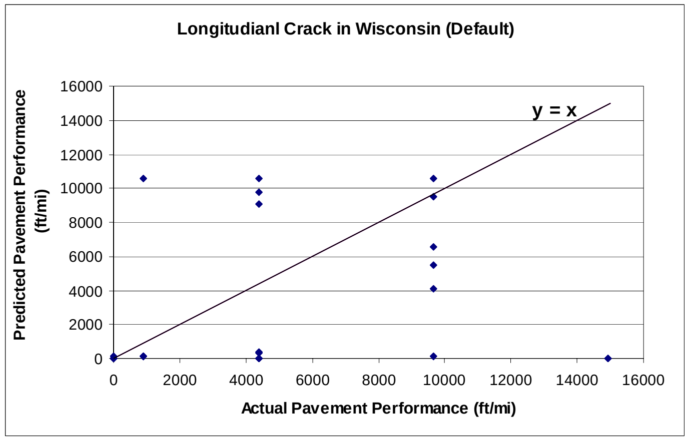

Figure 4. Longitudinal cracking comparison plot for Wisconsin (default)

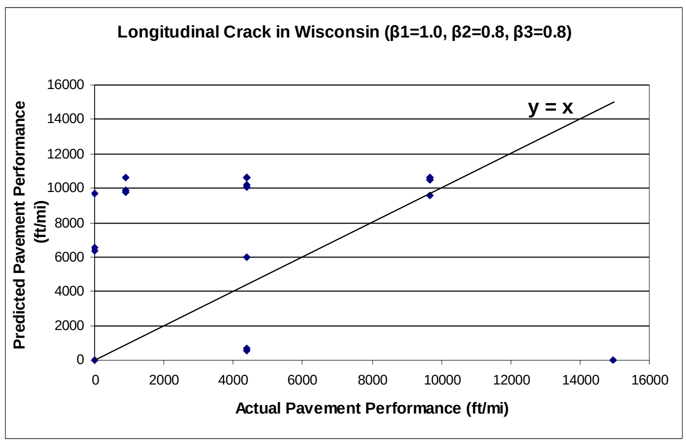

Figure 5. Longitudinal cracking comparison plot for Wisconsin (β_{f1}=1.0, β_{f2}=0.8, β_{f3} =0.8)

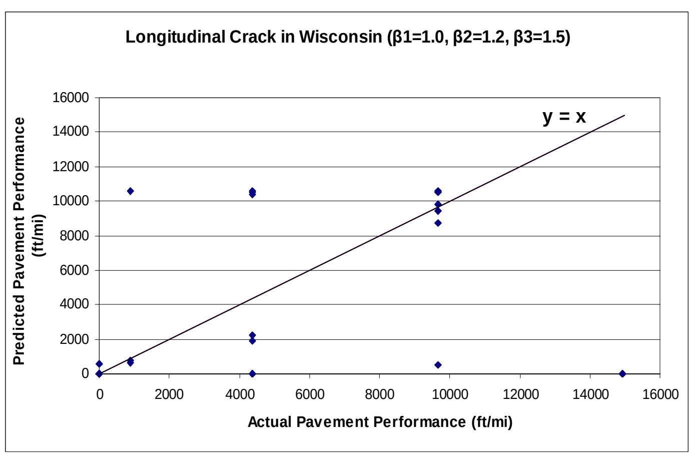

Figure 6. Longitudinal cracking comparison plot for Wisconsin (β_{f1}=1.0, β_{f2}=1.2, β_{f3} =1.5)

Calibration of Alligator Fatigue Cracking Model

The fatigue cracking-damage transfer function used in the calibration of the alligator (bottom-up) cracking is presented in the MEPDG as follows:

$$F.C. = \left(\frac{6000}{1+e^{C_1 - C_2 * LogD}}\right) * \left(\frac{1}{60}\right) \qquad (5)$$

where

$F.C.$ = fatigue cracking (% of lane area)

D = Damage in percentage

C_1, C_2 = regression coefficients

Similar to longitudinal cracking, the default values, C_1 and C_2, are used in this calibration. Damage in percentage, D, depends on the fatigue cracking model. To find the calibration values for alligator cracking, nine runs were conducted for each section. Comparing the outputs from the MEPDG for various calibration factors determines the β_{f2} and β_{f3} with the least SS of errors. Two sets of betas (β_{f1}=1.0 β_{f2} =0.8, β_{f3}=0.8, and β_{f1}=1.0, β_{f2} =1.2, β_{f3}=1.5) for Section 10 and three sets of betas (β_{f1}=1.0, β_{f2} =0.8, β_{f3}=0.8; β_{f1}=1.0, β_{f2} =1.2, β_{f3}=1.5; and β_{f1}=1.0, β_{f2} =1.0, β_{f3}=1.5) for Section 5 have a high chance of reducing the SS values for alligator cracking. Figures 7 and 8 show the plots of the output from the MEPDG after changing the calibration values. Again, the output with default calibration values is also shown in the figures. The number by "MEPDG" represents the beta values (β_{f1}, β_{f2}, and β_{f3}), and "PIF" denotes the collected pavement performance data in the figures.

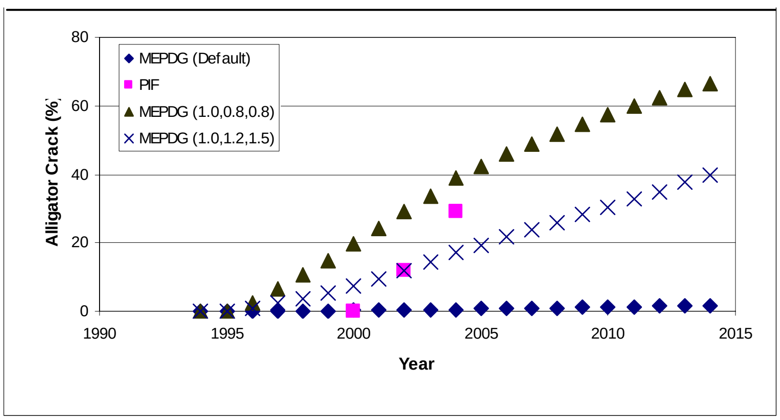

Figure 7. Alligator cracking in Section 10 in Wisconsin by various calibration values

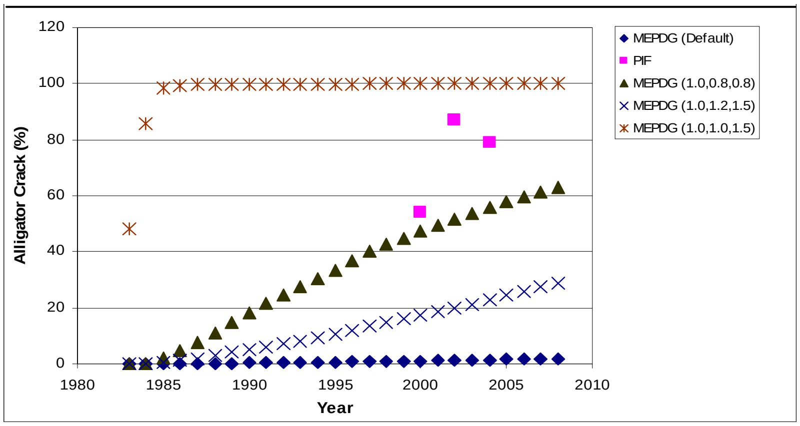

Figure 8. Alligator cracking in Section 5 in Wisconsin by various calibration values

The prediction model with β_{f2}=1.2 and β_{f3} =1.5 shows the smallest SS values for Section 10 and β_{f2}=0.8 and β_{f3} =0.8 for Section 5. Two sets of calibration values (β_{f2}=0.8, β_{f3} =0.8 and β_{f2}=1.2, β_{f3} =1.5) were applied to other sections. Comparison graphs were plotted showing actual pavement performance vs. predicted pavement performance. Ideally, if there is no difference between actual performance data and predicted performance data, the points should be on the perfect 45 degree line. The comparison plots with default calibration factors (β_{f2}=1.0 and β_{f3} =1.0) and the best fit calibration factors (β_{f2}=0.8, β_{f3} =0.8 and β_{f2}=1.2, β_{f3} =1.5) are shown in Figures 9 to 11.

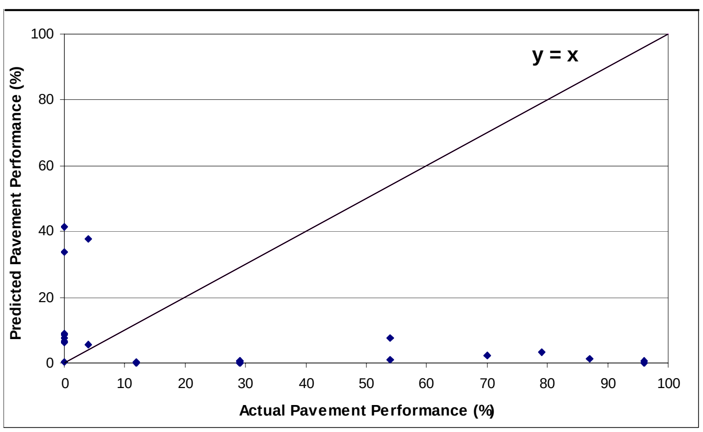

Figure 9. Alligator cracking comparison plot in Wisconsin (default)

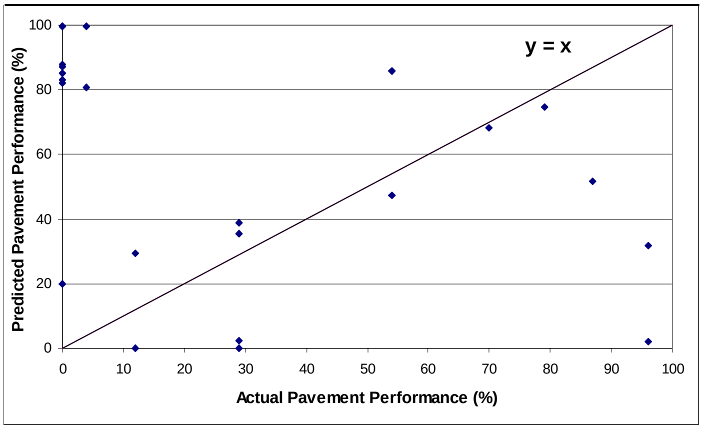

Figure 10. Alligator cracking comparison plot in Wisconsin (β_{f1}=1.0, β_{f2}=0.8, β_{f3} =0.8)

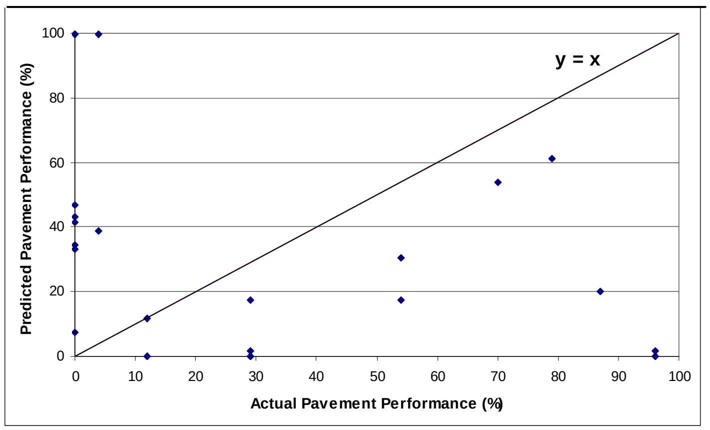

Figure 11 Alligator cracking comparison plot in Wisconsin (β_{f1}=1.0, β_{f2}=1.2, β_{f3} =1.5)

From these three figures, it is difficult to determine which set is the best. Clearly, the plot with default values is the worst. But the plots in Figure 10 and Figure 11 were spread out. Thus, it can be said that both of the calibration sets can be applied to alligator cracking in Wisconsin. However, one is not allowed to input different calibration factors for longitudinal cracks and alligator cracks in the MEPDG. Thus, the proper calibration values for fatigue cracking were concluded: β_{f2}=1.2 and β_{f3} =1.5.

Calibration Fit for Michigan and Ohio

The calibration values were determined only from Wisconsin data. Due to time and budget limitations, calibration factors for Michigan and Ohio could not be determined. Instead, this section presents comparisons of the following calibration methods for both states: predicted performance using the MEPDG, predicted performance using Wisconsin's calibrated model, and observed pavement field performance. Because only two calibration values were determined using Wisconsin fatigue data, only two distresses are presented: longitudinal cracking and alligator cracking.

The calibrated MEPDG model predicted longitudinal cracking poorly for Michigan. Figure 12 and Figure 13 compare the outputs of longitudinal cracking from the MEPDG and field-collected pavement performance data. Both plots show that neither of the two predictions from the MEPDG is accurate. As is especially evident in Figure 13, the prediction using the default values is better than that using the calibrated values.

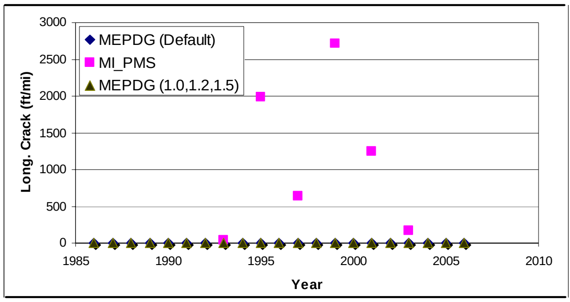

Figure 12. Longitudinal cracking in Section 2 in Michigan

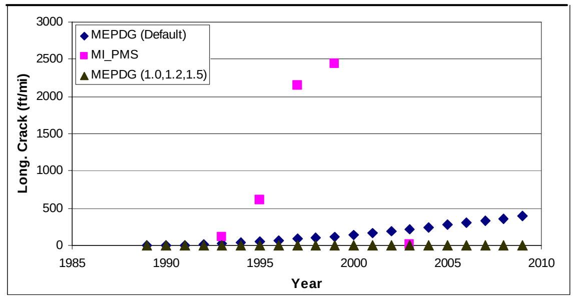

Figure 13. Longitudinal cracking in Section 5 in Michigan

Unlike with longitudinal cracking, the calibrated MEPDG model predicted alligator cracking well for Michigan. Figure 14 shows that the calibrated prediction model can reduce minimize the difference between the prediction and the field-collected data. Figure 15 suggests that the prediction using the default calibration model is better than that obtained using the calibrated values. However, if the deterioration rate of field data is considered, a prediction using calibrated values may match field data better than a prediction obtained using default values.

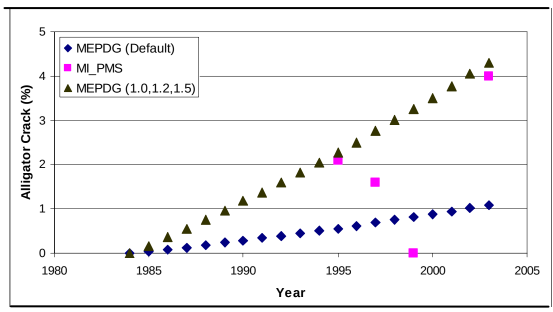

Figure 14. Alligator cracking in Section 1 in Michigan

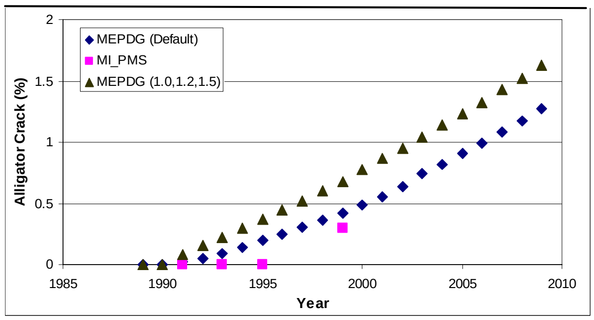

Figure 15. Alligator cracking in Section 5 in Michigan

The collected field data from Ohio does not seem suitable for calibration. As can be seen in Figure 16 and Figure 17, the collected longitudinal data stay at “0” or rise quickly and reach 6,000 ft./mi. in only a couple of years. Thus, it is difficult to judge whether calibrated prediction is good for longitudinal cracking in Ohio.

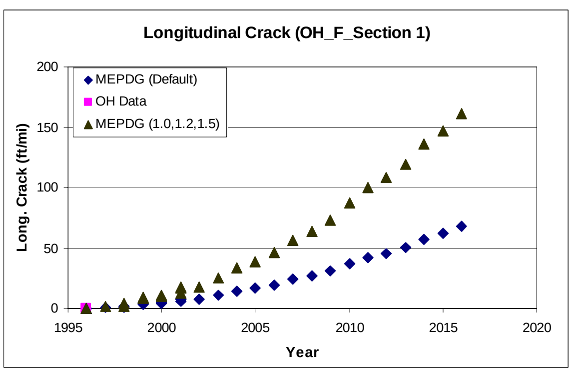

Figure 16. Longitudinal cracking in Section 1 in Ohio

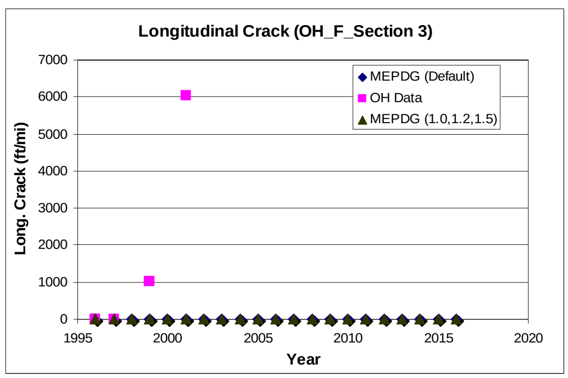

Figure 17. Longitudinal cracking in Section 3 in Ohio

The collected pavement performance data is inadequate for alligator cracking. Because the pavement has deteriorated too quickly, neither of two models could predict alligator cracking well for Ohio. Figure 18 illustrates that alligator cracks increase 0% to 6% in five years, which is an increase 20 times greater than that observed in the Michigan data (Figure 15). Thus, it is difficult to predict alligator cracking using Ohio with default or calibrated values.

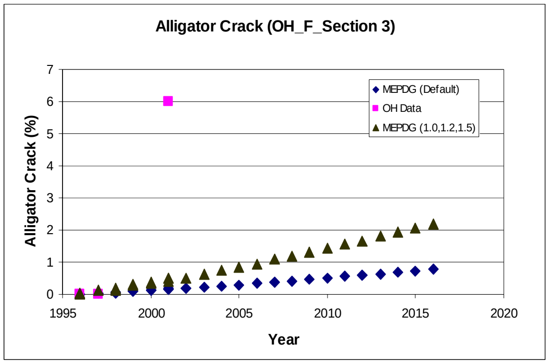

Figure 18. Alligator cracking in Section 3 in Ohio

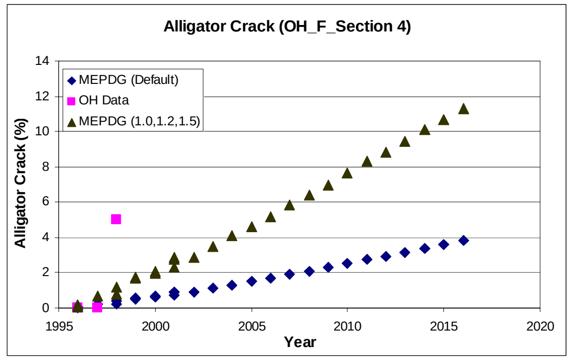

Figure 19. Alligator cracking in Section 4 in Ohio

CONCLUSIONS AND RECOMMENATIONS

This paper has presented the results of an effort to calibrate the MEPDG models. Pavement data from three state transportation agencies in the Midwest region, Michigan, Ohio, and Wisconsin, were collected using a uniform template to serve as input variables in the MEPDG. Data collection was tremendously laborious, causing delays in getting data. Due to time limitations, the data from Michigan and Ohio could not be included in the calibration analysis. A comparison of the pavement distresses predicted using the MEPDG models to the distresses observed in collected pavement field performance data has determined the recommended calibration values for Wisconsin. The distresses predicted with these calibrated factors were compared to field pavement performance data from Michigan and Ohio. Table 4 summarizes the default and recommended calibration factors for the distress models in the MEPDG.

The final goal of calibrating pavement performance prediction is to implement the MEPDG in the regional state transportation agencies. Thus, agency personnel, as well as pavement design consultants, need to be educated and trained in the new pavement design guide. A training program should be established, and MEPDG should be implemented correctly.

Table 4. Calibration factors for prediction models in the MEPDG

Type	Parameter	Formula	Calibration factor	Default value	Recommended calibrated values
Flexible pavement	Fatigue	$N_f = \beta_{f1} k_1 (\varepsilon_t)^{-\beta_{f2} k_2} (E)^{-\beta_{f3} k_3}$	β_{f1}	1.0	1.0
			β_{f2}	1.0	1.2
			β_{f2}	1.0	1.5
	Longitudinal cracking	$F.C. = \left(\frac{1000}{1+e^{C_1 - C_2 * LogD}}\right) * (10.56)$	C_1	7.0	Default
			C_2	3.5	Default
	Alligator cracking	$F.C. = \left(\frac{6000}{1+e^{C_1 - C_2 * LogD}}\right) * \left(\frac{1}{60}\right)$	C_1	1.0	Default
			C_2	1.0	Default

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Routine Highway Maintenance: Relationship between Cost and Condition

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ABSTRACT

When transportation agencies prepare a design for new highway construction or major improvements to existing highways, the life-cycle, agency, and user costs are considered in project design decisions. However, when highway projects are completed, maintenance budgets are rarely adjusted to accommodate the routine maintenance of new lane miles. Maintenance budgets thus often do not keep pace. Even if budgets per lane mile remain relatively constant, the number of vehicle miles traveled per highway mile increases. The growing disparity between maintenance budgets and maintenance requirements leads to difficult choices for maintenance priorities. Concerns about safety and mobility tend to trump preservation of capital investment.

This paper presents a study of the relationship between maintenance cost and performance. Using data from the state of Wisconsin and regression tree analysis, the study identified physical, environmental, operational, and socioeconomic parameters that influence maintenance costs for asphalt and concrete pavements, shoulders, litter pickup, vegetation control, and ditches. Equations were determined for estimating the annual cost for maintenance of state and interstate highways in jurisdictions responsible for the upkeep of 100 to 900 lane miles.

Key words: highway—maintenance condition—maintenance cost—routine maintenance

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PROBLEM STATEMENT

When transportation agencies prepare designs for new highway construction or major improvements to existing highways, the agency and user life-cycle costs are considered in project design decisions. However, when highway projects are completed, maintenance budgets are rarely adjusted to accommodate the routine maintenance of new lane miles.

This problem is worsened by the fact that each year the number of vehicle miles traveled per highway mile increases. Maintenance budgets do not keep pace or remain constant. The growing disparity between maintenance budgets and maintenance requirements causes agencies to make difficult choices about maintenance priorities. In addition to concerns about preserving capital investment, highway operations and maintenance bureaus have concerns about safety and loss of operational efficiency due to deteriorating condition of roadways.

RESEARCH METHODOLOGY

The focus of this research is to estimate the cost of ongoing routine operations and the maintenance cost components of total highway life-cycle cost. There are two main tasks:

1. Perform a correlation study to characterize the relationship between maintenance cost and maintenance condition.
2. Develop model equations for estimating the recurring annual cost for routine, county-level highway maintenance.

This research began with a characterization of cost and condition data. This included gathering and evaluating data from Wisconsin roads and mapping maintenance costs to maintenance conditions. The scope of the project included maintenance of all highway components within an agency's right-of-way, including pavement, shoulders, roadside vegetation, drainage, signs, and pavement markings. Cost and condition data were for fiscal years 2004, 2005, and 2006, and all 72 Wisconsin counties were considered in the study.

The second portion of the research involved the analysis of correlation between condition, costs, and potentially influencing parameters. It was expected that condition will deteriorate as budgets decrease. It was also expected that county-level parameters, such as lane miles, would influence the relationship between expenditures and condition. Sixteen parameters were examined for their influence on cost and condition. For example, soil characteristics, influencing the growth of plants, lead to varying costs for roadside mowing and the control of woody vegetation and noxious weeds. The regression tree approach using the Generalized, Unbiased, Interaction, Detection, and Estimation (GUIDE) modeling algorithm (Loh 2002) considered the 16 parameters as potential categories for refining the prediction models.

Two types of models define the relationships between budgets and maintenance condition: models that predict expected condition given available budget, and models that estimate the budget required to achieve a certain condition. Equations that predict condition given cost can be used to determine the effects of increased or decreased funding. Equations that predict cost given condition can be used to determine the cost to achieve a certain level of condition. The cost models developed in this research will give transportation decision makers the tools required to make informed decisions regarding transportation maintenance budgets.

BACKGROUND

Six percent of Wisconsin's segregated transportation fund ($173.2 million for FY 2006 and $179.4 million in FY 2007) goes toward the maintenance and operation of the current system, including snowplowing, applying salt, inspecting bridges, maintaining rest areas and waysides, replacing signs, installing traffic signals, and repainting highway markings (WisDOT 2007). In Wisconsin, maintenance of state and interstate highways is performed by county crews, and each county services the highways within its jurisdiction. The Wisconsin Department of Transportation (WisDOT) assesses maintenance quality through its Compass maintenance quality assurance program, and maintenance costs are tabulated in its Highway Maintenance System (HMS).

Maintenance Condition

Maintenance condition is cataloged according to Compass elements and their component features. Maintenance elements are defined based on their location or function along a highway. Examples include pavement, shoulders, and traffic management. Elements are made up of features, whose condition is measured with respect to a particular characteristic (Adams and Smith 2006). A maintenance feature is defined as a physical asset or activity whose condition is measured in the field.

Annually, the conditions features are measured through random sampling, with the exception of the traveled way features. All state-maintained highway pavements are inspected on a two-year cycles, with half of the state's pavements inspected in one year, and the other half in the next year. WisDOT then sets grades (A, B, C, etc.) for the individual features based on percent backlogs. If a maintenance feature's condition is to the point at which the feature will become part of the potential maintenance workload within the next 12 months, it is considered to be backlogged. Table 1 lists all Compass elements, their corresponding features, and the thresholds for being backlogged in the year 2005. For continuous features or features measured by the mile, the threshold value includes both sides of the road for a mile segment. For discrete features (i.e., culverts), the threshold value is for the overall inventory of features in the segment being reviewed.

Table 1. WisDOT compass elements, features, and thresholds

Element	Feature	Thresholds for backlogging
Pavement, asphalt	Alligator cracking	10% or more of surface has unsealed alligator cracking (within a mile)
	Block cracking	10% or more of surface has unsealed block cracking (within a mile)
	Edge raveling	Visible cracking is present for 10% or more of the mile
	Flushing	Flushing is present in more than small, isolated areas (within a mile)
	Longitudinal cracking	Any unsealed longitudinal cracking (within a mile)
	Longitudinal distortion	Significant distortion affects 1% or more of roadway (within a mile)
	Patch deterioration	Any patch is deteriorated enough to affect ride quality (within a mile)
	Rutting	Ruts are ¼ inch or deeper (within a mile)
	Surface raveling	The aggregate and/or asphalt binder has worn away and the surface texture is rough or pitted (within a mile)
	Transverse cracking	Any unsealed transverse cracks at least 6' in length (within a mile)
	Transverse distortion	Significant distortion affects 1% or more of roadway (within a mile)
Pavement, concrete	Distressed joints/cracks	Distress in wheel path greater than 2 inches wide (within a mile)
	Longitudinal joint distress	Faulting or signs of distress are present (within a mile)
	Patch deterioration	Any patch is deteriorated enough to affect ride quality (within a mile)
	Slab breakup	Slab is divided into at least 2-3 large blocks, affecting 10% or more of the slab (within a mile)
	Surface distress	Any measurable surface distress is present (within a mile)
	Transverse faulting	Any measurable faulting (within a mile)

Table 1. Continued

Element	Feature	Thresholds for backlogging
Traffic Control and Safety	Centerline/ Edgeline markings	Line with > 20% paint missing (within a mile)
	Delineators	Missing OR not visible at posted speed OR damaged (by delineator)
	Protective Barriers	Not functioning as intended (linear feet of barrier)
	Other signs (emergency repairs)	Missing OR not visible at posted speed (by sign)
	Other signs (routine)	Beyond service life (by sign)
	Raised Pavement Markings	Missing OR damaged (by RPM)
	Regulatory/ warning signs	Missing OR not visible at posted speed (by sign)
	Reg./warn. signs (routine)	Beyond service life (by sign)
	Special Pavement Markings	Missing OR not functioning as intended (by marking)
Shoulders	Cracking	200 linear feet or more of unsealed cracks > ¼ inch (by mile)
	Cross-slope	200 linear feet or more of cross-slope at least 2x planned slope with the maximum cross slope of 8% (by mile)
	Hazardous Debris	Any items large enough to cause a safety hazard (by mile)
	Drop-off/ buildup	200 linear feet or more with drop-off or build-up > 1.5 inches (by mile)
	Erosion	200 linear feet or more with erosion >2 inches deep (by mile)
	Potholes/ raveling	Any potholes OR raveling > 1 square foot by 1 inch deep (by mile)
Drainage	Culvert	Culverts that are >25% obstructed OR where a sharp object-e.g., a shovel-can be pushed through the bottom of the pipe OR pipe is collapsed or separated (by culvert)
	Curb & gutter	Curb & gutter with severe structural distress OR >1 inch structural misalignment OR >1 inch of debris build-up in the curb line (by linear feet of curb & gutter)
	Ditches	Ditch with greater than minimal erosion of ditch line OR obstructions to flow of water requiring action (by linear feet of ditch)
	Flumes	Not functioning as intended OR deteriorated to the point that they are causing erosion (by flume)
	Storm sewer system	Inlets, catch basins, and outlet pipes with >=50% capacity obstructed OR <80% structurally sound OR >1 inch vertical displacement or heaving OR not functioning as intended (by inlet, catch basin & outlet pipes)
	Drains	Under- and edge-drains with outlets, endwalls or end protection closed or crushed OR water flow or end protection is obstructed (by drain)
Roadsides	Barriers	Noise barrier or retaining wall not functioning as intended (by LF of barrier)
	Fences	Fence missing OR not functioning as intended (by LF of fence)
	Litter	Any pieces of litter on shoulders and roadside visible at posted speed, but not causing a safety threat. (by mile)
	Graffiti	Any graffiti and nonnatural encroachments visual at posted speed. (by mile)
	Mowing	Any roadside has mowed grass that is too short, too wide or is mowed in a no-mow zone (by mile)
	Mowing vision	Any instances in which grass is too high or blocks a vision triangle (by mile)
	Noxious weeds	Any visible clumps (by mile)
	Woody vegetation	Any instances in which woody vegetation blocks a vision triangle (by mile)
	Woody vegetation vision	Any instances in which a tree is present in the clear zone OR trees and/or branches overhang the roadway or shoulder creating a clearance problem (by mile)

Maintenance Cost Accounting

Maintenance cost is documented through the HMS (WisDOT 2006a), in which individual maintenance costs are recorded according to their designated activity code. HMS is the data source for all cost data in this study. Some of the activity codes relate directly to Compass features, but the majority do not. This limited the scope of Compass elements and features that could be analyzed.

The method by which costs are aggregated to determine partial element-level costs involves distribution and summation of WisDOT HMS activity code data. Table 2 lists the HMS activity cost codes applicable for each partial element. Each activity cost code is then broken down into labor cost, equipment cost, and material cost. Since the budget amounts cover overhead costs, these are included in the value for each activity. The summation of these costs gives the total cost for the individual HMS activity code.

Table 2. HMS cost activity codes mapped to compass elements/features

Element (cardinality)	Compass Element: Features	HMS Code	HMS Code Description
Asphalt Traveled Way (many-to-many)	*Traveled Way, Asphalt:* alligator cracking, block cracking, edge raveling, flushing, longitudinal cracking, longitudinal distortion, patch deterioration, rutting, surface raveling, transverse cracking, transverse distortion	1	Spot repair/pothole repair
		2	Crack sealing/filling
		3	Seal coating
		4	Wedging/rut filling
		5	Milling/bump removal
		8	Thin resurfacing
Concrete Traveled Way (many-to-many)	*Traveled Way, Concrete:* distressed joints/cracks, longitudinal joint distress, patch deterioration, slab breakup, surface distress, transverse faulting	11	Emergency repair of concrete pavement
		12	Non-emergency repair of concrete pavement
		13	Repair of distressed pavement
Unpaved Shoulders (one-to-many)	*Unpaved Shoulders:* cross-slope, drop-off/build-up, erosion	21	Gravel shoulders
Paved Shoulders (one-to-many)	*Paved Shoulders:* cracking, potholes/raveling	22	Paved shoulders
Mowing (one-to-many)	*Roadside:* mowing, mowing for vision	41	Mowing
Litter Pickup (one-to-many)	*Roadside:* litter; *Shoulders:* hazardous debris	42	Litter pickup
Woody Vegetation (one-to-many)	*Roadside:* woody vegetation, woody vegetation control for vision	43	Woody vegetation
Noxious Weeds (one-to-one)	*Roadside:* noxious weeds	44	Control of unwanted vegetation
Drainage Structures one-to-many)	*Drainage:* culverts, curb/gutter, flumes, storm sewer, under/edge drains	51	Clean/repair drainage structure
Ditches (one-to-one)	*Drainage:* ditches	52	Maintain roadside drainage
Safety Appurtenances (one-to-many)	*Traffic:* delineators, protective barriers *Roadsides:* barriers, fences	55	Maintain safety appurtenances
Permanent Sign Repair (one-to-many	*Traffic:* routine other signs, routine regulatory/warning signs	81	Permanent sign repair
Temporary/Emergency Sign Repair (one-to-many)	*Traffic:* emergency other signs, emergency regulatory/warning signs	85	Temporary/emergency sign repair
Pavement Markings (one-to-many)	*Traffic:* centerline markings, edgeline, raised pavement markers, special pavement markings	3881	Powerplay traffic program code
		3882	Powerplay traffic program code

Three years of HMS cost data were used in the analysis. As recommended by WisDOT (2006b), costs were adjusted by the urban consumer price index (CPI) to base year 2006 costs. The urban CPI allows data from the years 2004 and 2005 to be plotted and statistically analyzed with year 2006 data. The urban CPI for the years 2004–2006 are listed in Table 3, and a formula for adjusting cost is show in Equation 1.

Table 3. Urban CPI

Year	Urban CPI
2004	186.1
2005	191.6
2005	195.9

$$\text{Cost Year i} = \text{Cost Year j}\left(\frac{\text{Urban Consumer Price Index Year i}}{\text{Urban Consumer Price Index Year j}}\right) \tag{1}$$

For each activity and county, labor costs were adjusted for location, and then total costs were adjusted by the consumer price index and normalized by million vehicle miles traveled per year in the county. Equation 2 shows the formula for adjusting costs to account for time and location factors so that data for multiple counties and years can be compared. Thus, up to 216 data points (72 counties multiplied by 3 years) are available for analysis of each activity.

$$\frac{\text{Cost}}{\text{mVMT}} = \frac{\dfrac{\text{FY 2006 Dollars}}{\text{Year}}}{\left(\dfrac{\text{Million Vehicle Miles Traveled}}{\text{Year}}\right)} \tag{2}$$

Regression Tree Analysis

A regression tree is a piecewise constant or piecewise linear estimate of a regression function, constructed by recursively partitioning the data and sample space. The regression tree approach provides a way to find county groups and the corresponding model for each group. Within each tree, models capture differences among the counties depending upon county size (lane miles), vehicle miles traveled, population density, etc. The result is sets of models that may be applicable to counties in other states with similar characteristics (Adams et al. 2006). The regression tree modeling algorithm, GUIDE, has unbiased variable selection at the splits and local sensitivity to pair-wise interactions among predictor variables. The algorithm automatically searches through numerous multiple linear models to fit different subsets of the data and uses cross-validation to pick groupings and corresponding models that minimize the least squares error for the prediction. The result is models with improved prediction accuracy and the effect of the outliers alleviated. Additionally, the models are much simpler and easier to use than transformed (nonlinear) models based on the scatter plots (Adams et al. 2006).

RELATING COST TO CONDITION

Cost is mapped to condition at the lowest level possible because it is logically difficult to aggregate the condition of several features to represent the condition of a single element. The result of breaking down the elements as far as possible and mapping costs to conditions is shown in Table 2, which illustrates the relationship between HMS cost codes and Compass elements/features as per the HMS cost code descriptions in the Wisconsin State Highway Maintenance Manual (WisDOT 2006a). The relationships can be characterized as either one-to-one, many-to-one, or many-to-many. Examples of each and associated assumptions are described as follows. The ideal situation is a one-to-one relationship because there is a direct correlation between one HMS cost code and one Compass feature. A one-to-one relationship leaves little room for ambiguity, so confidence is high that expenditures assigned to the HMS cost code are spent to maintain that feature. The two examples of one-to-one relationships in Table 2 are the partial elements noxious weeds and ditches.

One-to-many relationships relate one cost to maintenance of many features. For these, one HMS cost code is mapped to several Compass features, and condition measures must be aggregated. Equal weight is assumed among the measures. This assumption may be satisfactory if distress in one feature of an element does not influence distress in the others features and there are an equal number of observations for each feature. One-to-one relationships avoid the need to aggregate feature backlogs and therefore do not require these assumptions. Examples of one-to- many relationships include the elements unpaved shoulders, drainage structures, and safety appurtenances.

Finally, analysis of many-to-many relationships requires more assumptions. Many-to-many relationships occur when more than one HMS cost code is mapped to more than one Compass feature. In addition to the assumptions for one-to-many relationships, analysis of many-to-many relationships require another assumption: equal distribution of costs among the features. Two examples of many-to-many relationships are the asphalt and concrete traveled ways. For these elements, the data sets include observations of all highway miles. Therefore, this analysis should give particularly accurate results.

ANALYSIS OF COST AND CONDITION

The objective of this analysis is to find relationships between cost and condition so that we may estimate the expenditure necessary for obtaining a specific backlog or to predict the backlog that may be expected given a specific funding allocation. The scatter plots of adjusted cost/million vehicle miles traveled (mVMT) versus backlog showed no trends.

Numerous physical, environmental, operational, and socioeconomic factors may influence the costs, expenditures, and outcomes of highway maintenance activities. A regression tree analysis is a tool for identifying the influencing parameters, to find groups of similarly behaving counties, and to develop models for those behaviors. The regression tree modeling algorithm, GUIDE, was used to model the relationships.

Table 4 lists the parameters of the regression tree analysis. The parameters may be used to fit the data to model equations and/or to split the data points into groups with similar trends. Each of the predictor variables in Table 4 are designated *c*, *d*, *f*, *n*, or *s*. These designations are interpreted as follows: *c* indicates a categorical variable used only for splitting counties into groups, *d* indicates a dependent variable, *f* indicates a numerical variable used only for fitting the equations, *n* indicates a numerical variable used for both splitting and fitting, and *s* indicates a numerical variable used only for splitting counties into groups.

Table 4. Parameters for regression tree analysis

Parameter	Description and Units (Parameter values are for each county)	GUIDE Type
Cost	Cost/mVMT in 2006 FY dollars	*d, f, n*
Backlog	Element or feature backlog – percentage	*d, f, n*
mVMT	million Vehicle Miles Traveled per year	*n*
LM	Lane Miles	*n*
CLM	Center Line Miles	*n*
CLM_{paved}	Center Line Miles of unpaved shoulders	*n*
$CLM_{unpaved}$	Center Line Miles of paved shoulders	*n*
$LM_{asphalt}$	Lane Miles of asphalt traveled way	*n*
$LM_{concrete}$	Lane Miles of concrete traveled way	*n*
$Age_{asphalt}$	Pavement Age – Average number of years since last resurface of reconstruction	*n*
$Age_{concrete}$	Pavement Age – Average number of years since last resurface of reconstruction	*n*
WSI	Winter Severity Index – Measure of winter severity	*s*
LGroup	Latitude Group – Numbered 1 to 5: North to South	*s*
CountyCode	Counties numbered from 1 to 72	*c*
LandArea	Land Area - Square Miles	*s*
PopDensity	Population Density - Population per square mile	*s*
Income	Median Household Income per county - 2000 FY dollars	*s*
$Soil_{type}$	Soil type – Numbered 1 to 5: Silty, Sandy, Loamy, Wetland, Sandstone	*c*
$Soil_{pH}$	Soil pH – Ranges from 5.0 – 7.3, average = 6.6	*n*
$Soil_{K}$	Soil Potassium Content - parts per million, ranges from 80 – 166, average = 134	*n*
$Soil_{P}$	Soil Phosphorus Content - parts per million, ranges from 30 – 153, average = 52	*n*
$Soil_{OM}$	Soil Organic Matter – percentage, ranges from 1.2 – 7.0, average = 3.0	*n*
truckVMT	County Truck Vehicle Miles Traveled per Year	*n*

Various parameters were selected for each model. The variable mVMT is based on traffic counts taken for one-third of the counties each year, and a growth factor is applied to keep the estimates current (WisDOT 2006c). The exception is Milwaukee County, in which one-third of the counties are counted every year. The variable mVMT was used as a possible fitter and/or splitter variable for every partial element. Lane miles (LM) were included in the model for litter pickup because we hypothesize that roadways with more lanes will have more litter than roadways with fewer lanes. Similarly, centerline miles (CLM) were included as potential parameters for the mowing, litter pickup, woody vegetation, noxious weeds, and ditches models. These costs are related to the distance to be maintained. Related parameters are centerline miles of paved and unpaved shoulders (CLM_{paved} and $CLM_{unpaved}$); these were used in the models for paved and unpaved shoulders, respectively. Lane miles of asphalt and concrete pavements ($LM_{asphalt}$ and $LM_{concrete}$) were included as parameters in the models for asphalt traveled way and concrete traveled way, respectively.

These models also included parameters for average age of the asphalt and concrete traveled way pavements in each county ($Age_{asphalt}$ and $Age_{concrete}$). The age of each mile of pavement is measured from new construction or most recent resurface or reconstruction. LM and CLM for each of the counties were provided by WisDOT from its state trunk network (STN) database. It was hypothesized that older pavements will need more maintenance than new ones. Winter severity index (WSI) is a measure of the winter snow and freezing rain events. WSI was used as a splitter variable because we hypothesize that winter weather may impact condition and therefore cost of maintenance on the traveled ways, shoulders, and ditches. To additionally capture weather effects, each county was assigned to a latitude group (LGroup), as shown in Figure 1. The latitude groups are numbered 1 to 5, with 1 being the northernmost counties and 5 being the southernmost counties. Population density and land area were obtained from the U.S. Census Bureau (2000). Median household income was obtained from the Wisconsin Department of

Administration (2007). These were included as splitter parameters to capture possible socioeconomic effects on maintenance priorities in each county. Parameters for soil characteristics include soil type ($Soil_{type}$) (Hole 1968), pH ($Soil_{pH}$), potassium content ($Soil_K$), phosphorus content ($Soil_P$), and organic matter content ($Soil_{OM}$) (UW Soil Lab 2007). The prevailing soil types are 1-Silty, 2-Sandy, 3-Loamy, 4-Wetland, and 5-Sandstone. Soil characteristics were added as parameters for the mowing, woody vegetation, noxious weeds, and ditches models. Finally, truck vehicle miles traveled (truckVMT) were obtained from WisDOT's 2000 statewide travel demand model. The truckVMT values are based on commodity freight estimates. We hypothesize that truckVMT may be an important parameter for splitting and fitting the cost models for asphalt traveled way, concrete traveled way, paved shoulders, and unpaved shoulders.

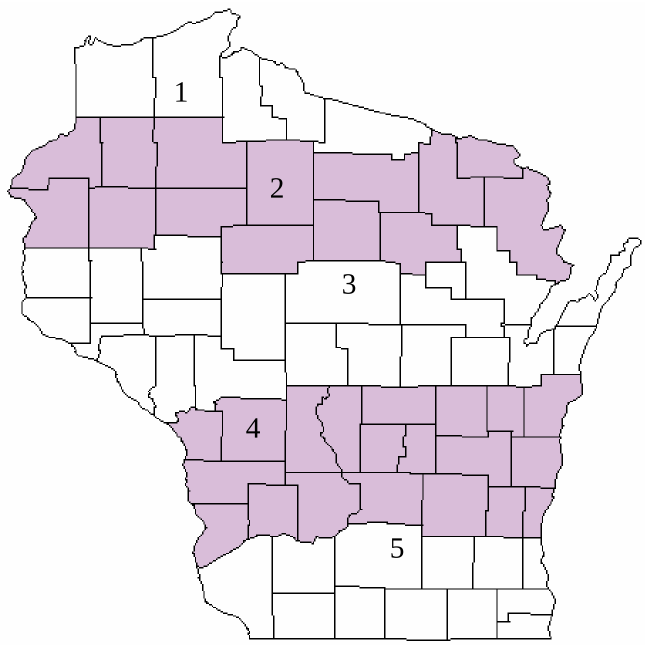

Figure 1. Latitude groups

COST MODELS

We used the regression tree analysis tool to search for cost models in terms of backlog and other potentially influencing parameters. We also used the tool to search for models to predict backlog in terms of cost and the other potentially influencing parameters. For each analysis, we allowed the tool to use cost and backlog as independent variables to fit the model or to both fit and split the models. Thus, we analyzed four combinations of parameter designations for each relationship between maintenance cost and condition.

Figure 2 shows an example output of the analysis for estimating the annual cost of mowing. The figure shows nine models that depend upon the counties' latitudes, the soil type, the soil phosphorus and potassium contents, the total CLM, and land area. Logically, these parameters should influence the growth of plants and the area of mowing. The regression tree shows the conditions under which each of the prediction models should be used. For example, if the county is in latitude groups 1 or 2, then we can predict the annual cost of mowing using the formula associated with node 4. The average expected cost is $37,300 in year 2006 dollars. If the latitude group is 4 or 5, the soil potassium content is greater than 54.5ppm, and the county has more than 221.26 centerline miles of highway to mow, then we predict the annual cost of mowing using the formula associated with node 15. The average expected cost is $164,000 in year 2006 dollars.

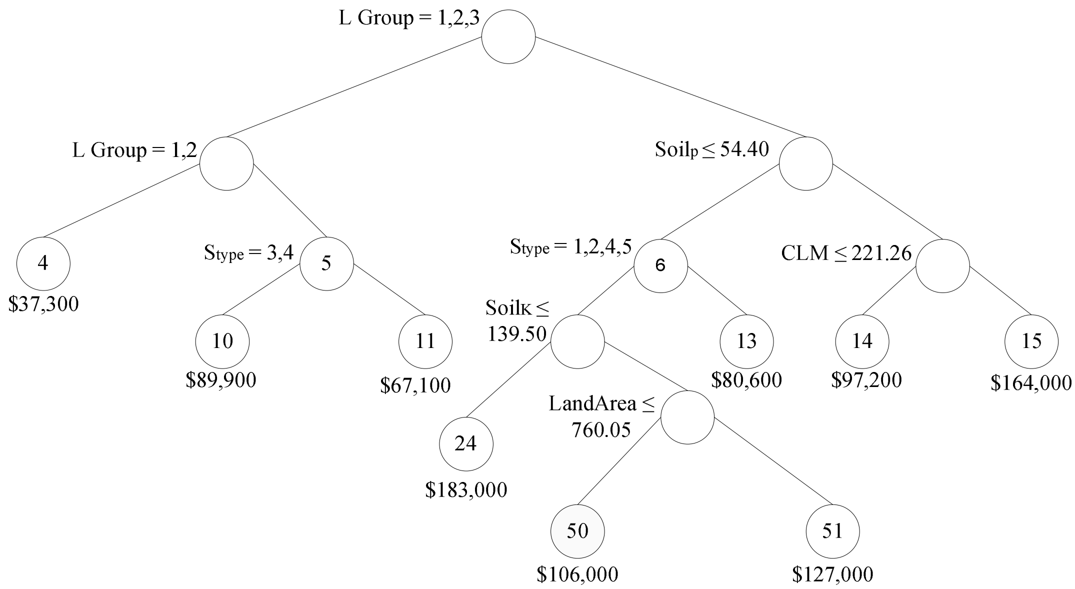

Figure 2. Regression tree model for estimating annual cost for mowing highway roadsides

Information pertaining to the measure of fit includes the R^2 for each individual node and the R^2 for the tree model as a whole. Table 5 lists the model equations and R^2 for each terminal node shown in Figure 2. For the tree in Figure 2, the R^2 is 0.953, while the R^2 for some of the individual models is much lower. Another measure of fitness is the p-value for each parameter in the models. A p-value of zero indicates confidence that the correct predictor parameters are being used. Although the R^2 value for node 4 is low, indicating a poor fit, the p-value for mVMT is also low, indicating that vehicle miles traveled influences mowing costs. This seems reasonable, since highways with the more use are more likely to be mowed.

Table 5. Model equations for estimating cost of mowing maintenance activity

Node	Model for estimating annual cost	R^2
4	Cost = 26831 + 34.7(mVMT)	0.125
10	Cost = 909200 + 63.4(mVMT) – 918800($Soil_{pH}$) – 2542($Soil_K$) + 2153.2($Soil_p$) – 217.82(CLM)	0.773
11	Cost = 319800 + 55.3m(VMT) – 44407($Soil_{pH}$)	0.930
13	Cost = -71079 + 212(Backlog) + 1001.2(CLM)	0.986
14	Cost = 12323 + 132.4 (mVMT)	0.801
15	Cost = -662480 + 3447.1($Soil_K$) + 1206.7(CLM)	0.959
24	Cost = 65804 + 55.6(mVMT) – 1381.1($Soil_p$) + 501.5(CLM)	0.977
50	Cost = -204090 – 47.6(mVMT) – 1665 ($Soil_K$)	0.814
51	Cost = 2831500 – 47.6 (mVMT) – 393790 ($Soil_{pH}$)	0.935

Table 6 lists a few of the recommended models for estimating cost and predicting backlog. These models show good fit with relatively high R^2 and low p-values. In three models, the coefficients for backlog and cost on either side of the equation have the same sign. This does not agree with the expectations that backlog increases as cost decreases and vice versa. In one model, moreover, cost is independent of backlog. These results occur because budget allocations are based on inventory, not backlog. These models are perhaps reflective of the agency's allocation models. However, the study shows that, for some elements, condition is related to cost, while for other elements current expenditures are sufficient to maintain steady state condition.

Table 2. Cost models for routine highway maintenance

Element	Valid conditions	Equation	R^2	Sample size
Concrete Traveled Way	$Age_{concrete}$ 15.95 yrs	Backlog = -0.018 – 0.000014(Cost) + 1.89($Age_{concrete}$) + 0.0043($LM_{concrete}$)	83%	81
Asphalt Traveled Way	All	Backlog = -0.452 + 0.0000011(Cost) + 0.39($Age_{asphalt}$)	87%	168
		Backlog = -0.484 + 0.00014($CLM_{Asphalt}$) + 0.107($Age_{asphalt}$) + 0.000000004(truck VMT)	87%	168
Litter Pickup	All	Cost = 56516 + 19.72(Backlog) + 274(mVMT) – 282(LM) – 234(CLM)	86%	206
Woody Vegetation	LGroup = 4 or 5	Cost = 787891 – 2805.8(Backlog) + 13.9(mVMT) – 83790($Soil_{pH}$) – 894($Soil_K$) – 1798($Soil_P$) + 339(CLM)	65%	92

CONCLUSIONS

This study has attempted to make the best use of the data available from WisDOT in order to provide meaningful equations that relate maintenance cost to maintenance condition. Maintenance cost and condition are difficult to study because data is often in separate systems that support different business processes, and the data sets are therefore not easy to relate physically. However, a more challenging obstacle is that the parameters are often incompatible, making it nearly impossible to relate cost to condition logically.

Physical, environmental, operational, and socioeconomic parameters were introduced to find models that explain the relationships between cost and condition. The following conclusions were drawn regarding the models:

- The most influential parameter for fitting and splitting in the asphalt and concrete traveled way cost models is pavement age. Backlog and maintenance expenditures increase as pavement ages.
- Parameters that are most influential for fitting in the mowing, litter pickup, and woody vegetation models are million vehicle miles traveled and centerline miles. Counties with more centerline miles and heavily used roadways spend more money on these activities. Interestingly, another influential parameter for mowing and woody vegetation is latitude group. Counties in the southern latitudes have higher maintenance costs for these activities than counties in northern latitudes. A hidden effect behind this parameter may be the fact that larger cities in the southern half of the state have higher labor costs.
- Median household income influences cost in two models: concrete traveled way and mowing. In both cases, median household income is a splitter variable, and maintenance costs are higher in counties with high median household income. This is an interesting result because activities costs were adjusted to account for varying labor rates in different locations. This finding may indicate that counties with higher median household incomes may have higher expectations for maintenance condition.

Relationships with many-to-many and one-to-many cardinality between activity costs and maintenance condition produced better fitting models than the one-to-one relationships. This was contrary to what was expected. A possible explanation for this finding may be that, for the particular one-to-one relationships chosen, the costs do not accurately represent the condition features to which they were mapped.

The modeling effort for asphalt and concrete traveled ways produced R^2 values greater than 80% and was more successful than for other elements. This is consistent with our expectation, since the condition data for these elements are complete and not sampled, as it is for other elements. For some elements, models with high R^2 value are customized for a few counties in Wisconsin. Their applicability to other locations has not been tested.

This research produced many other cost models that had good fit but not related condition. This suggests that budget allocations are based on previous allocations, not on backlog condition. Counties receive a fixed amount based on vehicle miles traveled and centerline miles, and these allocations are adjusted according to available funds rather than current backlog or maintenance conditions.

These models will help maintenance programmers justify budgets and show accountability. The models apply to other states where maintenance backlog is calculated and categorized in a similar manner to that of Wisconsin. A follow up study is currently underway to investigate the relationship between maintenance cost and condition in other states in the region, namely Michigan and Ohio. For these studies, alternative methods to aggregate and relate cost and condition must be developed.

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Investigation of the Impacts of Rural Development on Iowa's Secondary Road Systems

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ABSTRACT

Rural areas across the state of Iowa and the entire nation are facing a number of problems associated with increasing development. The impact of this development on rural road systems is one significant problem. While research has been directed at other impacts, such as loss of quality agricultural land, fragmentation of natural habitat, water quality, land use compatibility, and provisions for other infrastructure and government services, much of this work does not sufficiently address the physical impacts on local roadways. Furthermore, very little research is designed to provide local decision makers with tools for making day-to-day decisions on development proposals. Many counties in Iowa are increasingly faced with proposed rural developments, such as rural residential subdivisions and livestock production operations, that generate substantial new traffic on secondary road facilities. In fact, the creation of rural residential subdivisions is a much more significant producer of land use change in Iowa than is urbanization in the form of municipal annexation (Iowa State University Extension to Communities 2001).

In order to better understand the impact of rural development on the secondary road system, a geographic information systems analysis was used to quantify the spatial relationship between these developments and various physical features and illustrate the nature of the rural development impacts. Previous work conducted by the Center for Transportation Research and Education on land use change in Iowa indicates that rural residential subdivisions that provide primary residences appear to be locating in areas with excellent access to major transportation arteries within a half-hour commute of Iowa's metropolitan centers or other trade centers. They also tend to be locating near amenities such as surface water and forested land and not on prime farm land. This means that such subdivisions tend to be concentrated in

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areas that fit a specific spatial profile. On the other hand, livestock operation locations are regulated by the Department of Natural Resources' Master Matrix, and they tend to develop in rather isolated areas so that environmental and social impacts can be minimized. They appear much more randomly distributed across the map of Iowa.

This spatial analysis provides a better understanding of where and how rural development happens, ultimately providing local decision makers with better tools to quantify potential traffic generation, analyze build-out scenarios, estimate true costs of community services, and further understand the fiscal impacts and associated legal issues of such development.

Key words: economic analysis—geographic information systems—rural development—secondary roads

Evaluation of Multiple Blade Snowplow Designs

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ABSTRACT

The design of plows used to remove snow and ice from roadways has changed very little in the last 50 years. A typical snowplow is comprised of a very heavy metal frame that usually relies on gravity to keep the plow in constant contact with the roadway surface. Attached to the plow are steel or carbide blades that help cut or peel the snow or ice from the road. These rigid blades do not conform to the contours of the road, which results in snow and ice being missed by the carbide blades and left in the traveled portion of the roadway. Subsequently, any deicing materials spread at the rear of the snowplow truck will have to melt the remaining snow or ice missed by the plow plus any additional snow or ice falling from the sky. The remaining snow or ice can continue to affect traffic and can contribute to the dilution of deicing chemicals.

The Iowa Department of Transportation has developed, built, and tested three new plow designs that incorporate multiple cutting edges that can adjust to the contours of the roadway and that are designed to more effectively clear the road of snow and ice in one pass. In each multiple-blade test plow design, a traditional carbide blade is positioned on the front portion of the plow so it can first remove the bulk of the snow and ice from the roadway. Immediately behind the traditional carbide blade, a scarifying blade is used to scrape and loosen hard-packed snow or ice left by the traditional blade. Behind the scarifying blade, a rubber blade is used to squeegee the roadway surface of any remaining loose snow, slush, and liquid. The rubber blade can follow the shape of the road surface and can effectively clear areas missed by traditional rigid blades. Although the three test plow designs had different outward appearances and operation controls, each combination blade approach allowed deicing materials spread from the truck to be placed on a roadway with a smaller amount of snow and ice, which resulted in less material usage and a quicker return to normal driving conditions.

Key words: equipment—plow design—snow removal—winter maintenance

Development of a New Methodology Based on Conductivity Measurements to Detect the Presence of Deleterious Fine Particles in Concrete Aggregate

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ABSTRACT

Under the current Wisconsin Department of Transportation specifications, the presence of fine particles (passing the #200 sieve) in coarse and fine aggregates is limited to 1.5% and 3.5% of mass, respectively. The recent experience of technical personnel involved in field pavement operations and the latest investigations of the scientific community indicate that within some reasonable limit this is not an issue of the quantity of fine material but rather its mineralogical nature. Of special significance is the presence of certain types of clays that can alter water distribution in concrete and therefore induce changes in the aggregate-paste interface and in the hydration process. Based upon this observation, it seems that the p200 method cannot be used to distinguish between harmful or innocuous particles. At present, the California Cleanness Test (CCT) and the Methylene Blue Test (MBT) are considered to be more informative than the p200 in this regard.

However, these methods also have some significant disadvantages. The CCT cannot differentiate between large clay particles with macroscopic swelling and small non-clay particles with long sedimentation times. It should be noted that the cation exchange capacity (CEC) is a distinctive characteristic of clays, and it has been related to their potential harmfulness. While the MBT measures the CEC of the fines, the results of this test depend on operator objectivity. We have developed a new method to measure the CEC of fines, and this method also provides information on the nature of the exchangeable cations. This test is more precise than the MBT, and it does not require the separation of the fines from the aggregates. This is an important advantage, but more importantly it allows this method to be applicable to the characterization of fine aggregates.

This method is based on changes in ionic conductivity in solutions of different salts, induced upon adding a quantity of aggregates to a solution. The cations of these solutions exchange with the cations of the fines, and the magnitude of the ionic conductivity is determined by both the quantity and nature of the exchangeable cations. Equivalent conductivity, the concentration of the salts mixed with the aggregate,

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and the experimentally measured ionic conductivity of the liquid phase of the mixtures are all parameters contained in a set of equations in which the unknowns are the quantity and nature of exchangeable cations per unit mass of aggregate.

Key words: aggregate—cation exchange capacity (CEC)—clays—concrete—conductivity—deleterious—microfines

Determining Practical Standing-Corn Snow Fence Configuration through the Evaluation of Drift Characteristics

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ABSTRACT

Snow fences are an effective means for preventing snow buildup on road surfaces and reduced visibility caused by blowing snow. The Iowa Department of Transportation establishes contracts with Iowa farmers to leave rows of unharvested corn through the winter to act as a natural snow fence. Accepted guidelines regarding traditional man-made snow fence placement with respect to the road may not be appropriate for standing-corn snow fences because the drift configuration resulting from standing-corn snow fences are different from those resulting from other more common snow fence types. It was necessary to determine the appropriate distance from the standing-corn snow fence to the edge of the roadway because it is usually best for the farmer to leave the corn near the edge of the field next to the highway right-of-way, but placement too close to the road can result in a drift forming on the road itself.

In the winter of 2006–2007, the Iowa Department of Transportation conducted an evaluation of the drift characteristics of 12 rows of standing corn to determine the minimum distance between the standing corn and the road. Measurement stakes were driven into the ground at regular intervals upwind of the fence, through the corn rows, and extending to the right-of-way line to allow the researchers to track the depth and evolution of the drift through the entire winter. The results of this evaluation determined that a standing-corn snow fence placed near the highway right-of-way is far enough from the road to protect the area from blowing and drifting snow if sufficiently small amounts of snow are experienced during the winter. If more severe blowing snow conditions are present, the drift formed by the standing-corn snow fence may encroach on the road if the corn rows are placed near the right-of-way. Standing-corn snow fence placement guidelines will be presented.

Key words: snow fence—winter maintenance

Strategic Highway Research Program 2 (Invited Presentation)

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ABSTRACT

The second Strategic Highway Research Program (SHRP 2) is conducting applied research in four focus areas that were identified through a Congressionally commissioned planning study that began 1999. The focus areas were selected on the basis of their importance to the nation's economic system and quality of life and because strategically targeted research in these areas promises to yield high payoffs. The objectives of each focus area are as follows:

- Safety: Prevent or reduce the severity of highway crashes by understanding driver behavior
- Renewal: Address the aging infrastructure through rapid design and construction methods that cause minimal disruption and produce long-lived facilities
- Reliability: Reduce congestion through incident reduction, management, response, and mitigation
- Capacity: Integrate mobility, economic, environmental, and community needs in the planning and designing of new transportation capacity

Funding for the SHRP 2 program was provided in August 2005 by SAFETEA-LU legislation. The program is now in its second year. Twenty-two contracts valued at $26 million are underway in the four focus areas, and about $80 million in additional requests for proposals will be released through 2009. This presentation describes the strategic objectives of each focus area, the projects underway, future proposal opportunities, and the products expected.

Key words: safety—Strategic Highway Research Program

Braking Behavior at Rural Expressway Intersections for Younger, Middle-Aged, and Older Drivers

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ABSTRACT

High speed expressway intersections can be problematic for drivers of all ages. The purpose of this study is to evaluate driver performance at a high crash rate rural expressway intersection for three age groups: older (65 to 80), younger (18 to 25), and middle-aged drivers (35 to 55). This study reports preliminary findings associated with 30 drivers (ten drivers in each age group) who participated in an instrumented vehicle study. Of particular interest was to understand how braking behavior may differ on the approach to an expressway intersection prior to executing one of three different maneuvers (e.g., going across the intersection, left turn, and right turn). The specific performance measures include mean deceleration, initial brake point, brake pedal differential time (difference in initial to maximum brake pedal depression), and whether or not the driver came to a complete stop. Overall, individual differences based on age were observed. Both younger and older drivers showed more dramatic changes in the braking profile when compared to middle-aged drivers. Middle-aged drivers had the highest number of complete stops prior to entering an intersection and less dramatic or longer brake pedal differential times when compared to the other two age groups. Younger drivers were the least likely to come to a complete stop. This can be a potential safety concern given the lack of time allocated to make appropriate decisions. Future analyses will compare these findings to a low crash rate intersection as well as investigate how physiological measures such as heart rate variability may differ for these same age groups.

Key words: age differences—expressway intersections— human factors—instrumented vehicle

INTRODUCTION

Studies show that age and roadway characteristics are significantly related to an increase crash likelihood (Chin and Quddus 2003; Kim et al. 2007). According to a recent U.S. study, intersection crashes account for over one-third of vehicle crashes and lead to approximately 9,000 fatalities every year (Alexander et al. 2006). Crashes at intersections without traffic controls were 1.7 times more likely to result in fatalities compared to crashes at intersections with traffic controls (Zhang et al. 2000). Rural expressway intersections are particularly problematic. These intersections consist of a high-speed, multi-lane, divided highway (major roadway) and a low-speed two-lane roadway (minor roadway) in a rural area (Hochstein et al. 2007). The typical rural expressway intersection is two-way stop-controlled (TWSC).

Older drivers (defined as 65 and older) have been shown to have a higher likelihood of crashes at intersections compared to other driver age groups (Keskinen, Ota, and Katila 1998; Guerriera, Manivannanb, and Nair 1999). The relative crash risk for older drivers is also much higher at uncontrolled or stop-controlled intersections when compared to signalized intersections (Preusser et al. 1998; Bayam, Liebowitz, and Agresti 2005) and if the intersection is located within a rural area (Garber and Srinivasan 1991). However, other studies also indicate that the crash rates at two-way stop-controlled intersections are also high for younger drivers (Retting, Weinstein, and Solomon 2003). Younger drivers tend to accelerate and decelerate at much quicker rates and, therefore, increase their potential for a crash (McGwin and Brown 1999; Neyens and Boyle 2007). Studies have shown that factors that influence safety at rural expressway intersections relate to roadway volumes (Maze, Hawkins, and Burchett 2004) and intersection designs (e.g., vertical/horizontal curves, skewed intersections) (Burchett and Maze 2006). However, few studies have fully examined the individual differences associated with the driver within these expressway intersections.

The objective of this study is to examine driver performances at a high-crash-rate rural expressway intersection as influenced by age-related differences. This objective is accomplished using data collected from an instrumented vehicle in an on-road study conducted in the state of Iowa. This paper focuses on one of the two intersections that were evaluated as part of the original study. More specifically, our hypothesis centers on differences that may exist in braking behavior among different driver age groups at the high crash rate intersection. The outcomes of this study will provide relevant design guidelines for future systems.

METHODS

Participants

A total of 30 drivers with equal proportions in three age groups were recruited through advertisements in the local newspaper. Younger driver were from 18 to 25 years old (M=22, SD=2.1, n=10); middle-aged drivers were from 35 to 55 years old (M=46, SD=3.5, n=10); and older drivers were from 65 to 80 years old (M=73, SD=5.3, n=10). All participants were active drivers with a valid U.S. driver’s license, and had been screened to ensure safe driving records. They were compensated (U.S.) $20 per hour for participation, and the study lasted approximately one hour.

Apparatus

A 2002 Ford Taurus instrumented sedan was used for this study. Two LP-850W weatherproof cameras and four MB-750 pinhole lens cameras were installed in the vehicle to capture foot movements, visual scanning behavior, steering position, and lane deviations. The four pinhole cameras were located inside the car body, the two weatherproof cameras were located under the left and right mirrors, and all cameras were completely unobtrusive to the drivers. The video was captured with a sample frequency of 15Hz. A Garmin GPS-17N GPS receiver was installed on the back of the trunk so that the driver's position was known at all times. For the analyses, the driver's foot movement behavior is further examined.

Driving performance measures that were recorded include driving speed, braking force, throttle position, and GPS location. All data were automatically recorded by National Instrument Labview software and saved onto a computer that was located in the trunk of the instrumented vehicle and later transferred to a personal computer for analysis.

Procedure

Prior to starting, all participants were provided with a brief explanation of the main purpose of the study and an IRB consent form to sign. During the study, all participants were asked to perform three intersection maneuvers at a two-way stop-controlled (TWSC) intersection: drive across the intersection, turn left onto the expressway, and turn right onto the expressway. For this paper, the intersection examined was identified as having a high crash rate, as defined by the Iowa DOT, with approximately five crashes per year over the most recent four years of data (2002 to 2006). The TWSC intersection is composed of an expressway (major road) and rural (or minor) road. The expressway was a divided highway with two lanes of traffic on each side and speed limit of 65 mph (approximately 105km/h). The rural (or minor) road at the high-crash-rate intersection was a two-lane road with a speed limit of 35 mph (approximately 56 km/h). All participants were told to drive as they normally would, follow the instructions provided by the researcher who sat in the front passenger seat, and adhere to the posted speed limits whenever possible. Prior to beginning data collection on the study, all drivers had an opportunity to drive the car in order to become familiar with the controls. Immediately after the drive, all participants were asked to fill out three surveys that assessed their mental workload and perceived stress as it related to the completed drive. All experiments were conducted on dry roads and under normal weather conditions (i.e., no rain, sleet, or snow).

Independent and Dependent Variables

The focus of this study is to investigate the age differences in braking behavior as drivers slowed down for a stop sign at the high crash rate intersection. The independent variables for this analysis include age groups (younger, middle-aged, older) and gender (male, female). There were three dependent measures that relate to braking behavior and were used to describe how drivers respond prior to entering the intersection. To minimize learning effects, the braking behavior related to each continuous outcome was averaged across the three intersection maneuvers (left turn, right turn, and straight across). A binomial variable was also included in this analysis to investigate whether or not drivers came to a complete stop prior to entering the intersection.

Brake pedal differential time (in seconds). Measures the time from initial to maximum depression of the brake pedal (equation 1). Lower values represent a more sudden brake, and higher values indicate a more gradual braking profile.

$$\text{Brake pedal differential time} = \frac{\sum_{m=1}^{3}\left(t_{f,mn} - t_{o,mn}\right)}{3} \tag{1}$$

where

$t_{f,mn}$ = time at maximum brake pedal depression for driver n at manuever m

$t_{o,mn}$ = time at initial brake pedal depression for driver n at manuever m

Mean deceleration (in m/s^2): Describes the average deceleration from initial brake depression until the vehicle reaches its lowest speed during its approach to the stop sign prior to entering the TWSC intersection.

Initial brake point (in meters): Describes the distance or point at which the driver initially responds (by braking) to the stop sign prior to entering the intersection. This value is measured as the distance from the stop sign.

Complete stops (binary): Specifies whether or not a driver made a full stop (i.e., velocity>0) at the stop sign before execution of each maneuver. It was coded as a binary variable with '1' =complete stop and '0' =incomplete stop. There was a separate binary code for each drive maneuver. According to U.S. traffic regulations, drivers are required to make a full stop (i.e., velocity=0) at a stop sign. It was therefore of interest to observe whether drivers would comply and whether differences would exist between age groups.

RESULTS

The statistical software package, SAS 9.1, was used for the data analysis. A logistic regression model using the Proc GENMOD procedure was developed to predict the likelihood of an incomplete stop. The analyses on the continuous dependent variables were performed with the Proc ANOVA procedure. There were no significant differences in mean deceleration ($p>0.05$). All other dependent measures are discussed in this section.

Age was found to have a significant impact on the brake pedal differential time ($F(2, 24)=8.6$, $p=0.0015$). As observed in Figure 1, both older and middle-aged drivers took significantly more time to go from initial to maximum brake pedal depression when compared to younger drivers (Pairwise comparisons: $t(18)=2.86$, $p=0.01$, $\Delta= 2.81$ sec, CI: 0.75,4.88 ;$t(18)=3.63$, $p=0.002$, $\Delta= 3.52$ sec, CI: 1.48,5.55). The results suggest that younger drivers moved more quickly from initial to maximum brake pedal depression (mean=7.24 sec) than middle-aged (mean=10.8 sec) and older drivers (mean=10.1 sec).

There were significant age differences observed at the initial brake point ($F(2, 24)=10.45$, $p=0.0005$). Generally, middle-aged drivers braked on the approach to an intersection significantly earlier than younger drivers ($t(18)=3.53$, $p=0.002$, $\Delta= 29.1$ sec, CI: 9.7388,38.42) or older drivers ($t(18)=3.7$, $p=0.002$, $\Delta= 22.2$ sec, CI: 9.57,34.75). No differences in the initial brake points were observed between older and younger drivers.

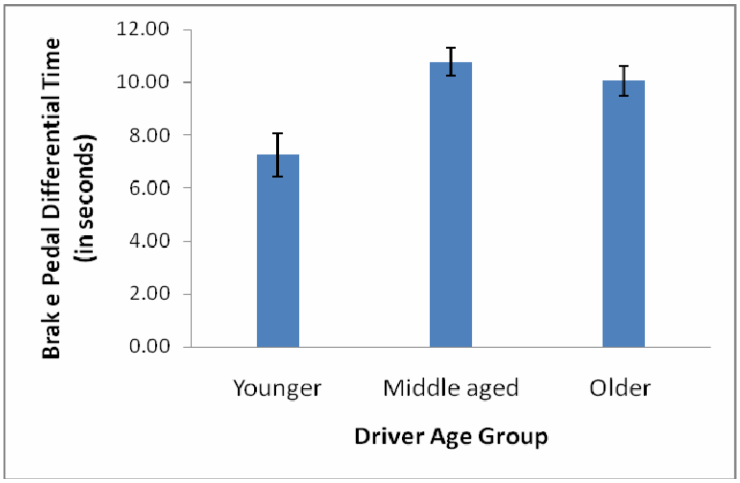

Figure 1. Mean difference in time between initial and maximum brake pedal depression (with standard error bars) for the three age groups

Figure 2 shows the average brake load sensor voltage readings from the instrumented vehicle over each consecutive five-meter interval for the three age groups. The brake load sensors are used to measure how much force is being applied to the brake pedal and the voltage reading from the sensor is an indicator of the pressure placed on the brake pedal with higher voltage values indicating higher brake pressure. For this vehicle, normal braking usually falls in the 2.8 to 3.3V range. As observed from Figure 2, the three driver age groups demonstrated very different brake profiles. Middle-aged drivers responded to the stop signs by braking significantly earlier and had a comparatively slower brake pressure profile than both older and younger drivers. Younger drivers depressed the brake much later and for a shorter distance thereby reaching maximum brake pressure in significantly less time, suggesting a more sudden and harder brake. Older and middle-aged drivers had similar brake profiles at the end of the braking event. However, older drivers do show a greater change from initial to maximum brake pedal pressure in a shorter distance when compared to the middle-aged drivers.

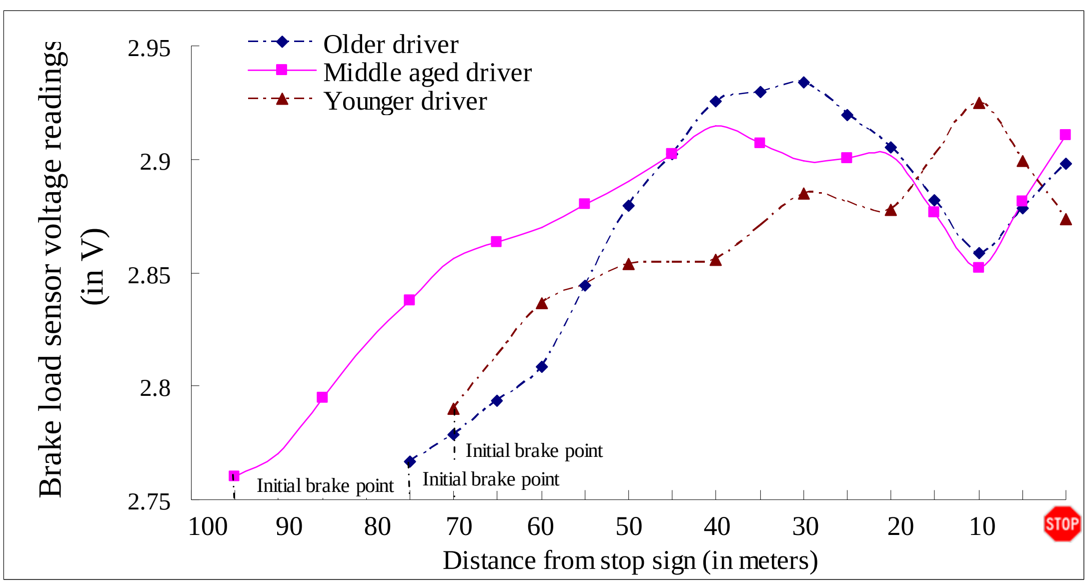

Figure 2. Profiles of brake load sensor voltage (in V) on the approach to the stop sign

Complete Stops

Both age (χ^2(2) =8.37, p=0.015) and drive maneuver (χ^2(2) =15.1, p=0.0005) had significant impacts on whether or not a driver came to a complete stop prior to entering the intersections. Younger drivers had a higher likelihood of not coming to a complete stop when compared to middle-aged drivers (Table 1). As observed in Figure 2, younger drivers did not maintain brake depression as forceful as the other two age groups further supporting the finding that younger drivers were less likely to come to a complete stop. The percentages of incomplete stops at the high-crash-rate intersection were 33% for young drivers, 20% for older drivers, and 7% for middle-age drivers. The possibility of an incomplete stop was higher for right turns than for other maneuvers with 3% being incomplete stops for going straight across, 17% for left-turn maneuvers, and 40% for right-turn maneuvers.

Table 1. Likelihood of an incomplete stop at a high crash rate expressway intersection

Effect	**Contrast**	**Estimate**	**Std error**	**Odds ratio (95%CI)**	***p*-value**
Age	Older vs. middle-aged	1.40	0.90	4.05 (0.69, 24.05)	NS
	Younger vs. middle-aged	2.27	0.89	9.67 (1.69, 55.9)	0.010
	Younger vs. older	0.87	0.60	2.39 (0.64, 8.99)	NS
Drive Maneuver	Right turn vs. straight across	3.21	1.11	24.53 (2.8, 221.4)	0.004
	Right turn vs. left turn	1.37	0.66	3.94 (1.1, 14.5)	0.040
	Left turn vs. straight across	1.84	1.14	6.23 (0.66, 59.7)	NS

DISCUSSION

The objective of this study was to investigate how drivers in different age groups respond to stop signs at a high-crash-rate rural expressway intersection. This study used data gathered from an on-road experiment that was conducted in the state of Iowa. The high-crash-rate intersection was the focus of the analyses and paper because differences that may be observed in age would be more readily apparent if crashes were greatly influenced by that factor. Results of this study confirm the initial hypothesis that differences in age groups do exist. More specifically, middle-aged drivers have a steadier and more gradual brake profile when compared to the other two age groups and, therefore, respond much earlier when approaching a TWSC intersection. Although older drivers and middle-aged drivers transitioned from initial braking to maximum braking within a similar timeframe, older drivers appear to have much higher brake pressure values than middle-aged drivers. Other studies have also shown that intersections are problematic for older drivers (Preusser et al. 1998; Bayam, Liebowitz, and Agresti 2005), and this may be explained by the braking profiles observed among this age group.

Older drivers tend to be more cautious and usually compensate for driving difficulties by avoiding problematic situations (Stamatiadis, Taylor, and McKelvey 1991; Hakamies-Blomqvist 1994). Decreasing functional abilities of older drivers might provide one explanation why they respond to stop signs significantly slower and more sudden than middle-aged drivers. After the age of 65, most adults experience measurable functional impairment including a reduction in the visual field (Preusser et al. 1998). Driving requires continual and complex visual search and decision making to appropriately respond to abrupt changes and potential hazards that can arise. This can be particularly demanding at complicated roadways such as those observed in a two-way stop-controlled rural expressway intersection. Due to the high speed and density of non-stop traffic on major highways, drivers on minor roads at these intersections need to have enough time to perceive and appropriately judge the cross traffic. Younger drivers also showed greater brake pedal differential time when compared to middle-aged drivers. They also appeared to brake more sudden and harder. Their ability to respond quickly to situations appears adequate, but their decision to wait before initiating a response can pose some additional risks, including a greater likelihood of rear-ends collisions. Younger drivers also have greater crash risks at intersections (Retting, Weinstein, and Solomon 2003) and education on appropriate decision making and the consequences of inappropriate decision making may be of value.

All drivers were less likely to come to a complete stop prior to executing a right turn when compared to the other two intersection maneuvers (i.e., going straight across and making left turns). This finding is expected given that there is only one direction of traffic that needs to be observed for potential conflicts prior to the maneuver. Most drivers have a greater comfort level merging into the same direction than judging gap acceptance for left-turn maneuvers and traversing across multiple traffic streams. However, the higher likelihood of incomplete stops at right-turn maneuvers may result in more right-angle collisions and requires further research.

In conclusion, both younger and older drivers appear to take greater risks when compared to middle-aged drivers as exhibited by their braking profiles prior to entering rural expressway intersections. It is important to note that these analyses were based on a subset of data. There will be a total of 60 drivers at study completion, and all drivers' braking profiles will be examined and compared with these existing results. The analyses will also include physiological measures (i.e., heart rate variability) and differences in visual scanning patterns. Thus, future analysis will examine all these outcomes across the high and low crash rate rural expressway intersections in enhancing our understanding of driver performance and mental workload.

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Early-Age Cracking of Structures, TxDOT's Experience

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ABSTRACT

Early-age cracking of elements such as pavements and bridge decks may adversely affect the service life of the structure and negatively influence its durability. Although there are a number of causes, one primary cause is due to rapid drying of the concrete surface. The Texas Department of Transportation (TxDOT) has not been immune to this type of cracking, in part because of the vast number of climatic regions located within the state. Three of the most recent forensic investigations performed by TxDOT relating to this type of distress are summarized. These case studies reveal the diagnosis of the distress, including a detailed investigation and petrographic analysis, the causes (e.g., weather conditions, mixture design, construction techniques) of the distress observed, prevention techniques to avoid this problem in the future, and the suggested remedial repairs of the structures. These real-world experiences are an educational opportunity for engineers and researchers to gain more knowledge of this phenomenon to better enhance the durability of our infrastructure and to avoid similar problems in the future.

Key words: early-age cracking—drying—durability—forensic investigation

A Discussion on the Efficiency of NBI Translator Algorithm

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ABSTRACT

The National Bridge Inventory (NBI) database is an extensive source of information on highway bridges in the United States. Among more than 100 NBI elements, four condition ratings are of special interest for bridge engineers and managers. These NBI elements are deck, superstructure, substructure, and culverts condition ratings (NBI items 58, 59, 60, and 62 respectively). The data for these condition ratings come from biannual bridge inspections in the field and stored in the NBI database. As a part of their bridge management programs, many states have been collecting element-level condition data (mostly Pontis inspections) for over 15 years. Element-level data provide more detailed condition data on sub-elements of the aforementioned general NBI element categories and allow bridge engineers and managers to make cost-effective decisions regarding bridge maintenance, rehabilitation, and replacement. Due to having such detailed condition data at hand, there has been an interest in developing algorithms that have the capability of estimating the NBI condition ratings for deck, superstructure, substructure, and culverts from the Pontis element inspection data. If a sound estimation tool could be developed, the biannual NBI inspections done for these condition ratings would be deemed unnecessary. The NBI Translator (developed by FHWA) is one of the algorithms that has been developed to achieve that goal. Recently, there has been some concern on the degree of accuracy of this algorithm by users of both Pontis and the translator. The Iowa Department of Transportation (Iowa DOT) uses the Pontis software for its bridge management system. The NBI Translator is part of Pontis and can be utilized to calculate the NBI ratings from element condition data. This paper presents a literature review on bridge management systems and bridge inspections in the United States. In addition, background on NBI Translator algorithm and discussions on the efficiency of the tool are provided. The paper is concluded by a comparison study between the calculated values of the NBI ratings using the translator and the field inspection data on NBI condition ratings collected by the Iowa DOT.

Keywords: asset management—bridge management—element condition ratings—NBI translator—NBI ratings

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INTRODUCTION

In the last 40 years, there has been a shift from constructing new infrastructure to maintaining and managing the built infrastructure in the United States. Assessment of the deficiencies for the nation's infrastructure gained significant importance during this period. As the infrastructure gets older, more resources are required to maintain it at an acceptable level of service. Since the funds eligible for maintenance and rehabilitation activities are limited, effective resource allocation is now more necessary than ever. Agencies are required to keep condition data on their pavements, bridges, and other infrastructure elements and justify their reasons for decision making and funding requests.

As an important segment of the infrastructure system, bridges and their management have also been in the spotlight for the last four decades. Unlike pavements, the failure of bridge structures may result in disasters. Agencies in the United States learned from these incidents and started implementing an extensive and comprehensive approach to bridge management.

Biannual National Bridge Inventory (NBI) rating is an effort to support bridge management and to form a basis for funding of bridge improvements in the United States. Agencies have also been collecting lower level detailed condition data for their bridge management systems. Modeling NBI ratings from lower level element condition data has been a topic of interest due to the significant resource savings it will facilitate (Al-Wazeer, Nutakor, and Harris 2007). There have been efforts but the degree of efficiency of the models is a discussion subject.

BRIDGE INSPECTIONS AND BRIDGE MANAGEMENT SYSTEMS IN THE UNITED STATES

On December 15, 1967, the Silver Bridge on U.S. Highway 35 suddenly collapsed into the Ohio River during rush hour (LeRose 2001). At the time of this tragic event, there were 37 vehicles crossing the bridge, and 31 of them fell down to the river. Forty-six lives were lost during this event, and nine people had severe injuries (National Transportation Safety Board 1967). In addition to the loss of life, an important road connecting West Virginia and Ohio was no longer in service. The catastrophe evoked concern over the reliability of the national network of bridges in the United States.

The 1968 Federal-Aid Highway Act put the states in action to collect and keep an inventory for Federal-aid highway system bridges. In the early 1970s, the National Bridge Inspection Standards (NBIS) that form the basis of bridge inspection and inventory in the United States today were developed and implemented by the Federal Highway Administration (FHWA). This legislation guided the data collection on bridge condition all over the nation. The failure of the Mianus River Bridge in 1983 and Schoharie Creek Bridge in 1987 were other two unfortunate events after the collapse of the Silver Bridge that drew attention to the importance of keeping the nation's bridges in sufficient condition and keeping up-to-date condition data (Small et al. 1999).

In general, bridges are inspected every two years, and the condition ratings are reported to the FHWA. The inspection data are compiled by the FHWA into the NBI. After the analysis of the data, reports on bridge conditions are prepared and submitted to Congress. Decisions on the distribution of federal funding through programs such as Highway Bridge Replacement and Rehabilitation Program are based on these reports (Dunker and Rabbat 1995).

In addition to the biannual NBI inspections, many states also collect element-level bridge condition data for the bridge management systems. Along with the Intermodal Surface Transportation Efficiency Act of 1991, which required the states to develop and implement bridge management systems, most of the states realized the importance and advantages of implementing bridge management systems. Although development of bridge management systems was made optional later in 1995 by the National Highway System Designation Act, many states decided to implement bridge management systems and took action (Sanford, Herabat, and McNeil 1999). Forty-eight states were reported to be implementing a bridge management system as of September 1996 (Scheinberg 1997). Efforts to develop efficient national bridge management tools encouraged research in the area. A research project initiated by FHWA resulted in the development of Pontis Bridge Management System which later became the most popular bridge management tool in the United States. Forty-two states reported that they considered implementing Pontis Bridge Management System. Few states preferred to develop their own bridge management systems (Pennsylvania, Alabama, New York, and North Carolina). The state of Maine implemented BRIDGIT which was developed as a result of a National Cooperative Highway Research Program (NCHRP) Project (Sanford, Herabat, and McNeil 1999).

Pontis Bridge Management System

As previously stated Pontis (Cambridge Systematics 2007) is the most popular bridge management system in the United States that aims to help transportation agencies in the decision making process regarding maintenance, rehabilitation, and replacement of bridge structures. Agencies are now aware that the aging highway system has considerable improvement needs; however, funding resources are limited. Therefore, they need to make the best possible decisions for improvement, and these decisions should be based on facts. Pontis input data structure is a relational database which contains complete bridge inventory and inspection data. FHWA and American Association of State Highway and Transportation Officials (AASHTO) adopted Commonly Recognized (CoRe) Elements for Bridge Inspection in order to standardize element-level condition data collection within the United States. Bridges are presented by the CoRe elements in Pontis, and percentage of condition states for bridge elements are inspected and stored in the database. For each bridge element, specific condition states and related deterioration models were developed. Based on this detailed element inspection data, the program keeps track of current situation, simulates future condition, identifies bridge and network level needs and makes project recommendations in order to gain maximum benefits from scarce funds.

Although Pontis has been extensively used for maintaining bridge element condition data inventory, not all states benefit from the tool for resource allocation and identifying future projects literally for the time being. Implementing a bridge management system is a big organizational change, and it takes time to prepare the organization for such a strategic change and customize the implementation.

NBI CONDITION RATINGS AND BRIDGE ELEMENT CONDITION DATA

The FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges (Coding Guide) helps inspectors for the data collection process. States are encouraged to use the coding guide for standardization purpose (Dunker and Rabbat 1995). The Structure Inventory and Appraisal Sheet (SI&A) lists the NBI items necessary for inspecting individual structures, and these items can be divided into three main categories: inventory items, condition rating items, and appraisal rating items. NBI condition rating for an element is an evaluation of its current condition when compared to its new condition. In order to make the NBI condition ratings as objective as possible the inspectors are provided with the general condition rating guidelines listed in Table 1. NBI condition rating elements are different from bridge management system elements. Three subsystems of bridges and culverts receive overall condition ratings in NBI inspections (Dunker and Rabbat 1995):

- Item No. 58 Deck
- Item No. 59 Superstructure

- Item No. 60 Substructure
- Item No. 62 Culverts

Table 1. NBI general condition rating guidelines*

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION (No problems noted)
7	GOOD CONDITION (Some minor problems)
6	SATISFACTORY CONDITION (Minor deterioration in structural elements)
5	FAIR CONDITION (Sound structural elements with minor section loss)
4	POOR CONDITION (Advanced section loss)
3	SERIOUS CONDITION (Affected structural elements from section loss)
2	CRITICAL CONDITION (Advanced deterioration of structural elements)
1	"IMMINENT" FAILURE CONDITION (Obvious movement affecting
0	FAILED CONDITION (Out of service)

*Adapted from (Dunker and Rabbat 1995)

While the NBI condition ratings are assigned according to the 0-9 scale given in Table 1, element-level data collected for bridge management systems are assigned on a scale of 1 to 3, 1 to 4 or 1 to 5 based on the particular element. Of 106 CoRe bridge elements, 21 CoRe elements describe bridge decks, 35 CoRe elements describe superstructures, 20 CoRe elements describe substructures and 4 CoRe elements describe culverts. In addition, smart flags are defined to describe special defects in miscellaneous bridge elements such as each beam, column, or girder. The rest of the CoRe elements are a variety of items such as bridge railings, joints, or bearings (Al-Wazeer, Nutakor, and Harris 2007). Condition State 1 for an element is the best condition while condition states 3, 4, or 5 present the worst conditions for particular elements. In Table 2 condition state definitions of unprotected concrete deck from Pontis element configurations are provided as an example (AASHTOWare). The percentage/quantity of an element for each defined condition state is recorded during Pontis inspections.

Table 2. Condition state definitions of unprotected concrete deck

Code	Description
1	No damage
2	Distress <= 2%
3	2-10% distress
4	10-25% distress
5	Distress >=25%

The Pontis condition inspection data with extensive detail down to each individual element made agencies and experts in the field question the redundancy of NBI inspections for the same inspected bridges. Pontis inspection results provided agencies with much more detailed condition data for the aforementioned NBI items. Using the data at hand for other data requirements when possible is essential because data collection is a time- and resource-consuming process. For the year 1986, NBI costs were estimated to be approximately $150 to 180 million (National Council on Public Works Improvement 1986). Although NBI data and Pontis inspection data have discrepancies in item definition and rating scales, researchers have been trying to make a translation from bridge element condition data to high-level NBI ratings to reduce the huge cost and time spent for data collection (10, 11). Hearn et al. (1993) developed an estimator model for the purpose which was later developed as a software tool known as the NBI Translator or BMSNBI. Pontis program has this software tool as a built-in module, and the tool can be used for the translation from a defined set of element inspection state for specified bridges in the Pontis environment.

NBI TRANSLATOR

Hearn et al. (1993) and Hearn, Cavallin, and Frangopol (1997b) developed the NBI Translator at the University of Colorado, Boulder, with the collaboration of Colorado Department of Transportation. The translator generates condition ratings for deck (Item 58), superstructure (Item 59), substructure (Item 60) and culverts (Item 62) "by linking CoRe elements to corresponding NBI fields and mapping bridge management system condition states to NBI rating scale" (Hearn et al. 1993). Bridge inspection data that contains both the NBI ratings and element level condition state data of approximately 35,000 bridges were used to calibrate the NBI Translator (Hearn, Cavallin, and Frangopol 1997b).

Generation of NBI condition ratings is realized in four main steps (Hearn, Cavallin, and Frangopol 1997b). First, CoRe elements are grouped into matching NBI fields. Then, NBI condition ratings are generated for individual elements based on the quantities of that element in the different condition states. This table-driven procedure is shown in Figure 1 (adapted from Hearn, Cavallin, and Frangopol 1997b).

Requirements on element quantities		NBI Rating
P_1	$M_{1,9}$	9
P_1+P_2	$M_{2,9}$	
$P_1+P_2+P_3$	$M_{3,9}$	
$P_1+P_2+P_3+P_4$	$M_{4,9}$	
P_1	$M_{1,8}$	8
P_1+P_2	$M_{2,8}$	
$P_1+P_2+P_3$	$M_{3,8}$	
$P_1+P_2+P_3+P_4$	$M_{4,8}$	

Figure 1. Table for NBI Generation modify according to the guide

Hearn, Cavallin, and Frangopol (1997b) describe the table-driven element NBI generation as follows: Percentages of element quantities in condition states are denoted by P_i and taken from element inspection records. Each row in Figure 1 checks the sum of percentages for a minimum required sum. These minimum required sums, denoted by $M_{i,j}$, are called mapping constants. As previously mentioned, number and definition of condition states differ for CoRe elements for each material and use. For example, the condition states for steel deck are different from reinforced concrete deck. Overall, 20 different maps are required for generating NBI ratings. The four requirements for each NBI rating should be satisfied at the same time to assign that particular NBI rating to that particular element. The calibration process estimates these mapping constants. After assigning the NBI ratings for all elements, NBI ratings for each item (deck, superstructure, substructure, and culverts) are calculated by a weighted combination of element ratings. While the weights for deck and superstructure fields are based on relative quantity, the weights for substructure field are based on number of spans. Finally, NBI condition ratings are modified based on the smart flag condition reports. Smart flags may reduce the NBI ratings by a maximum of three points.

The objective of the calibration process is to find the mapping constants that will lead to the minimum difference between the NBI ratings given by inspectors and the generated NBI ratings from the element condition data.

Discussions on the NBI Translator Algorithm

Although the PC-based version of the NBI Translator algorithm has been available since 1994, the traditional NBI inspections for bridge subsystems are still being done since the translator results are not accepted as satisfactory. In some of the states that have access to the NBI Translator through Pontis, bridge engineers reported that they have concerns regarding the efficiency of the tool. A recent study (Hale, Hale, Sharpe 2007) on bridge management involving 17 state DOTs reported a general skepticism on the estimation accuracy of the NBI Translator. Among these states only Oklahoma has been using the rating translator. However, due to the variance of the generated ratings they are in the process of stopping the use of the translator. In another study, Scherschligt (2005) reports that Kansas Department of Transportation (KDOT) evaluated NBI Translator results as an alternative of performance measure for bridge prioritization. The coefficient of determination between generated and real ratings was only 25%. This implies that the translator was able to explain only 25% of the variation in the NBI ratings in the best case. KDOT decided that the translator results were statistically insufficient and inconsistent. Therefore, they eliminated the NBI Translator results from their alternatives of performance measures.

A study by Al-Wazeer, Nutakor, and Harris (2007) proposes an alternative for NBI generation to improve the results of NBI Translator. Based on data from Wisconsin and Maryland, artificial neural network (ANN) models were developed, and results of ANN models were statistically compared with the NBI Translator results. The statistical comparison was based on the differences between the predicted and the actual observed NBI ratings. NBI error ranges were defined such as:

- NBI Error = 0 (the difference between the predicted and the actual observed NBI rating is zero)
- NBI Error = 1 (the difference is equal to the absolute value of one)
- NBI Error = 2 (the difference is equal to the absolute value of two)
- NBI Error > 2 (the absolute value of the difference is greater than two)

Comparisons based on aforementioned error ranges showed that ANN model had a higher estimation capability with respect to the NBI Translator model for a particular state when the data used for ANN training is from the same state. The superiority of ANN model to NBI Translator cannot be generalized since the statistical results are valid for only the data used in the study. However, the study drew attention to the importance of customizing the prediction model for each state.

STATISTICAL COMPARISON OF ACTUAL AND GENERATED RATINGS FOR IOWA BRIDGES

For the state of Iowa, NBI generation from the element-level condition data was performed using the built-in NBI Translator in Pontis software. Six hundred and eighty data points were used for the analysis of culvert ratings and 3,038 data points were used for the analysis of substructure, superstructure, and deck ratings. Before using NBI Translator for Iowa bridges, it was customized according to the element configuration of the Iowa Bridge Management System. This customization was done by modifying the driver file, Elements.prn, in the Pontis program folder which defines the elements to be included in NBI generation (Hearn, Cavallin, and Frangopol 1997b). First, the list of elements defined in original Elements.prn file in the program folder and Iowa elements defined in the Pontis inspection manual were compared to find the differences. Some elements that were included in the original Elements.prn file were not being used in the Iowa Pontis system; therefore, those elements were discarded in the modified Elements.prn file. Some elements had different numbers in the Iowa system, and they were also renumbered accordingly in the driver file.

Elements.prn file contains seven fields of information. These information fields are element ID (element number), element NBI field (deck, superstructure e.g.), element material (unpainted steel, masonry, smart flag e.g.), element type (slab, truss bottom chord e.g.), element dimension (each, square feet e.g.), element name in both long and short forms (Hearn, Cavallin, and Frangopol 1997b). There were some elements in the Iowa Pontis elements which were not defined within the original Elements.prn file. In order to include these elements in the NBI generation, all seven fields of information for each element were coded into the modified Elements.prn file. The list of codes necessary for modifying Elements.prn file is provided by Hearn et al. (1993). After making all the modifications to the driver file, the modified file in the Pontis program folder is replaced by the modified version and used in NBI generation.

Figures 2–5 summarize the findings of the comparison. For each rating item the percentage distribution of actual and generated ratings among the data set are presented as pie charts. Figure 2 shows that the NBI Translator estimates lower deck ratings than the actual observed deck ratings. While 34% of actual deck ratings have values of 8 and 9, the NBI Translator estimates no deck rating within this range.

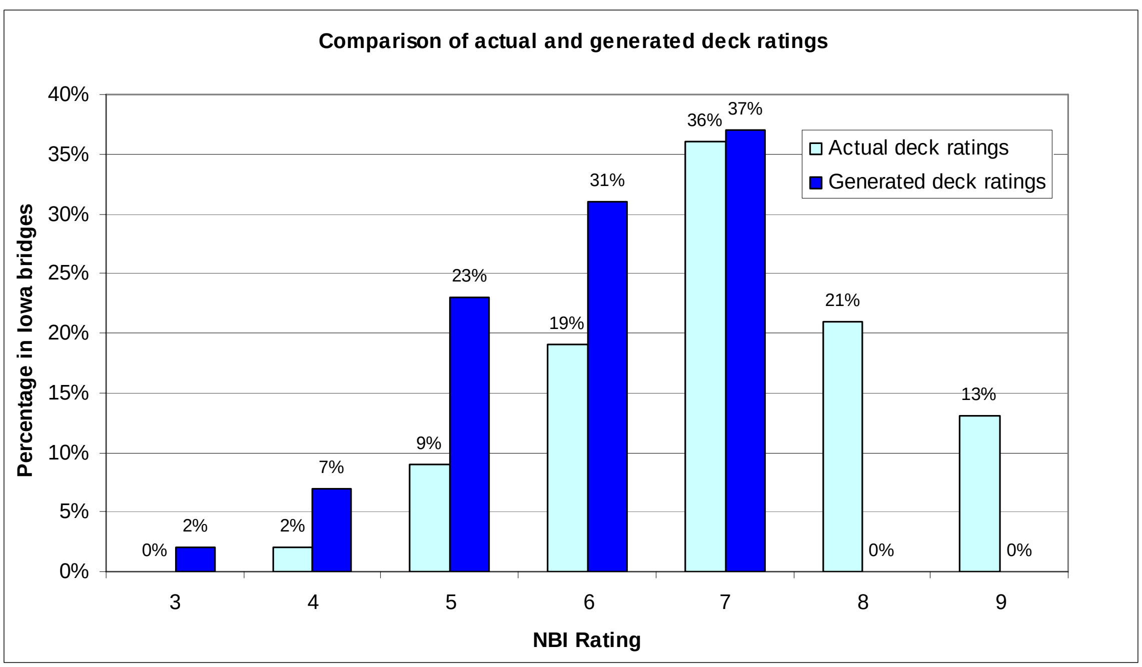

Figure 2. Comparison of actual and generated deck ratings

Figure 3 shows the comparisons for superstructure ratings. While 19% of the actual ratings are equal to 9, no observation equal to 9 appears in the generated ratings. The percentages of generated 5, 6, and 8 ratings are greater than the actual case, while the percentage of generated 7 ratings is lower than the actual case.

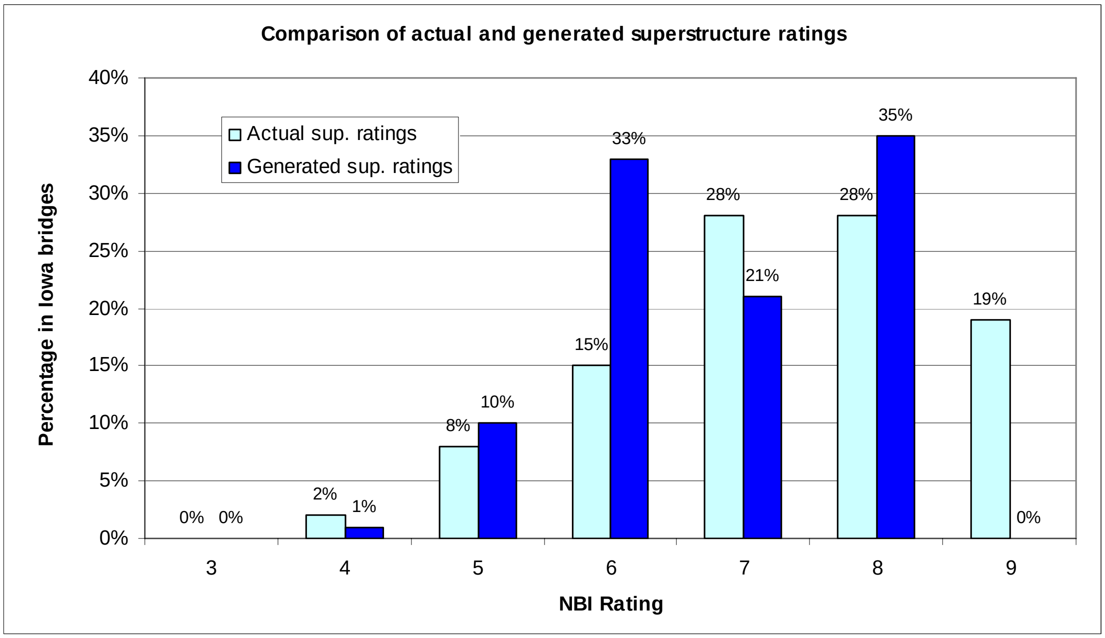

Figure 3. Comparison of actual and generated superstructure ratings

For the substructure ratings, once again, the NBI Translator algorithm tends to estimate lower values than the actual assigned ratings (Fig. 4). The percentages of 4, 5, and 6 ratings are very close for substructures. However, approximately 45% of the actual ratings that are equal to 8 and 9 are lost in the generated ratings.

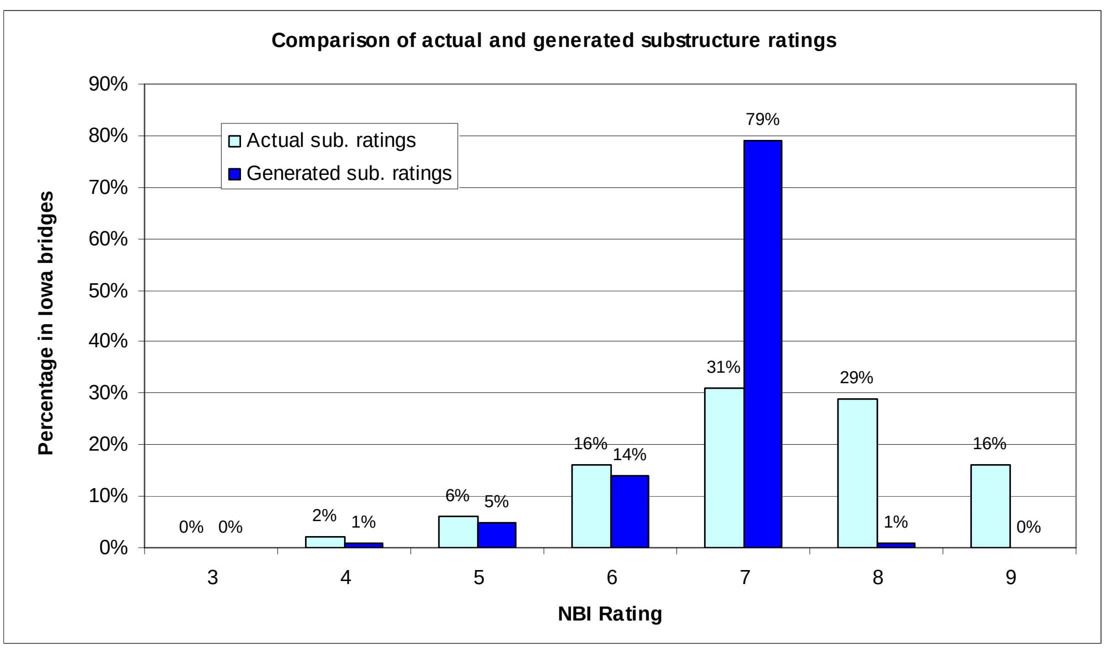

Figure 4. Comparison of actual and generated substructure ratings

For culverts, the algorithm generates 20% more 8 ratings, 22 % more 7 ratings, and 19 % fewer 6 ratings than the actual case. Once again, no 9 rating is generated by the algorithm (Fig. 5).

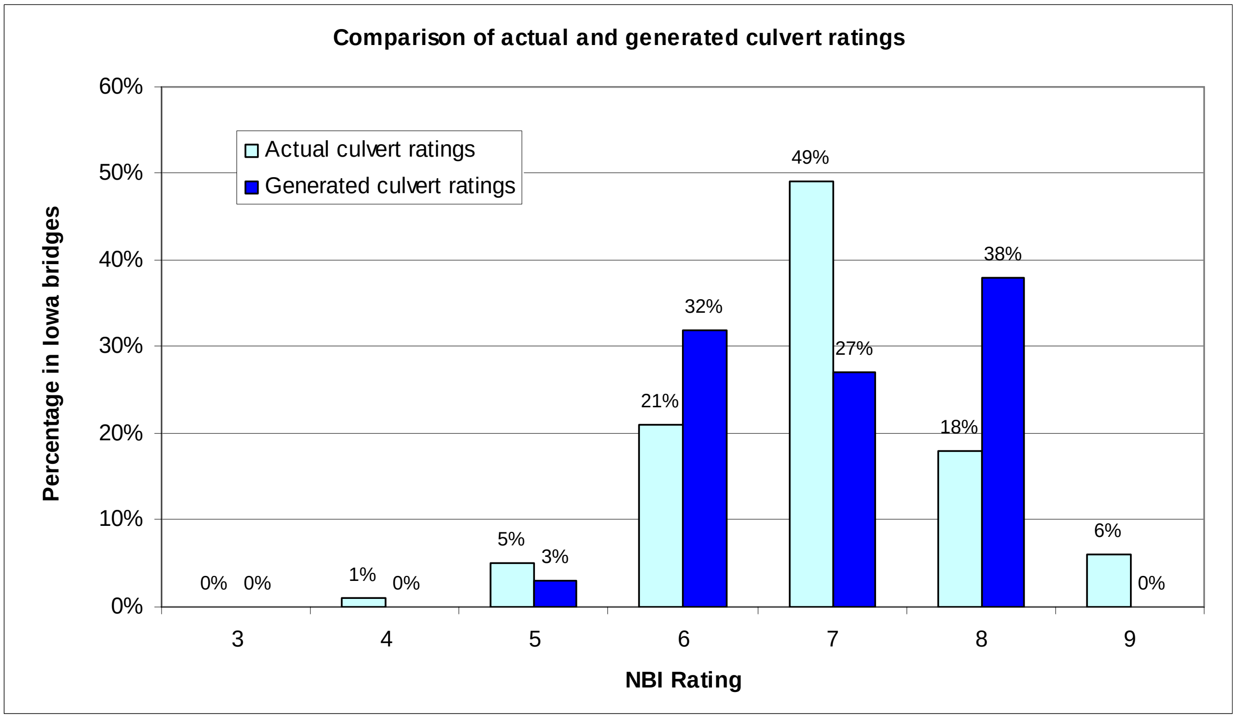

Figure 5. Comparison of actual and generated culvert ratings

CONCLUSION

This research paper reviewed bridge condition data and management in the United States and focused particularly on the estimation of NBI ratings from already collected bridge element condition data. The best-known algorithm for the purpose, which is also available within the most popular bridge management software in the United States was investigated and evaluated by a case study for Iowa bridge data.

The results of the statistical comparison for Iowa bridges showed that the generated ratings by NBI Translator algorithm with its current configuration are not representative of the actual NBI ratings. The results are supporting the concerns on the efficiency of the algorithm that have been previously reported. A more customized model for Iowa can lead to a more sufficient model. Using mapping constants specific to only Iowa bridge data instead of using the mapping constants calibrated with the data from 11 different states while creating the translator algorithm may be an option. An improved and more customized algorithm may yield better estimates of NBI ratings. A follow up to another study in the literature, an artificial neural network model can be developed for Iowa as another future research alternative. The current NBI rating system is prone to variation from the subjectivity of inspector decisions. An ultimate rating system based on more objective element condition data would result in a more consistent evaluation for the bridges in the states.

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Systematic Evaluation of a Dynamic Late Merge System in Central Missouri

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ABSTRACT

This paper describes the systematic evaluation of a dynamic merge system (DMS) deployed at a short-term work zone on I-44 near Lebanon, Missouri. Qualitative and quantitative data were collected for this evaluation from various sources. The data included Missouri Department of Transportation (MoDOT) staff feedback, video data at two locations upstream of the merge point and at two locations within the work zone, vehicle speed collected using speed radar, and global positioning systems within and beyond the work zone, traffic parameters derived from video, and detector and message logs from the DMS vendor. Eight objectives were formulated, and a performance measure and data collection methods for each were defined. The objectives were to evaluate ease of installation and startup of the system, system operation compared with system specifications, positioning of changeable message signs (CMS), accuracy and availability of traffic information from the vendor, work zone capacity, driver compliance to changeable message signs, safety, and overall effectiveness of the DMS. In terms of the operation of the system, system messages for late merge and high-speed merge conditions were evaluated as per the specifications provided by MoDOT. Based on an evaluation using data from the vendor and the data collected onsite independent of vendor's data, the late merge mode operated according to the

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specifications, but the high-speed mode did not. In terms of the accuracy and availability of traffic information, the system performed adequately. The positioning of the DMS components was also adequate, since the longest queue only extended to the CMS or approximately one mile upstream from the merge point. The lack of extensive queuing was due to the rerouting of traffic carried out by MoDOT. The comparison between the DMS work zone and the standard work zone showed that there was no difference in the traffic characteristics, i.e., neither the flow rates nor the discontinuous lane usage was significantly different between the DMS and the standard work zones. Average speeds for the left and right lanes near the merge point were also found to be similar during the DMS and standard operations. The main reason was that the work zone was already operating in a static late merge fashion as a result of the system setup and topography of the site. There was also no difference in vehicle speeds and speed variability during high-speed conditions. No clear evidence was found that the DMS improved the safety near the merge point of the work zone.

Keywords: changeable message signs—dynamic late merge system—work zone

Precast, Post-Tensioned Bridge Approach Pavement

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ABSTRACT

Bridge owners are frequently faced with the need to replace critical bridge components during strictly limited or overnight road closure periods. This paper presents the development, testing, installation, and monitoring of a precast, post-tensioned bridge approach pavement that was specifically designed for the Iowa Department of Transportation to address this condition.

Roadway pavement adjacent to a bridge has frequently been observed to undergo settlement due to a variety of causes. This settlement results in a loss of support for the pavement, and, if left untouched, the settlement creates a very noticeable bump at the end of the bridge. This bump can be very frustrating and dangerous for the traveling public. In addition, the bump results in considerable wear on the ends of the bridge and requires expensive maintenance for bridge owners.

A precast, post-tensioned concrete approach pavement was designed and constructed as part of the Federal Highway Administration's Concrete Pavement Technology program. The precast approach pavement system is intended for use in either new construction or retrofit applications and can be installed in single-lane widths to permit staged construction under traffic. This precast pavement offers a number of additional benefits, including the ability to span the areas of soil settlement near the bridge, significantly reducing the bump.

A project site near Sheldon, Iowa, was chosen as the location for installation of this precast bridge approach pavement. In order to eliminate a problematic open joint in the pavement near the abutment, the precast pavement was structurally connected to the bridge abutment using steel dowel rods. This type of connection had not previously been used by the Iowa Department of Transportation.

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

A structural health monitoring system was installed to document and evaluate the performance of both the approach pavement and the bridge over one full year of thermal expansion and contraction.

This presentation describes the development of the precast bridge approach pavement, its installation in the field, and the results of structural monitoring that will be continuing through the end of 2007.

Key words: bridge repair—precast—post-tensioned pavement—post-tensioning

Freight Planning at Small MPOs: Current Practices and Challenges

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ABSTRACT

This paper investigates the current state of planning activities among MPOs in the Midwest region with populations between 50,000 and 200,000. A sample of 19 MPOs is included. Using survey response data, we seek to determine (1) if freight is addressed in planning activities, (2) which freight-related issues are of most interest, (3) which methods are utilized for outreach, data collection, modeling, and analysis, and (4) similarities and differences in freight planning among MPOs. Attention is given to selected freight issues, including safety, congestion, air quality, impacts on low-income and minority populations, outreach activities, and integrating modes of freight. We conclude that the majority of survey respondents address freight in planning activities, but the scope of freight planning is limited and varies by MPO.

Key words: freight transportation—MPO—planning

INTRODUCTION

The movement of freight has far reaching impacts. Across the United States, residents depend on products that are made available through freight networks. Improving these networks increases access to goods and strengthens the economy. Firms depend on the efficient movement of goods to stay economically competitive, shaping employment in cities across the United States and around the world. It is important to remember, however, that the movement of freight has adverse effects that must be minimized. For example, emissions from trucks, airplanes, and trains have numerous environmental impacts, damaging water systems and air quality. In addition, freight movement through communities creates noise pollution and congestion. Due to these issues and others, the federal government, state governments, and planning organizations are giving increasing attention to freight issues (NCHRP 2007). Legislation has been created that emphasizes the importance of freight in the transportation planning process, including the Intermodal Surface Transportation Efficiency Act, the Transportation Equity Act for the 21st Century, and the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users. To help planning organizations meet the goals of these acts, detailed guides to freight planning have been created (NCHRP 2007). However, knowledge about whether and how MPOs have met the freight planning–related goals of the above legislation is rather limited.

This paper investigates the current state of freight planning activities among MPOs in the Midwest region with populations between 50,000 and 200,000. Information about freight planning activities was collected through an online survey. We were particularly interested in small MPOs because of their typically limited staff and resources. Given these constraints, expanding planning activities to incorporate freight may prove particularly challenging. Thus, when reviewing this paper, it is important to remember constraints that the surveyed MPOs face.

In the remainder of this paper, we first describe the study methodology. General characteristics of the survey respondents are presented, followed by a discussion of survey results to provide an overview of current practices at participating MPOs. Next, we provide a mapping analysis of freight planning practices. We conclude with a summary of findings and suggestions for further research.

METHODOLOGY

At the initial stage of this study, we reviewed the long-range plans of small MPOs in the Midwest region in an attempt to determine (1) the amount of attention given to freight issues, (2) which freight issues present challenges for planning organizations, and (3) what actions are being pursued to address freight. However, our thorough review of long-range plans yielded little information related to freight, suggesting that either many MPOs have very limited planning activities for freight or their long-range plans do not reflect the most up to date practices. To gain further insight into what MPOs are currently doing with regard to freight, our team devised an online survey to gather information directly from MPOs.

The design of our survey questions was guided by the existing literature, as summarized in the following section, and the information gathered from our initial review of long-range plans. The focus of the survey was to gain knowledge in the following four areas:

1. Whether freight is or is not addressed in planning activities
2. Which freight-related areas are addressed
3. Methods utilized for outreach, data collection, modeling, and analysis
4. Similarities and differences in freight planning among small MPOs in the Midwest

A link to the online survey was sent via email to all MPOs in the Midwest region with a population between 50,000 and 200,000, totaling 56 MPOs. Figure 1 displays the geographic location of the MPOs contacted and those participating in the survey. A total of 19 responses were deemed adequate for inclusion in our analysis, yielding a response rate of 34%.

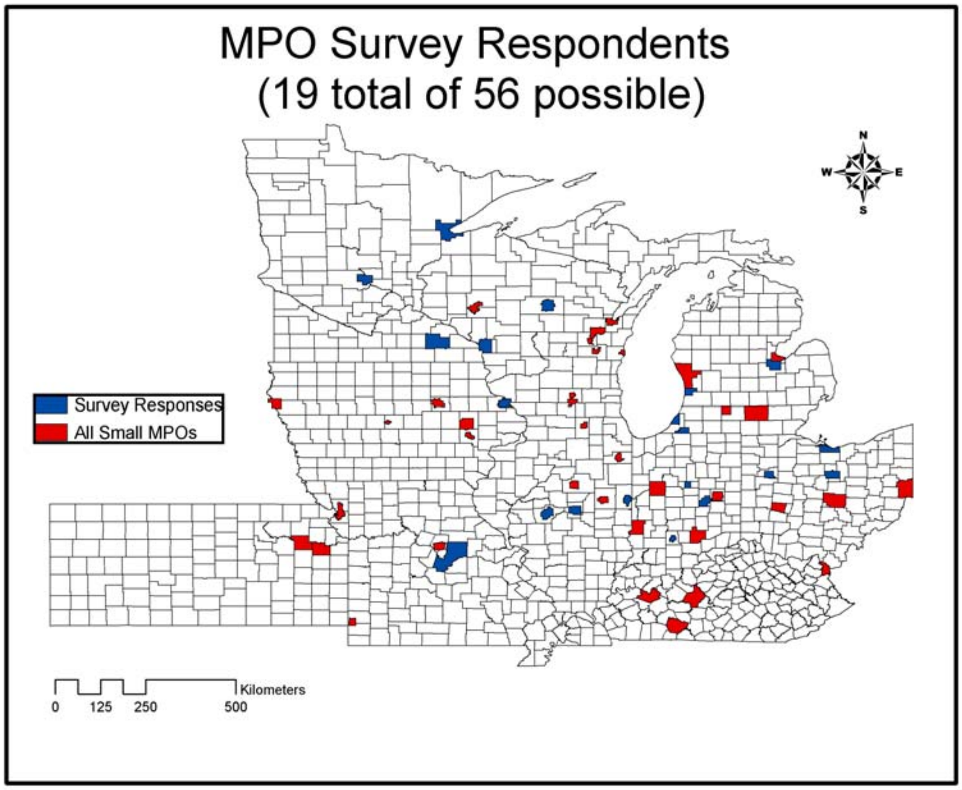

Figure 1. Geographic locations of small MPOs surveyed in the Midwest region

RESPONDENT CHARACTERISTICS

Table 1 provides information about the population size, staff size, designation year, and year of the most recent long-range plan corresponding to each of the MPOs that responded to our survey. The variation in the MPO size and age is particularly interesting. A few MPOs have no more than 2 staff members, while others have as many as 11. Some MPOs have been in existence for almost 40 years, while three MPOs have only been established in 2003. These significant variations suggest that the MPOs may have very different levels of resources and experience. These variations must be taken into consideration when interpreting the survey responses.

Table 1. Characteristics of survey respondents

MPO	State	Population	Number of Staff	Year of Last LRP	Designation Year
Dubuque Metropolitan Area Transportation Study	IA	76,932	4	2006	1974
Danville Area Transportation Study	IL	59,598	1	2005	2003
Decatur Urbanized Area Transportation Study	IL	105,435	4	2004	1964
Springfield Area Transportation Study	IL	161,792	9	2005	1962
Bloomington/Monroe County MPO	IN	98,945	2	2006	1982
Kokomo & Howard County Governmental Coordinating Council	IN	70,619	4	2005	1982
Madison County Council of Governments	IN	142,134	10	2005	1969
Macatawa Area Coordinating Council	MI	112,467	2	2007	1991
Saginaw Metropolitan Area Transportation Study	MI	159,102	3	2007	1965
Twin Cities Area Transportation Study (Southwest Michigan Planning Commission)	MI	121,280	11	2005	1974
Duluth - Superior Metropolitan Interstate Council	MN	145,163	8	2005	1975
Rochester-Olmsted Council of Governments	MN	127,758	6	2005	1972
St. Cloud Area Planning Organization	MN	104,447	6	2005	1970
Capital Area MPO	MO	67,160	3	2007	2003
Erie County Metropolitan Planning Organization	OH	79,551	2	2005	2003
Lima-Allen County Regional Planning Commission	OH	110,864	3	2005	1964
Richland County Regional Planning Commission	OH	98,927	9	2005	1962
La Crosse Area Planning Committee	WI	108,831	2	2005	1967
Marathon County Metropolitan Planning Commission	WI	83,817	1.5	2006	1983

To help interpret the survey findings, we also examined the current air quality status and freight facilities in these metropolitan planning areas. Five of the 19 MPO respondents are currently designated as nonattainment for ozone: Macatawa, Southwestern Michigan, Erie, Lima-Allen, and Madison County (see Figure 2). As one would expect, these MPOs are more concerned with air quality in the context of freight planning and are more likely to use vehicle emissions models in their planning efforts.

Intermodal centers are locations where freight is transferred from one mode of transportation to another. These centers are scattered across the Midwest, usually in areas of dense population and industry. However, as shown in Figure 3, most of the small MPOs who responded to our survey have at least one intermodal center within 20 km of their planning area. This suggests that freight transportation would likely be of interest and concern to these MPOs.

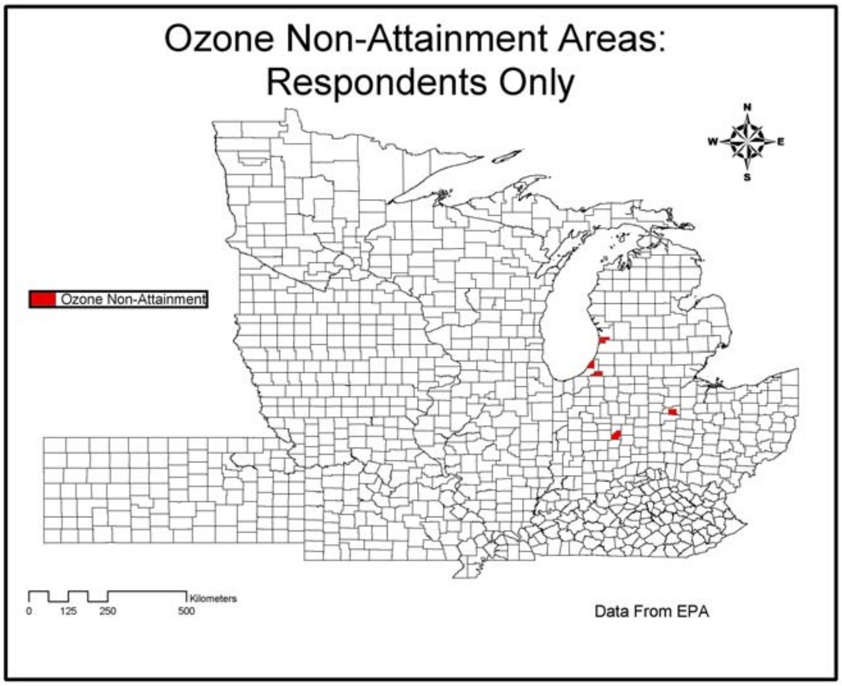

Figure 2. MPOs designated as non-attainment for ozone

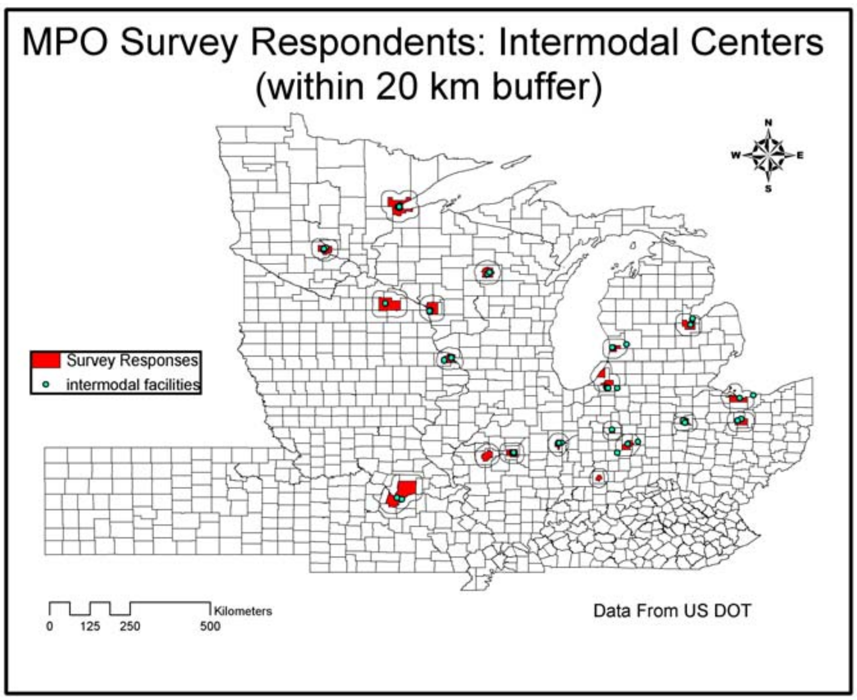

Figure 3. Intermodal facilities in proximity to MPO respondents

SURVEY RESULTS

This section summarizes and reports the total of 19 survey responses in the following topics: overall plan, outreach, freight issues, data collection, models, and training needs. It should be noted that the goal of this section is to disseminate information about activities that are currently taking place within these MPOs. Reasoning as to why activities are or are not taking place is therefore beyond the scope of this study.

Overall Plan

Of the 19 MPOs included in this study, 16 reported that they had addressed freight in their latest long-range plan. Modes of freight considered in each long-range plan varied by MPO, as shown in Table 1. Rail was most common, followed by truck travel. Reasons for beginning freight plans also varied across MPOs. Many MPOs included in this study do not have a freight plan. Of those that do, most developed the plan to comply with federal regulations (Table 2). Moreover, of the MPOs that do have freight plans, most consist of an inventory of freight-related facilities and generators (Table 3).

Table 2. Freight modes considered in MPO long range plans

Mode	No. MPOs
Truck	14
Rail	15
Air	9
Water	9
Truck/Rail Intermodal	6
Rail/Water Intermodal	3
Truck/Water Intermodal	3

Table 3. Motivation for developing a freight plan

Motivation	No. MPOs
N/A: Do not have a freight plan	8
To fulfill federal requirement	7
To address current freight problem	2
To address emerging freight problem	2

Table 4. Characterization of freight plan

Plan Focus	No. MPOs
N/A: Do not have a freight plan	8
Primarily an inventory of freight-related facilities and generators	7
Primarily a quantitative document	1
Primarily a policy document	1
A combination of the above	2

Outreach

A majority of our survey respondents engage in outreach activities as part of their planning process, as shown in Figure 4. Respondents were asked how much freight carriers, freight shippers, and residents each participate in the planning process. Response options included a lot, somewhat, very little or not at all. Responses for carriers, shippers and residents are provided in Table 4. It is important to note that none of the MPOs reported carriers, shippers, or residents as participating a lot in the planning process. Additionally, residents make up the only group that was never reported as not participating at all.

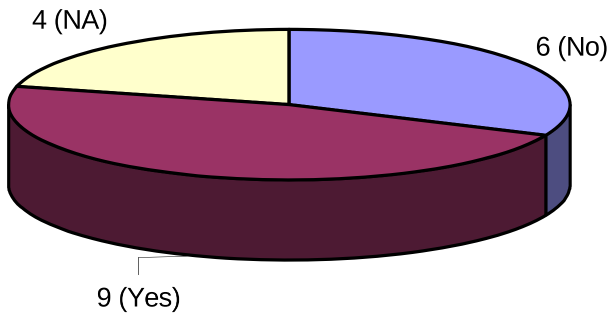

Figure 4. MPO experience with outreach activities

Table 5. Level of public participation in freight planning

Level	Freight Carriers	Freight Shippers	Residents
Not at all	2	2	0
Very little	4	5	2
Somewhat	5	3	7
A lot	0	0	0
N/A	8	9	10

Eight respondents provided details on methods that have proven most effective in gaining participation from freight carriers. The most common method was reported by four MPOs and includes a combination of private meetings held with specific individuals and phone calls. Other responses were not shared among MPOs and include private meetings held with specific individuals (without phone calls), participating in a transportation forum, and email correspondence. Eight respondents reported methods that have proven most effective in gaining participation from freight shippers. Only two respondents reported a combination of private meetings held with specific individuals and phone calls. Private meetings in combination with open meetings were mentioned once. Email correspondence, phone calls, participating in a transportation forum, and private meetings were each reported once individually. One respondent reported regular focus groups with shippers and carriers as the most useful method for engaging participation from both groups.

Eleven respondents provided information on methods that have been found most effective in getting participation from residents. Five MPOs reported open meetings alone as the best method. Five additional MPOs included open meetings in addition to other methods as being most effective. In all, only one participant that responded to this question did not include open meetings. Following open meetings in popularity, email correspondence was reported five times. Private meetings were reported four times, phone calls three times, newsletters twice, and newspaper articles once.

Freight Issues

Participants reported the level of attention given to the following freight-related areas in their long-range plans: congestion, air quality, safety, and intermodal issues. Response options included a lot, somewhat, very little, or not at all. Table 5 summarizes the results.

Table 6. Level of attention given to freight-related issues

	Issues addressed in the long range plan in relation to freight			
Level	**Congestion**	**Air quality**	**Safety**	**Intermodal**
None	4	9	2	3
Very little	2	6	3	3
Somewhat	11	3	10	8
A lot	0	0	2	3

It is interesting to note that none of the MPOs reported giving a lot of attention to congestion, despite the prevalence of the topic found in literature. When somewhat and a lot are combined, congestion, safety and intermodal issues are given approximately the same level of attention. Air quality received much less attention. Thus, it was not surprising that, when asked if freight truck idling was addressed, all participants said no.

It is important to note that the respondents were asked to comment on each of these categories as freight-related issues. Thus, much more attention may be given to these issues when they are not considered in relation to freight.

In addition, we included impacts on low-income and minority populations as a freight-related issue in this study. When asked if they felt that low-income populations were disproportionately affected by freight travel in their region, seventeen respondents said no and one said yes. We then asked if environmental justice issues related to freight travel were addressed. Twelve respondents said no, while seven provided details on how environmental justice is addressed. One respondent reported environmental justice as being a part of all planning activities, while another specified that environmental justice is addressed in all transportation improvement plans. Another respondent stated that the area where most freight travel takes place is analyzed for environmental justice issues. One respondent specified that truck routes are compared to an overlay of the locations of low-income and minority populations. Another responded that environmental justice was addressed in freight planning through public outreach. Another MPO specified that environmental justice issues were addressed through looking specifically at safety issues related to freight that affect low-income and minority groups.

Data Collection

Eleven participants responded that they do use freight data, and seven responded that they do not. Of those using freight data, five MPOs primarily used a state source to collect freight data. Two respondents reported using a regional source, and two reported using surveys/interviews to gather data. One respondent reported using a local source, and one reported using a federal source. We then asked which type of freight data is collected. Respondents were given the option of selecting up to eight types of data in addition to specifying other data types. Twelve MPOs provided information, as displayed in Figure 5. Six respondents reported engaging in outreach or partnership initiatives for data collection. Partnering organizations include a university, the department of transportation (DOT) for individual states, state police, local freight carriers, major retail stores, large manufacturers, and private firms in the freight sector. Outreach methods include personal contacts, surveys, and interviews. Twelve respondents reported analyzing freight data in-house, while one reported using a combination of in-house staff and hired contractors.

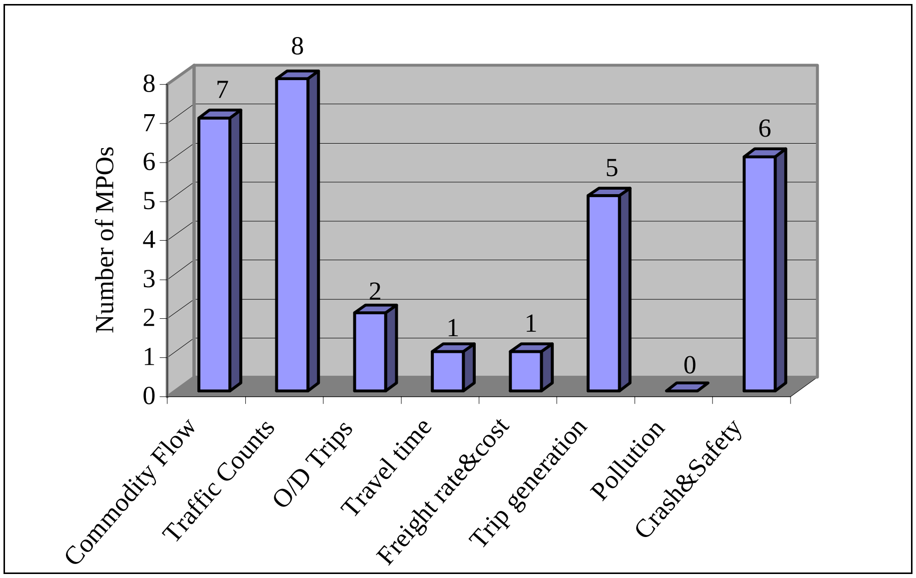

Figure 5. Type of freight data collected by MPOs

Models

Thirteen MPOs reported not using models to analyze economic, traffic, or air quality impacts related to freight. None reported using models to analyze modal shifts. Responses specific to each issue area follow:

- Of the MPOs analyzing the economic impacts of freight, two reported using a cost-benefit model and one reported using an input-output model. Fifteen stated that no model was used.
- Four MPOs reported using a conventional four-stage model to analyze freight traffic. Fifteen MPOs reported not using a model for this purpose.
- Three MPOs reported using the Mobile6 model to analyze air quality, and sixteen reported not using a model at all.

Four MPOs reported partnering with their state's DOT for modeling purposes. One MPO also reported working with the city and the county. Additionally, one MPO responded that due to its small size the state's DOT provides modeling services. Twelve MPOs reported not engaging in outreach or partnership initiatives for modeling.

Training Needs

Eight MPOs responded that they would benefit from freight-related training. Four MPOs reported an interest in data collection training, while interest in data analysis training was reported by three. One MPO specified a desire for training related to safety and modal shifts, and one expressed an interest in proven techniques for freight planning.

CONCLUSIONS

Freight clearly plays a role in the planning activities of the majority of the small MPOs analyzed in this paper. Most freight-related planning initiatives, however, are limited in capacity. Addressing the interest in further training as requested by several of the MPOs may help to broaden the scope of freight planning. Additionally, while there are variations in the level of freight planning among MPOs, similarities arise with the attention given to freight-related issues. Safety, congestion, and intermodal issues are generally given high levels of attention when freight planning does take place. Air quality, however, receives lower levels of attention. With regard to outreach, the majority of respondents reported private meetings with select individuals to be most effective for gaining participation from the private sector. Open meetings were found to be most useful for gaining participation from community residents.

It must be remembered that the MPOs analyzed are operating with extremely limited personal and resources. Given the extensive list of transportation issues small MPOs are responsible for addressing, the current level of freight planning is not surprising. Still, much can be gained from advancing the current state of the practice. Meeting the safety, congestion, equity, and economic challenges our society faces requires continuing the progress that has been made in freight planning. We intend for this paper to facilitate the transfer of information regarding current practices among small MPOs. At the very least, information on current practices may aid MPOs in knowing where to turn for guidance in expanding freight planning. In addition, this paper provides the groundwork on which additional research can be completed with the goal of aiding MPOs in freight planning. The survey experiences drawn from the current study has also helped the development of a more comprehensive survey study that is currently ongoing.

ACKOWLEDGEMENTS

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Construction and Testing of an Accelerated Bridge Construction Project in Boone County

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ABSTRACT

New bridge systems are needed that will allow components to be fabricated off-site and transported to the bridge site for quick assembly with minimal disruption to the traveling public. Depending on the specific site conditions, the use of prefabricated bridge systems can minimize traffic disruption, improve work zone safety, reduce the impact on the environment, improve constructability, increase quality, and lower life-cycle costs. This technology is applicable and needed for both the rehabilitation of existing bridges and the construction of new bridges. The Federal Highway Administration (FHWA) has recently developed a program to promote accelerated construction through the use of precast bridge elements.

This paper will present the construction process, construction schedule, and laboratory testing for one of the first applications of an accelerated bridge project utilizing precast components in the state of Iowa. Through the FHWA Innovative Bridge Research and Construction program, a bridge in Boone County, Iowa was constructed using several different precast, high-performance concrete elements.

Researchers from Iowa State University performed laboratory testing on the precast components that were used in the Boone County bridge. Field instrumentation and testing was used to verify the post-tensioning operation and to verify several of the construction methods. The laboratory portion of this investigation was funded by the Iowa Department of Transportation (Iowa DOT) and the Iowa Highway Research Board. Also, a comparison of the actual construction schedule with a theoretical schedule was completed. A discussion of the laboratory testing, structural instrumentation, monitoring, and scheduling of this innovative bridge is presented in this paper.

Key words: accelerated construction—bridge replacement—precast bridge elements—precast pier and abutment caps—post-tensioned

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INTRODUCTION

Constructing and rehabilitating bridges with minimal impact on traffic has become a transportation priority as traffic volumes nationwide increase. Renewal of the infrastructure in the United States is necessary for several reasons, including increases in population, projected increase in vehicle miles traveled, presence of obsolete or deficient structures, impact of road construction, and injuries and fatalities related to work zones (NCHRP 2003).

Rapid construction has several advantages over traditional construction methods. The six main advantages of rapid construction technology are

- Minimized traffic disruption
- Improved work zone safety
- Minimized environmental impact
- Improved constructability
- Increased quality
- Lowered life-cycle cost (NCHRP, 2003)

There are several different types of rapid construction technologies currently used in the United States. One technology uses precast concrete bridge components that are fabricated offsite, allowed to cure, and then transported to the construction site for installation. This technology allows bridges to be constructed faster than traditional construction methods, reducing the amount of time the bridge and/or associated roads are closed to the public, and reducing the total construction time. For bridges above waterways, the construction time is also reduced; thus the amount of debris that falls from the construction site is reduced, which in turn reduces the environmental impact.

The importance of rapid construction technologies has been recognized by the FHWA and the Iowa DOT Office of Bridges and Structures. This paper presents some of the results from the construction of a new accelerated construction precast bridge system located in Boone County, Iowa and evaluation of bridge components tested in the laboratory. Funding for the design, construction, and evaluation of this project was provided by the FHWA-sponsored Innovative Bridge Research and Construction (IBRC) Program. Funding for the laboratory testing was provided by the Iowa DOT and the Iowa Highway Research Board; funding for the documentation and the post-tensioning monitoring and verification was provided by the FHWA and Boone County.

This research focused on the bridge constructed on 120th Street in Boone County over Squaw Creek; the bridge replaced an existing Marsh Arch bridge at the site. The new bridge is a continuous, four-girder, three-span bridge with a full-depth, precast deck. Bridge dimensions are 151 ft. and 4 in. long and 33 ft. and 2 in. wide with spans of 47 ft. and 5 in., 56 ft. and 6 in., and 47 ft. and 5 in. Deck panels (8 in. thick, 8 ft. and 1 in. wide, and half the width of the bridge (16 ft. and 1 in.) in length), were prestressed in the transverse direction. Each panel had two full-depth channels, located over the prestressed girders, for longitudinal post-tensioning. Once the panels were erected, the entire bridge deck was post-tensioned in the longitudinal direction, after which concrete was cast in the four post-tensioning channels. Although this exact design had not been previously constructed, a similar partial-depth deck system has been constructed and tested in Nebraska (Badie, Baishya, and Tadros 1998). Precast pier caps and precast abutments were used in the bridge substructure.

BRIDGE CONSTRUCTION

Precast Fabrication

The fabricator selected by the general contractor to produce the precast elements for the Boone IBRC Bridge project was Andrews Prestressed Concrete, Inc. located in Clear Lake, Iowa. Andrews is a PCI certified plant and commonly fabricates Pretensioned Prestressed Concrete (PPC) beams for Iowa DOT projects.

Andrews initially cast three test panels that were purchased by Iowa State University (ISU) for their laboratory testing program. Prior to shipment to ISU, one of the test panels was used by the contractor to conduct a leveling device test. This test was required by the contract documents for the leveling device that was designed by the contractor. The selected leveling device operates as a screw jack; the deck panel transverse reinforcing bears on a steel plate with a nut welded to the bottom of the plate and a screw passing through the plate and nut. A pipe wrench was used to turn the screw to raise and lower the deck panel. The contractor demonstrated that the device was stable while supporting the deck panel over the PPC beam and could be adjusted to the desired elevation and deck cross slope. Once the leveling device was accepted, the bridge deck panels were fabricated.

Three deck panels could be cast in one casting operation. Panels could be fabricated every other day with a maximum of nine panels cast per week. Andrews fabricated reusable steel forms shown in Figures 1 and 2; the panels were cast on a steel casting bed in the open.

Figure 1. Panel forms and reinforcing

Figure 2. Panel form release

The anchorage zone was very short for the development of the pretensioning strands. Thus, spiral reinforcing was used to reinforce the bursting zone. Use of spiral reinforcement also improved strand development (see Figure 3). At the longitudinal centerline of the bridge, a longitudinal joint was cast in place. Reinforcement of the longitudinal joint was provided with double hairpin bars projecting from the panels, shown in Figure 4, and straight reinforcing bars threaded longitudinally. One benefit of a cast-in-place longitudinal joint at the centerline was to allow the panels to be cast flat and introduce the bridge crown in the longitudinal joint. The longitudinal joint at the centerline of the bridge did not add any construction time to the critical path because the longitudinal joint was cast concurrently with the open channels over the four beams after the post-tensioning.

Figure 3. Spiral reinforcing

Figure 4. Longitudinal joint reinforcing

For the vertical reinforcing connection in the coral style barrier rail, the contractor was given the option in the plans to project the reinforcing bars from the deck or use mechanical splicers; the contractor and fabricator chose the mechanical splicer option. End panels contained welded wire reinforcing and the post-tensioning anchorage zone. Concrete consolidation during panel concrete casting was closely monitored due to reinforcement congestion, especially due to the spiral reinforcing and welded wire reinforcing. No problems were detected in the consolidation and the concrete flowed well into the spiral reinforcing zone. A concrete strength of 4,000 psi was required for panel release which was easily achieved in 24 hours. The panels were released from the forms and stockpiled at the precast fabricator's yard to await shipment to the bridge site. Andrews was also the fabricator for the PPC beams, pier caps, and abutment caps for the project. Beams used were Iowa Standard "B" beams modified for a wider spacing than the typical standard beam spacing.

Substructure

Precast abutment caps and precast pier caps founded on H-piling and pipe piling, respectively, comprised the substructure. The units were reinforced with mild reinforcing and included blockouts for the piling that were created using corrugated metal pipe (CMP). During the design process, no research was found regarding the pile connection detail considering a bond or development of resistance between concrete and the CMP. A fairly conservative connection design was completed which was later validated by testing. The contractor had an end of driving tolerance of three in. in any direction for each H-piling in order to fit the precast abutment cap over the H-piling. Standard specifications typically only specify a start of driving tolerance for H-piling. Special plan notes were included to specify the end of driving tolerance. Care was taken during the pile driving operation, and the contractor had no problem meeting the end of pile driving tolerance or fitting the precast abutment cap over the H-piling shown in Figures 5 and 6; time required to set a single precast abutment cap was less than 30 minutes.

There were five H-piling supporting the abutment caps and nine 16 in. diameter pipe piling supporting the pier caps. The pier cap end of driving tolerance was 2 1/2 in. A driving template was fabricated that helped the contractor meet the end of driving tolerance so that the precast units fit over the piling. No problems were encountered, and the pipe piles were all well centered within the pier cap.

Figure 5. Setting precast abutment

Figure 6. Abutment CMP blockout

A high early strength concrete mix was used for filling the substructure blockouts. The concrete was cast with a maximum slump of two inches prior to adding a high-range water reducer (HRWR) to improve workability. With the HRWR the maximum slump allowed was seven in. Prior to PPC beam placement the concrete was required to achieve a 3,500 psi compressive strength.

Superstructure

The superstructure for the bridge utilized traditional PPC beams. Beams were modified from the standard design in order to eliminate a beam line. A standard bridge for the county would have a five-beam cross section and that was reduced to a four-beam cross section. To modify the beams, additional prestressing strands were added and the concrete release and 28-day strengths were increased.

Erection of the PPC beams was started early in the morning and completed shortly after noon. The day following the PPC beam erection the deck panel delivery (three per truck load) was scheduled. Panels were offloaded to a storage area and then erected. Half of the panels were scheduled for delivery the first day with the remainder scheduled for the following day. Panel delivery was divided in half because the contractor had not performed an operation like this before and did not know how long the panel erection would take.

The first half of the deck panel erection took the whole day. Panels were erected from the centerline of the bridge working outward (see Figure 7). Erection of the second half of the panels took half of the day. The primary difficulty erecting the deck panels is the alignment of the first deck panel erected. Once the first panel is properly positioned, the remaining panels were uniformly offset 3/8 in. and maintained the correct alignment. Panel leveling devices were installed the same day the deck panels were erected, as shown in Figure 8.

Transverse joints were cast in place with a high early strength concrete mix. Due to the tight deck panel spacing, a small aggregate size was used with a maximum top size of 3/8 in. Maximum water cement ratio was 0.38 and the slump was increased using a HRWR that allowed the slump to go to a maximum of eight in. A retarding admixture was used as well that seemed to extend the life of the HRWR for workability.

Figure 7. Deck panel erection **Figure 8. Leveling device installation**

During the curing of concrete in the transverse joints, the post-tensioning strands were threaded through the end anchorages and down the channels for a total of 48 strands to post-tension. Each of the four channels contained twelve 0.6 in. diameter strands. The bridge was short enough to allow for post-tensioning from one end. Less than four hours was required to complete the entire post-tensioning operation. All the strands, except one, were post-tensioned with no problems. One strand became pinched between an adjacent strand and "extra" deck panel concrete. This strand was released and fully post-tensioned by applying the post-tensioning force from the opposite end of the bridge. The correct post-tensioning force application in that strand was doubly verified by gage pressure and summing the total strand elongation at each end.

Post-tensioning forces were verified by calibrated gage pressure. Strand elongation was checked as a final confirmation, shown in Figure 9. The jack stroke length was monitored during post-tensioning as a safety precaution against over tensioning.

Figure 9. Strand elongation check **Figure 10. Casting longitudinal joints**

Concrete was cast in the longitudinal joints on the same day the post-tensioning force was applied, shown in Figure 10. As shown in Figure 11, the same concrete mix used for the transverse joints was used for the longitudinal joints. The longitudinal joints were congested with post-tensioning strands, transverse mild reinforcing, transverse prestressing strands, stirrups, and leveling plates with leveling screws.

The HRWR was very effective in aiding in the placement of the concrete. Concrete consolidation observed in the longitudinal channel haunch area, shown in Figure 12, and between the strands and reinforcing was excellent. Following the curing of the longitudinal joint concrete the leveling screws were "backed out" and the hole was filled with a hydraulic cement grout.

Figure 11. Channel congestion

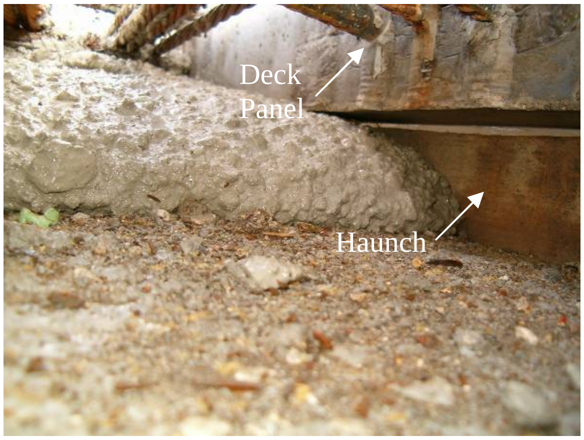

Figure 12. Concrete consolidation in haunch

A cast-in-place concrete diaphragm and deck end section was constructed to complete the integral abutment. The cast-in-place end section also allowed for panel erection tolerance; the total length of the deck panel portion of the bridge was nine inches longer than anticipated in the plans because the panels were fabricated on the high end of the dimensional fabrication tolerances.

To complete the bridge, the corral style barrier rail was cast in place, and the deck was ground for smoothness and grooved for texture prior to opening the bridge to traffic. Figures 13 and 14 show the completed bridge.

Figure 13. Bridge profile

Figure 14. Bridge approach

CONSTRUCTION SCHEDULE

Researchers on the Boone IBRC Bridge project have examined the Iowa DOT's *Weekly Report of Working Days* for the project and have created both an actual and a theoretical schedule that reflects production rates observed during the project. Both schedules have been compared and comments have been provided.

Actual Work Schedule

The Boone IBRC Bridge had a late project start date of July 5, 2006, and was specified 80 working days for contractual completion. September 8, 2006, is the date the contractor's bridge crew actually moved on site with 50 working days remaining for contractual completion. On December 28, 2006 the final task of the Boone IBRC Bridge construction was completed. A total of 90 working days were required for the project, 10 days beyond the contractual allowance. *The Weekly Report of Working Days* for the Boone IBRC Bridge from September 8, 2006 to December 28, 2006 revealed that there had been 2.5 days where the contractor was charged with a working day but did no work. Also revealed in the *Weekly Report* was that between the dates of July 5, 2006 and September 8, 2006 there had been four days the contractor was charged a working day but did no work. In total, the contractor was charged with 6.5 non-productive working days. Ultimately, the contractor was charged liquidated damages for the 10 working days over the contractual allowance of 80 days.

Theoretical Work Schedule

The theoretical schedule developed by the researchers was based on the assertion that the existing structure would have been previously removed and the abutment berms for the new structure would be in place. Durations of construction activities utilized in the theoretical schedule were obtained from actual durations observed at the Boone IBRC Bridge. It was observed by the researchers that some of the activities of construction could occur parallel or even before the initiation of onsite construction. With all of these factors taken into consideration the researchers determined that a similar structure to the Boone IBRC Bridge could be assembled in 12 working days. Cure times for any cast-in-place concrete elements is not included in arriving at the 12 working day value.

Comparison of Actual vs. Theoretical Work Schedules

A brief comparison of the actual work schedule to that of the theoretical schedule was performed. The researchers found that in theory the structure could have been assembled 48 working days ahead of the actual schedule. When the structure was constructed in the summer and fall of 2006 it took 90 working days to complete. However, only 60 of those working days were directly related to assembly of the structure (i.e. after September 8, 2006). The other 30 working days had been utilized to construct portions of the project not directly associated with bridge assembly. Because the contractor, material suppliers and engineers had never constructed a project like the Boone IBRC Bridge, there were many Requests for Information (RFI) that required answers and, therefore, the construction took longer than predicted by the theoretical schedule. In future projects contractors, material suppliers, and engineers alike will be more adapt to the accelerated construction and delivery method used at the Boone IBRC Bridge.

LABORATORY TESTING

Laboratory tests were performed on several components of the bridge substructure and superstructure. The substructure tests were performed on sections of the abutments and pier caps, and isolated shear tests of the pile connection to the abutments. Superstructure testing included deck panel testing and testing of the flowability of the concrete used in the post-tensioning channels.

Substructure Testing

A series of laboratory tests were performed by ISU to verify the strength of the abutment section before bridge construction began. Tests were performed to ensure that punching shear failure would not occur in the abutment before the CIP cap was placed. In order to verify the design strength, a ten ft. section of the abutment and pier cap was tested in the laboratory; the section of the abutment the test specimen replicates can be seen in Figure 15.

Simulated beams were fabricated out of concrete and placed on the laboratory floor. Neoprene bearing pads were placed on top of the simulated beams, and the specimens were placed on the bearing pads. In this configuration, the pile extended upwards and was loaded from above. One of the test specimens, situated under the load frame prior to loading, is shown in Figure 16. Note the specimens were inverted for stability when tested.

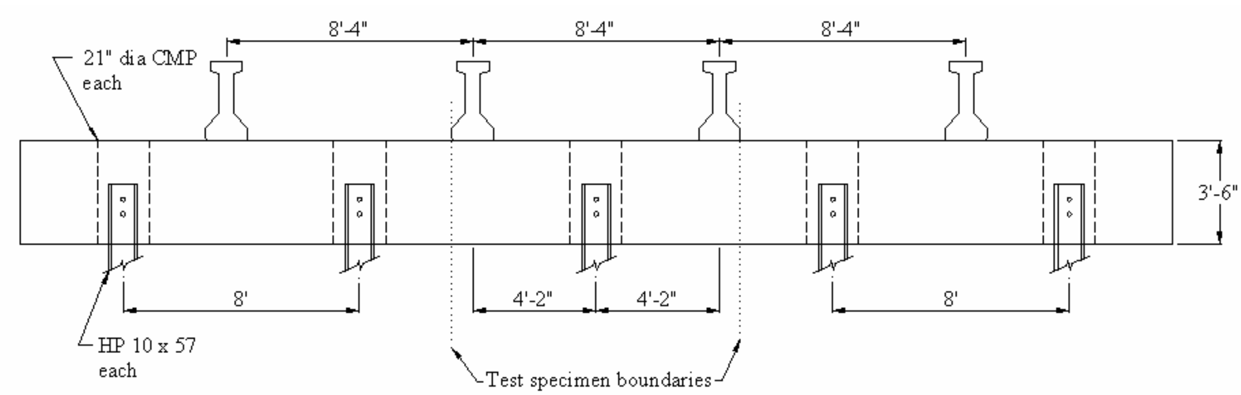

Figure 15. Side view of the precast abutment and laboratory test section

Similar tests were performed on the inverted pier cap specimen using a ten ft. section of the pier cap. Each specimen used the appropriate cross section and pipe pile for the pier caps, but otherwise used the same laboratory testing system as was used for the abutment testing.

In total, eight laboratory pile and abutment tests were performed. The average strength of the abutment specimens and pier cap specimens, along with the unfactored service loads for each is presented in Table 1. In addition to the described tests, shear tests were performed on four H-pile and CMP connections; each of the shear tests was loaded to 400 kip without cracking or appreciable differential movement. Based on the laboratory test results, punching shear failure was determined not to be of concern for the abutments or pier caps in the field.

Figure 16. Laboratory testing system

Table 1. Service load and experimental specimen strengths

Specimen	Unfactored Service Load (kip)	Average Maximum Load (kip)
Abutment	80	382
Pier Cap	72	384

Superstructure Testing

Laboratory tests were conducted to determine the flexural and punching shear capacity of the deck panels. Panels were designed for HS-20 loading. Because of the localized failure for each test, multiple tests were conducted on the specimen since previous tests on the specimen had minimal influence on the additional tests. Setup of the deck panels and locations of the three loads are shown in Figure 17. Construction of the test setup included placing each panel on two beams, placing steel for the closure joint, and casting concrete for the longitudinal and closure joints.

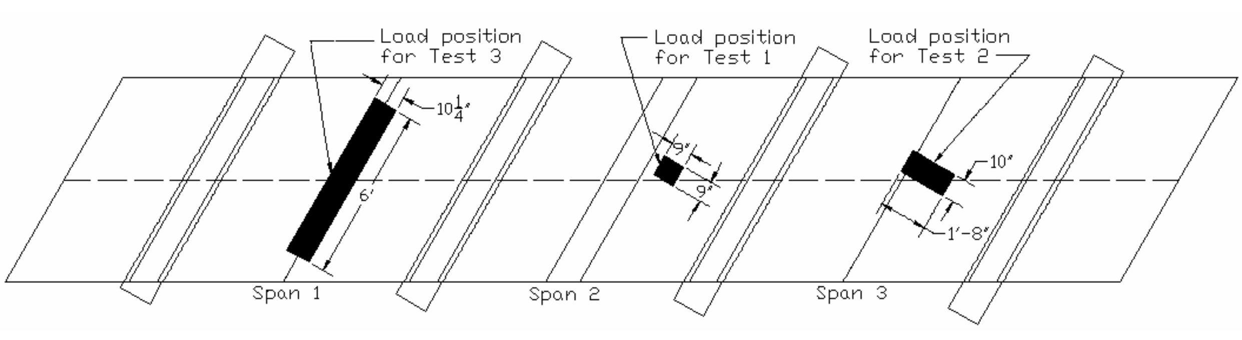

Figure 17. Load locations and footprints for bridge deck load tests

Span 2 was the first span tested. A nine in. square footprint was chosen for this test to have consistency with the service load tests previously conducted. In order to introduce shear into the closure joint and increase the probability of a failure, the load was not centered over the joint. A punching shear failure occurred at 150 kips.

Span 3 was tested second. For this span, a tandem wheel footprint was chosen so the bridge would be subjected to the standard design footprint. Also, use of this footprint gave the opportunity to see if punching shear would control in the field for the actual bridge. At a load of 150 kips, concrete began spalling from the surface of the deck panel. When the load reached 157 kips, a combination of flexural and punching shear failure occurred.

In order to determine the flexural capacity of the deck panel, a beam was placed across the center of Span 1 to act as a line load. The beam was positioned to be parallel to the support beams for the deck panels. The load beam was 6 ft. long and 10.25 in. wide. A flexural failure occurred at a load of 196 kips.

CONCLUSIONS

The following were concluded from this project:

- Placement of a single precast pier cap or abutment cap could be done in less than 30 minutes because piles were driven within tolerances.
- Deck panels for half the bridge could be erected in half of a day.
- Construction began on July 5 and was completed on December 28, requiring a total of 90 working days.
- A theoretical work schedule predicted that the Boone IBRC Bridge could be assembled in 12 working days.
- The abutment connection capacity is at least 4.5 times greater than the unfactored service load.
- The pier cap connection capacity is at least 5.3 times greater than the unfactored service load.
- Deck panels failed due to a combination of flexure and punching shear at a load of 157 kips applied by a tandem wheel footprint.

ACKNOWLEDGMENTS

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Implementing Percent within Limits for Hot Mix Asphalt

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ABSTRACT

The implementation of percent within limit (PWL) specifications for hot mix asphalt is being advocated by the Federal Highway Administration as an improved method for assessing quality over other more traditional methods utilizing mean values. A synthesis published by the National Cooperative Highway Research Program in 2005 shows that 27 out of 45 state agencies surveyed now utilize a form of PWL specifications. The purpose of this paper is to introduce PWL and some of the related aspects, including how to address risk in transitioning from more traditional construction specifications to ones utilizing PWL.

PWL specifications date back to the 1950s, when they were used by the military, and they were first applied by the New Jersey Department of Transportation in the 1970s. The main steps of application are introduced in this paper, together with linking PWL to the common statistical concept of the normal distribution. The PWL methodology has several advantages for both owner/agencies and contractors. There are also several disadvantages to PWL; one notable disadvantage is an incomplete understanding of the PWL concept. Field data are used to evaluate the implications of implementing PWL through simulations and evaluating how to balance the risk between PWL and other traditional specifications.

Key words: hot mix asphalt—pay factor—percent within limits

INTRODUCTION

One of the main issues in any construction project is the evaluation of the project. This includes acceptance or rejection of the final product or applying a factor to the project payment. Pavement is one of the aspects of construction for which these issues are applicable. As with any other project, specifications are set and contractors place their bids based on those specifications. After the project or a project stage is complete, the owner/agency evaluates the product, and based on this evaluation the contractor gets paid. The question at this point is whether the contractor will be paid in full, penalized, or rewarded. This depends on the performance and the quality of the product.

Many different types of pay factor systems exist for highway agencies. A pay factor can be defined as a multiplication factor that is often expressed as a percentage and is used to determine a contractor's payment for a unit of work, based on the estimated quality of work. This term is usually applied to quality characteristics (TRB 2005). The pay factors are calculated based on relations to the main volumetric characteristics specified in a quality control/quality assurance system for hot mix asphalt. Pay factors are determined using three main systems: "percent within limits (PWL), difference between average sample quality characteristics and target job mix formula (JMF), and probability based. All three methods use sample mean values, but only the percent within limits and probability based specifications use the sample standard deviation or variance, which takes into account material variability" (Bausano et al. 2006). In a synthesis published in 2005 by the National Highway Research Program (NCHRP), the most commonly used method is PWL or its complement, percent defective (Hughes 2005). Figure 1 shows that 27 out of the 45 agencies surveyed apply PWL. This paper will focus on the PWL specifications due to the importance of this methodology.

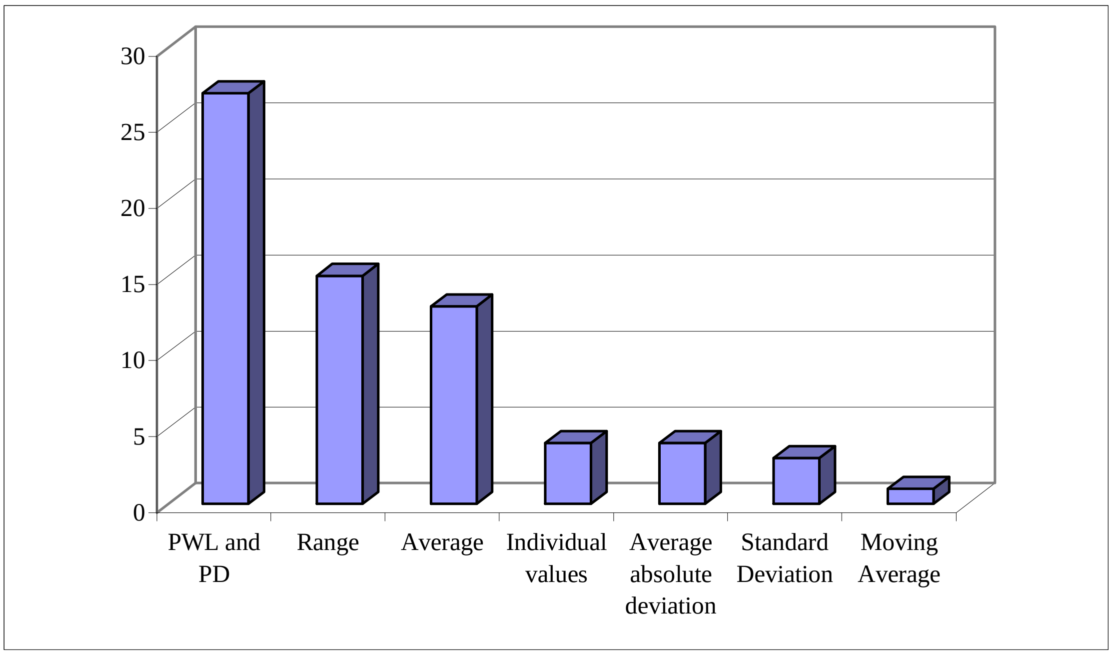

Figure 1. Results from NCHRP Synthesis 346 (Hughes 2005)

BACKGROUND INFORMATION

PWL is defined as the percentage of the lot falling above the lower specification limit (LSL), beneath the upper specification limit (USL), or between the LSL and the USL. PWL may refer to either the population value or the sample estimate of the population value. The following formula describes PWL: PWL = 100 – PD, where PD is the percentage defective (TRB 2005). The history of PWL dates back to the 1950s, when the military started a percent defective method (Newcomb and Epps 2001). The New Jersey Department of Transportation was the first to implement a PWL methodology in the 1970s; however, at that time the name “percent defective” was utilized (Hughes 2006). Later, partially for psychological reasons, the phrase “percent within limits” was implemented in the highway industry (Newcomb and Epps 2001). The other primary reason for the switch to the PWL methodology is that the rejected portion is not defective as the name implies, but it is just of lower quality than the specification limits (Hughes 2006). The PWL, or its complement, the PD, is the recommended statistical measure for material and construction quality in many places. In many cases, the acceptable quality level is 90% within limits, which means 10% is defective or of lesser quality. The advantage of this method is that it combines both the mean and the standard deviation into a single quality measure. For most acceptance quality characteristics, PWL provides a better measure of specified quality than other single statistical measures, such as the average, moving average, average absolute deviation, conformal index, or various other quality indexes (Kopac 2005).

METHODOLOGY

The concept of applying the PWL procedure is simple and based on the concept of the normal distribution. In a normal distribution curve, the percentage of the population within certain limits can be calculated by knowing the area under the curve. The same concept is applied to specify the percentage within limits for a certain lot. There is in this case the quality index, Q, that is used to estimate the PWL; it is equivalent to the z-value for the normal distribution curve. The Q value is used with a PWL table to determine the estimated PWL for the lot (Burati et al. 2003). The distance of offset of the sample mean from the specification limits in sample standard deviation units is represented by the Q value. The number of standard deviation units that a sample mean falls inside the specification limits is represented by a positive Q value. On the other hand, the number of standard deviation units that a sample mean falls outside the specification limits is represented by a negative Q value (Hughes 2006). In theory, the use of the PWL method assumes that the population being sampled is normally distributed. In practice, it has been found that statistical estimates of quality are reasonably accurate, provided the sampled population is at least approximately normal, i.e., reasonably bell shaped and not bimodal or highly skewed (Burati et al. 2003).

To understand the parameters of the PWL and the main concerns when this method is applied, the calculation procedure is presented. To calculate the PWL, the following procedure is used, as outlined in NCHRP report 451 (Anderson and Russell 2001):

1. The approved JMF and the upper and lower tolerances should be known.
2. Calculate the LSL and the USL using the following equations:

$$\text{LSL} = \text{JMF} - \text{Lower Tolerance} \tag{1}$$

$$\text{USL} = \text{JMF} + \text{Upper Tolerance} \tag{2}$$

3. The lower and upper quality indices Q_l and Q_u are calculated using the following equations:

$$Q_l = \frac{\bar{x} - LSL}{s} \tag{3}$$

$$Q_u = \frac{USL - \bar{x}}{s} \quad (4)$$

where

Q_l = Lower quality index
Q_u = Upper quality index
$\bar{x}$ = Lot average
s = Lot standard deviation

4. The lower percent within limits (LPWL) and the upper percent within limits (UPWL) are then determined from a PWL table using Q_l for LPWL and Q_u for UPWL together with the n value, which represents the sample size.
5. For each quality parameter, the total percent within limits (TPWL) is calculated using the following equation:

$$\text{TPWL} = (\text{LPWL}+\text{UPWL}) - 100 \quad (5)$$

6. TPWL is calculated for each parameter and each lot to obtain the value for a parameter; the average of the TPWL is taken.
7. To determine the PWL for the whole project, a weighted average is taken for the TPWL of each quality parameter according to the weight given to this parameter.

ADVANTAGES AND DISADVANTAGES OF PERCENT WITHIN LIMITS

The reason that many agencies use PWL specifications can be related to the associated advantages that were presented by Hughes (2006):

- The best combined estimate for the population parameters is provided by PWL.
- PWL as a quality measure is sensitive to variability. This sensitivity gives an advantage to contractors with lower variability in their production.
- The method offers control over production for the contractor. A contractor with low variability can work near the specification limits. A contractor with high variability has to work near the job mix formula to avoid producing materials with quality characteristics outside the limit.
- When PWL is applied, a varying sample size is accounted for in the estimate of quality, which is not the case in the average method.
- Both a contractor and an agency can calculate their risks using PWL.
- Use of PWL is compatible with the American Association of State Highway and Transportation Officials (AASHTO) because it is presented as the “featured method” in R9.
- FHWA technical advisory T6120.3 presents PWL as the “used quality measure.”

With an understanding of the main steps in calculating PWL and knowing the advantages of the method, the following material introduces some of the challenges that may be faced when applying PWL analysis. For example, when the samples are analyzed, the results depend on the mean and standard deviation of the samples. As a result of this, a test may be performed in which all the samples satisfy the requirements, but the statistical analysis may show the percent within limits to be less than 100%. In this case, AASHTO R9 recommends reevaluating the results before applying penalties (AASHTO 2005). The pay factor is applied when the PWL is between two specified levels. If the PWL is below the lower level, the contractor is required to remove and replace the subject lot. The dependence or independence of the selected evaluation items affects the results tremendously. That is, if the factors are dependent, the probability of their compliance or noncompliance moves together. Alternatively, if the factors are independent, the probability of having one of them not complying is high (Newcomb and Epps 2001).

In an ideal world, the results achieved through testing samples with the same properties should be equal, but in practice they are not. This can be shown clearly in the multilaboratory and multiuser variability, even if the samples are split between the parties. This variation causes uncertainty in the results and causes a risk of rejection when PWL is applied (Manik 2006). The use of PWL imposes uncertainties on all project parties that would not have occurred had the project been evaluated for compliance (e.g., average quality characteristics) without using the PWL approach.

PWL can be misleading because it rewards uniformity. The method does not distinguish between uniformity around a desirable target and uniformity around the threshold of unacceptable properties. This leads to the presence of two types of PWL procedure application. These two types were presented by Mahboub and Hancher (2002), who identified these two types as “blind PWL” and “smart PWL.” Blind PWL is when the PWL procedure is applied without judgment; in this case, the application of PWL procedures may be much too penalty-oriented. Smart PWL is when judgment is involved; this method should replace the blind PWL. Smart PWL is the method that provides for moving the process toward desirable targets. Mahboub and Hancher (2002) presented a two-step process to avoid the problems of PWL:

1. A sublot that does not conform to the specification should be rejected. If this sublot has a high PWL and is not rejected based on this, a second-order adjustment is needed. The severity of the specifications’ violation must be accounted for in this second-order adjustment.
2. A second adjustment should be applied to the results achieved from applying PWL to provide an incentive to the contractors to perform close to the target, not to the specification limits. This adjustment should be applied in case all sublot values are within the specification limits. The adjustment should be made to account for the closeness of the sublot values to a desirable target within the specification limits. This will facilitate contractors making changes in their processes to move away from a uniform but marginally acceptable material to a more desirable material.

One of the problems of PWL is that the number of samples can be infinite, which will lead to a value for the percentage within limits and consequently the percent defective. In this case, it is not clear what is the defect and what are the consequences from this defect. For example, in the case of asphalt content, the percent defective does not show whether it is higher or lower asphalt content, although each case will result in a different pavement performance because too much asphalt content leads to bleeding and loss of skid resistance and too little asphalt content leads to early deterioration (Kopac 2005). It can be concluded that the use of PWL is good as a statistical method, but it does not correlate strongly with performance. This causes problems for highway agencies that seek to have a pay factor related to quality represented in the expected pavement performance (Kopac 2005).

One of the main factors that affects the results from PWL analysis is the calculation and rounding procedure used. When the pay factor is calculated based on PWL, this issue can be a point of conflict. This is because the values from the tables can be rounded up, rounded down, or interpolated. A contractor usually prefers the rounding up because it increases the PWL achieved. The method of rounding must be specified prior to implementation to ensure that there are no conflicts regarding this issue. As a result, it is important for an agency to stipulate the calculation process, including number of decimal places to be carried in the calculations, as well as the exact manner in which the PWL table is to be used (Burati et al. 2003).

There are three cases that can occur when a PWL is applied. The first case is when the percent within limits is very high; in this case, the contractor is rewarded. The second case is when the percentage within limits is low, but higher than a certain value; in this case, the contractor is penalized. The last case is when the percentage within limits falls below the lower specified limit; in this case, the contractor has to repeat the work (Pavement Digest 2006).

CASE STUDY

Percent within limits simulation was conducted on data provided by Iowa contractors to examine the implications of its implementation. It is important to understand that these simulations represent project data that includes multiple lots, because multiple test values of certain quality characteristics were not measured for each lot, as this is not required for the existing specification. The quality characteristics examined were laboratory voids based on field mix compacted in the laboratory (laboratory voids), binder content (P_{be}), percent passing the 0.075mm sieve, and the field voids based on field cores. The following criteria for the quality characteristics were used in conducting the percent within limit simulation utilizing Iowa contractor data:

- Laboratory voids: Design +/- 1% (3%–5%),
- Effective binder content (P_{be}): Design +/- 0.3%,
- Percent passing the 0.075 sieve (P200): Design +/- 2%, and
- Field voids: 4%–8%.

The calculation of the lower and upper quality indices was done according to the Oregon Department of Transportation (ODOT) specification, which follows the NCHRB 451 methodology presented earlier. Tables presented by ODOT (2006) were used to calculate the PWL and pay factors from the quality indices. The project pay factor was calculated to be the average of the pay factors of lab voids, binder content, percentage passing sieve number 200 (P200), and field voids, as is done in Oregon. Other PWL specifications have various weightings of the quality characteristic pay factors, i.e., they are not necessarily equal. Table 1 summarizes the outcomes of the PWL simulations.

Table 1. PWL Calcultion for Iowa projects using ODOT specifications

Project	Characteristic	Mean	St. dev	n	Q_L	P_L	Q_U	P_U	P_T	PF	Average PF
1	Lab Voids	3.30	0.58	10	2.93	100	0.52	69	69	0.91	1.0000
	Binder Content	5.11	0.10	10	4.40	100	1.60	95	95	1.04	
	P200	3.66	0.21	10	8.28	100	10.76	100	100	1.05	
	Field Voids	6.81	1.13	9	1.05	85	2.49	100	85	1.01	
2	Lab Voids	3.86	0.40	24	2.85	100	2.15	99	99	1.05	0.9700
	Binder Content	4.82	0.10	24	5.70	100	0.30	62	62	0.81	
	P200	3.52	0.31	24	5.10	100	7.81	100	100	1.05	
	Field Voids	5.93	0.36	24	5.75	100	5.36	100	100	1.05	
3	Lab Voids	4.18	0.32	20	2.56	99	3.69	100	99	1.05	1.0417
	Binder Content	4.93	0.10	20	4.60	100	1.40	92	92	1.025	
	P200	4.75	0.15	20	9.67	100	17.00	100	100	1.05	
	Field Voids	6.90	0.64	20	1.72	96	4.53	100	96	1.04	
4	Lab Voids	3.10	0.40	11	4.75	100	0.25	59	59	0.83	0.9767
	Binder Content	5.48	0.07	11	3.57	100	5.00	100	100	1.05	
	P200	4.60	0.12	5	14.17	100	19.17	100	100	1.05	
	Field Voids	5.95	0.35	10	5.86	100	5.57	100	100	1.05	

Table 1. Continued

Project	Characteristic	Mean	St. dev	n	QL	PL	QU	PU	PT	PF	Average PF
5	Lab Voids	4.25	0.52	17	1.44	93	2.40	99	92	1.03	1.0433
	Binder Content	5.65	0.13	5	1.77	100	2.85	100	100	1.05	
	P200	4.87	0.08	5	11.62	100	38.38	100	100	1.05	
	Field Voids	6.82	0.47	5	2.51	100	6.00	100	100	1.05	
6	Lab Voids	3.90	0.45	45	2.44	99	2.00	98	97	1.04	1.0200
	Binder Content	5.58	0.23	14	1.04	85	1.56	95	80	0.97	
	P200	5.66	0.30	14	2.47	99	10.87	100	99	1.05	
	Field Voids	7.57	0.34	7	1.26	90	10.50	100	90	1.03	
7	Lab Voids	4.18	0.38	39	2.16	99	3.10	100	99	1.05	1.0100
	Binder Content	4.68	0.11	13	4.08	100	0.64	74	74	0.93	
	P200	5.15	0.42	13	2.74	100	6.78	100	100	1.05	
	Field Voids	7.02	0.91	13	1.08	86	3.32	100	86	1.005	
8	Lab Voids	3.89	0.52	40	2.13	98	1.71	96	94	1.025	Reject
	Binder Content	4.54	0.13	14	6.23	100	-1.62	50	50	**Reject**	
	P200	5.02	0.34	21	4.06	100	7.70	100	100	1.05	
	Field Voids	7.36	1.02	14	0.63	73	3.29	100	73	0.925	
9	Lab Voids	4.04	0.75	20	1.28	90	1.39	92	82	0.965	1.0100
	Binder Content	4.47	0.20	8	1.15	88	1.85	98	86	1.015	
	P200	4.66	0.63	10	2.76	100	3.59	100	100	1.05	
	Field Voids	6.74	0.90	8	1.40	93	3.04	100	93	1.04	
10	Lab Voids	3.50	0.50	52	1.00	85	3.00	100	85	0.97	1.0100
	Binder Content	5.04	0.09	19	5.44	100	1.22	89	89	1.01	
	P200	3.64	0.35	22	7.26	100	4.17	100	100	1.05	
	Field Voids	-	-	-	-	-	-	-	-	-	
11	Lab Voids	3.56	0.57	42	0.98	84	2.53	100	84	0.96	0.9900
	Binder Content	4.99	0.13	14	3.54	100	1.08	86	86	1	
	P200	4.88	0.38	17	7.58	100	2.95	100	100	1.05	
	Field Voids	6.54	1.29	91	1.97	98	1.13	87	85	0.95	
12	Lab Voids	4.07	0.50	16	0.36	64	0.84	80	44	**Reject**	Reject
	Binder Content	3.93	0.12	5	7.75	100	8.92	100	100	1.05	
	P200	3.57	0.39	7	6.33	100	3.92	100	100	1.05	
	Field Voids	6.73	0.79	28	3.46	100	1.61	95	95	1.03	

The data were also analyzed using the methodology utilizing quality characteristics and their acceptable ranges currently implemented by the Iowa Department of Transportation (Iowa DOT) to calculate the pay factor (Iowa DOT 2006). In this methodology, the quality index was calculated for the density using equation 6, and the pay factor for density, gradation, P200, binder content, and film thickness was determined from tables in the Iowa DOT specifications. The results of the analysis are presented in Table 2.

$$QI_{Density} = \frac{(Average\_G_{mb})_{FieldLot} - ((\%Density)_{specified} \times (Average\_G_{mb})_{LabLot}}{(S\tan dard\_Deviation\_G_{mb})_{Field\_Lot}} \qquad (6)$$

Table 2. Pay factor calculations using Iowa DOT methodology

	QI (Density)	Deductions: Density	Film Thickness	Gradation (without P200)	P(200)	Asphalt Content	Total	Pay Factor
1	0.0727	15	0	0	2	0	17	0.83
2	0.3152	15	0	0	2	0	17	0.83
3	0.1509	15	0	0	2	0	17	0.83
4	0.2750	15	0	0	2	0	17	0.83
5	0.4914	5	0	0	6	0	11	0.89
6	0.3799	15	0	0	6	0	21	0.79
7	0.1553	15	0	8	2	0	25	0.75
8	0.0814	15	0	8	6	12	41	0.59
9	0.1145	15	0	8	6	0	29	0.71
10	-	-	0	0	2	0	2	0.98
11	0.0670	15	0	0	4	0	19	0.81
12	0.1264	15	0	4	2	0	21	0.79

It is important to understand that the pay factors developed here are based upon project-level data and not based upon lot data, as the comparison for project-level comparisons can be made, e.g., multiple test results for all the quality characteristics do not exist in the current specifications for use in PWL specifications. The results from both analysis methods can now be compared, since the same projects were used for both of them. The following can be can be concluded from the comparison:

- Iowa DOT specifications do not include a bonus opportunity, which is available in the ODOT specifications.
- Pay factors calculated using the Iowa DOT specifications are lower than those calculated using ODOT specifications, and the difference cannot be attributed to the bonus opportunity, as this is more than the 5% bonus opportunity.
- PWL methodology identified two projects that should be rejected, while in the Iowa DOT methodology the pay factor was reduced from full pay.
- If the Iowa DOT implemented the PWL methodology, a set of equations or tables needs to be developed to calculate the pay factor based on the PWL.

CONCLUSIONS

The advantage of the use of the PWL is that it combines two important statistical measures, mean and standard deviation, in one parameter (Burati et al. 2003). Combining these two parameters can lead to some ambiguity in the results. Some highway agencies use the mean and standard deviation directly to calculate the pay factor. These agencies have charts relating the mean and standard deviation values to contractor payment. The use of this method eliminates the ambiguity caused by using PWL, which hides some of the population properties when the results are blended together (Kopac 2005). It is important when applying PWL to understand the statistical background behind the method and to apply judgment to the results to be able to obtain the optimum results. It is also important that the specifications be clear and that every issue is addressed in detail, e.g., rounding. Adding incentives to having results towards the desired value, not just within the required range, is important because PWL, as previously mentioned, rewards uniformity within any point of the specified range.

The methodology used by the Iowa DOT to calculate pay factor leads to lower pay factors than the methodology implemented by ODOT. It is important to note that, although two projects were acceptable using the Iowa DOT methodology, these projects were rejected using the ODOT methodology. For Iowa to implement the PWL methodology, a new calculation method for the pay factor should be adopted, and a decision should be made whether to incorporate an incentive or not.

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Modification of Driver Behavior Based on Information from Pedestrian Countdown Timers

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ABSTRACT

Pedestrian countdown timers (CDTs) are promoted as a means of improving pedestrian safety at intersections. However, there are concerns that drivers view the timers while approaching the intersection and use the information to drive more aggressively, an unintended consequence that may have adverse safety impacts. Pedestrian CDTs have been in widespread use in Lawrence, Kansas for three years, and so any novelty effect should have passed, allowing for an accurate analysis of the long-term effects of the devices on traffic.

Four intersections along an arterial corridor in Lawrence were studied, two with CDTs and two without. Continuous speed data were collected on approaching traffic and analyzed to determine if there were changes in speed between 400 ft. upstream from the intersection (the point when the CDT information could be read by drivers) and the intersection stop bar. Additionally, the ultimate decision of the driver (whether he/she stopped or not) was recorded. Analysis revealed that drivers were significantly less likely to increase their speed in order to reach the intersection before the beginning of the red phase when CDTs were present, and some drivers began to slow to a stop *before* the beginning of the amber phase. This indicates that drivers use the information provided from pedestrian CDTs to improve their driving decisions; even though the CDT information was not intended to be used by drivers, it appears that they are indeed doing so in a way that results in improved driving actions.

Key words: driver compliance—pedestrian countdown timers

INTRODUCTION

Pedestrian countdown timers (CDTs) continue to spread across the United States as their safety benefits for pedestrians become better understood. In addition to the traditional use of the symbolic or text-based WALK/DON'T WALK information, pedestrian CDTs provide pedestrians with a descending numerical countdown that reaches zero at the onset of the amber phase of the traffic signal. A typical pedestrian countdown display is shown in Figure 1. This provides pedestrians with information about whether they have enough time to safely make their crossing, allowing pedestrians to make more informed decisions.

Figure 1. Typical pedestrian countdown display

Previous research by the Minnesota Department of Transportation (Mn/DOT) found that the proportion of pedestrians which were successfully able to cross five urban intersections in appropriate times increased from 67 to 75% after the installation of CDTs (Farraher Undated). Additionally, when interviewed, an overwhelming majority (92%) of those pedestrians found the CDTs helpful in making their crossing decisions. Indeed, there is strong evidence that providing pedestrians with any kind of additional information beyond the traditional WALK/DON'T WALK indication can have a beneficial change in safety (Zegeer, Cynecki, and Opiela 1984). Other researchers have been able to show a significant reduction in pedestrian-involved crashes at intersections where CDTs have been installed (Markowitz et al. 2006).

However, because drivers can also see these indications, there are concerns that driver behavior may also change in a way that degrades safety (Streetsblog 2006; Metropolitan Transportation Commission 2007). While there have been many studies that examine how pedestrian actions are changed with the installation of CDTs, only a few studies have explored how these devices might change the behavior of drivers passing through the intersection. One study found that drivers at one intersection with CDTs were less likely to enter an intersection at the end of the amber phase than those at another nearby intersection without the CDTs (Huey and Ragland 2007). Furthermore, it was found that drivers exhibited different stopping behavior at the two intersections, which could indicate different braking habits exhibited by drivers based on the type of information available to them. In another study that examined driver behavior, red light running was reduced from 2% to 1%, but the authors conjectured that this was due to drivers speeding up because of the CDTs and avoiding the red phase altogether (Markowitz et al. 2006). An increase in aggressive driving behavior would be an unintended and undesirable consequence of the presence of pedestrian CDT installations, and a better understanding of how drivers react to them is needed.

RESEARCH OBJECTIVES

There are two concerns regarding how drivers may change their behavior when exposed to pedestrian CDTs: first, that they may increase their speed in order to avoid having to stop for a red phase, and second, that when stopped at an intersection they may use the CDTs as a way to depart early just prior to the beginning of the green phase. This research examined the first case in order to determine if drivers' approach speeds are affected by the presence of CDTs. Of particular interest were changes in the speed profile of vehicles from the point upstream where the CDTs were just readable until vehicles passed the intersection stop bar.

RESEARCH METHODOLOGY

Pedestrian CDTs have been in widespread use in Lawrence, Kansas for over three years so any novelty effect should have passed, allowing for an accurate analysis of the long-term effects of the devices on traffic. Four intersections along 23rd Street in eastern Lawrence were observed for this research. Details about the intersections can be seen in Figure 2 and Table 1. These intersections were selected as they have similar geometric layouts, are on the same arterial corridor, and a high proportion of daily traffic passes through all four locations. Two of the intersections are equipped with CDTs, and two are not.

Continuous speed data were collected on approaching traffic using LIDAR speed collection equipment linked to a laptop equipped with special acquisition software. Data collection was performed in both the eastbound and westbound directions of each intersection. The data collection team was positioned downstream of the intersections facing oncoming traffic. In all but two locations (Ousdahl Rd. westbound and Alabama St. westbound), the data collection was done from behind objects near the sidewalk in order to collect speed data inconspicuously. At both Ousdahl Rd. and Alabama St. westbound, there were no desirable objects from which to hide the data collection team, and so at these locations the data collection was done from a location as far downstream as possible to minimize the data collectors' visibility to vehicles approaching these intersections.

Only vehicles that were judged to be in or near an intersection's indecision zone were targeted for data collection. That is, the research team was only interested in a vehicle if its driver would have to make a decision on whether to stop or go as the intersection's amber phase began and if the surrounding traffic would allow the driver to adjust his/her speed if desired. No data were collected when the pedestrian warning was not activated (don't walk hand or countdown activation). As a result of these data collection requirements, many cycles were observed where no useable vehicles were present.

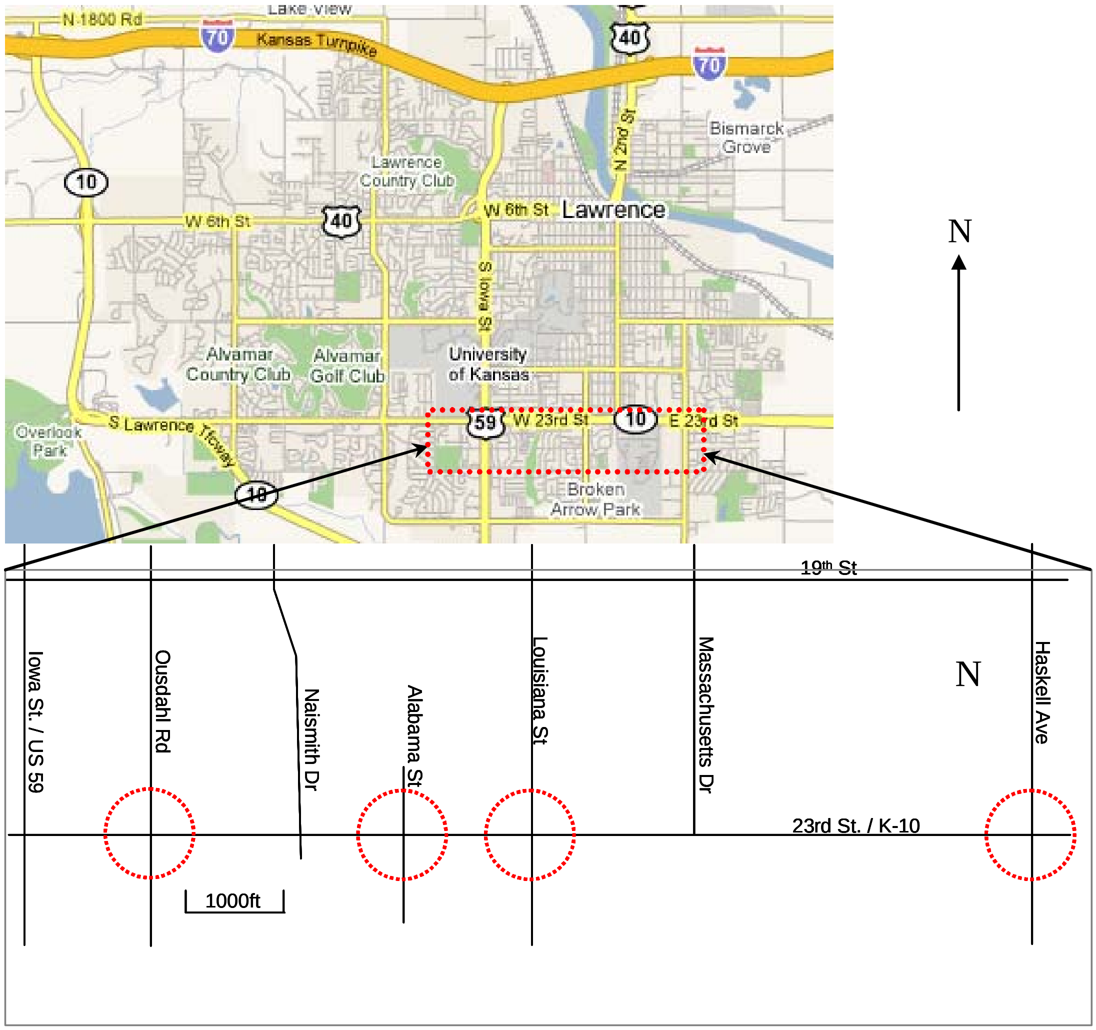


Figure 2. Location of four intersections along K-10 corridor in Lawrence, Kansas

Table 1. Properties of the four study intersections in Lawrence, Kansas

Minor Street Intersecting with 23rd Street	Ousdahl Road	Alabama Street	Louisiana Street	Haskell Avenue
CDT	No	Yes	No	Yes
Amber phase (sec)	4	4	4	4.5
Pedestrian warning time (sec)	15	15	23	18
Major approach lanes (23rd Street)	2 through 1 left	2 through 1 left	2 through 1 left	2 through 1 left
Minor approach lanes	1 through 1 left	1 through 1 left	2 through 1 left	2 through 1 left
Approach speed limit (mph)	35	35	35	45

In cases where the intersection had a CDT, the time on the CDT was noted at the instant the speed data collection began for each vehicle. At intersections without a CDT, data were taken when the flashing hand was present. For each vehicle, the position of the vehicle at the beginning of the amber phase and/or the red phase was determined. From this, it was possible to backcalculate the time displayed on the CDT when the vehicle was 400 ft. from the intersection. Data were collected on 11 clear, dry mid-week days in March through June 2007. Data were only collected at off-peak morning or afternoon time periods. In the observation of an estimated 600 cycles, 207 useable data points were generated.

Statistical Study Design

The CDTs each had nominal 8 in. character height, which conservatively correlates to a 400 ft. reading distance for drivers with normal vision, assuming 50 ft. of reading distance for every inch of letter height (Schwartz 1999). So then it could be reasoned that if a driver was likely to change his/her speed based on the information provided by the CDTs, it would happen at some point within 400 ft. upstream from the intersection. The data were analyzed to determine if there were more changes in speed during this range when CDTs were present.

For each vehicle, data were also collected on the ultimate decision the driver made (stop or go) and the status of the pedestrian display when the vehicle was 400 ft. upstream. Each vehicle was categorized based on whether the intersection that it passed through had a CDT or not and the action taken by the driver. The driver actions were subdivided into five categories:

- stopped (began decelerating at or after the beginning of the amber phase)
- stopped but began decelerating early (*before* the beginning of the amber phase)
- continued on normally through the intersection
- continued on through the intersection but accelerated in order to do so
- continued on through the intersection but ran the red light in order to do so

There were no observed instances of red light running, so this driver action category was removed from later analysis. In order to determine whether these distributions were different based on the presence or absence of CDTs, a chi square analysis was conducted to test the following hypotheses:

H_0: There is no difference in driver actions based on the presence or absence of CDTs.

H_A: There is a difference in driver actions based on the presence or absence of CDTs.

KEY FINDINGS

The results were examined in two ways. First, an examination was conducted on how driver decisions (stop or go) related to the CDT information that was displayed when the vehicles were 400 ft. from the intersection. Second, a statistical analysis was performed to determine if the distribution of driver decisions changed based on the presence or absence of CDTs.

Examination of Driver Decisions Based on CDT Displays

Speed profiles were collected on 207 vehicles as they approached the four intersections of interest. An example of two typical vehicles, one that stopped at an intersection and one that did not, is shown in Figure 3.

For each observed vehicle the time remaining until the beginning of the amber phase when the vehicle was 400 ft. from the intersection was determined. Note that for the CDT intersections this corresponded directly to the CDT display. For the intersections that did not have CDTs the time was determined based on the elapsed time since the beginning of the flashing hand on the pedestrian signal displays. These data were graphed against the expected time it would have taken the vehicle to reach the stop bar assuming no changes in speed. Of the 207 speed profiles that were recorded, only 128 were useable in this analysis; for during the data collection it was not always possible to acquire a vehicle's speed profile 400 ft. upstream the intersection. This was a result discussed above about the need to collect data from far downstream from the intersections.

The results of this examination are shown in Figure 4. As shown in the figure, a driver's decision on whether to stop or go as they approached the intersection was based on his/her speed and the CDT information that they had available to them. The diagonal lines indicate how fast a vehicle would have to travel from 400 ft. upstream in order to just reach the intersection at the beginning of the amber and red phases, respectively. For example, if a driver that reached the point 400 ft. upstream from the intersection required eight seconds to reach the intersection at the exact point in time when eight seconds remained before the onset of the amber phase, the data point would fall exactly on the line representing the beginning of the amber phase.

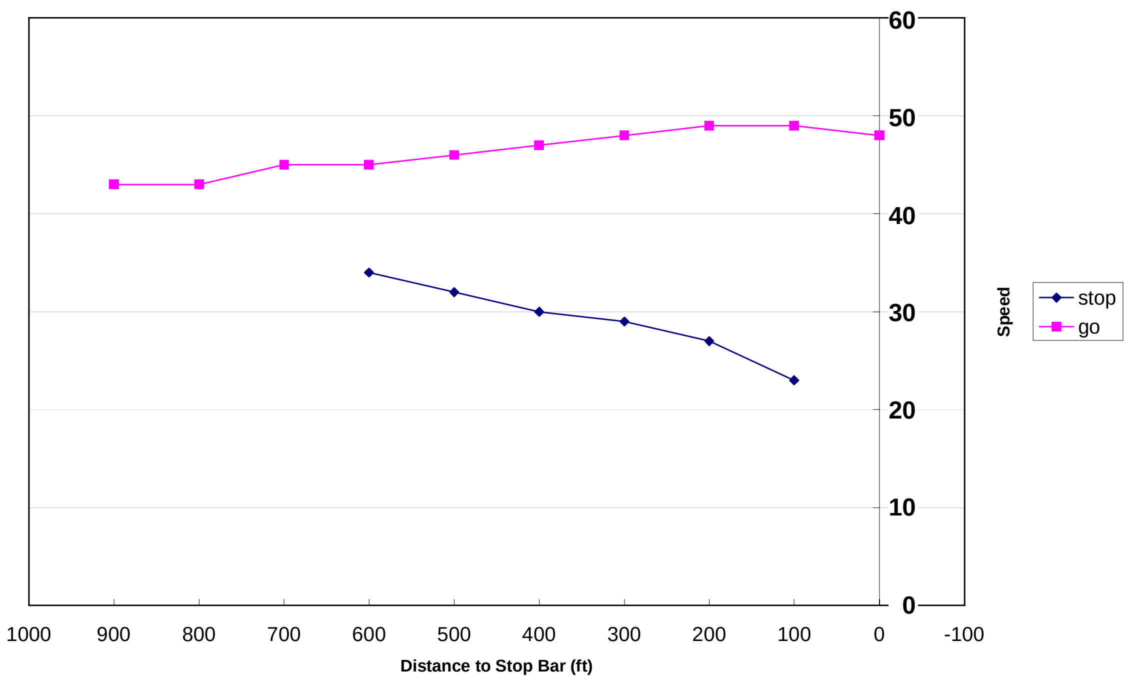
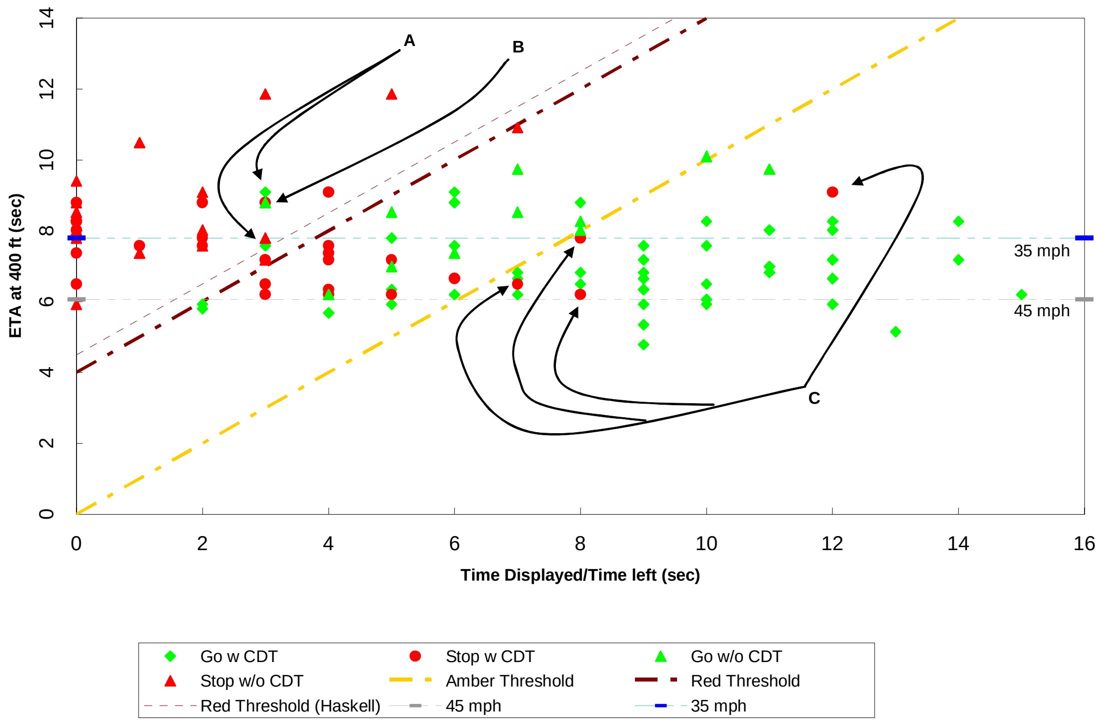


Figure 3. Two typical speed-distance profiles for observed vehicles



A: Driver increased speed in order to clear the intersection prior to the beginning of the red phase, CDT present.
B: Driver increased speed in order to clear the intersection prior to the beginning of the red phase, no CDT present.
C: Driver decreased speed and stopped when it would have been possible to clear the intersection at initial speed, CDT present

Figure 4. Driver decisions compared to speed and CDT display

Data points in Figure 4 that are far to the left of the line, indicating the beginning of the red phase, correspond to vehicles that arrived 400 ft. upstream from the intersection when only a few seconds remained before the beginning of the amber or red phases, and these vehicles did not have enough time to reach the intersection. Not surprisingly, the drivers of these vehicles decided to stop at the intersections. Likewise, data points far to the right of the line, representing the beginning of the amber phase, corresponded to vehicles that arrived 400 ft. upstream from the intersection with ample time to reach the intersection, and no speed changes were required in order to reach the intersection during the green phase. As expected, the drivers of these vehicles decided to proceed through the intersection.

Of interest are the vehicles that fall between and to either side of the diagonal lines representing the beginning of the amber and red phases; these drivers had opportunity to change their speeds as they approached the intersections. If a driver needed to increase his/her speed to avoid stopping at the intersection, the data would show as a “go” decision to the left of the diagonal line indicating the beginning of the red phase. This atypical result also could have occurred due to red light running, but because no instances of this were observed during the data collection period, this indicated that these drivers increased their speed in the last 400 ft. in order to avoid the red phase. As shown in the figure, there were two instances of this occurring at intersections with CDTs, and one at intersections without CDTs.

The opposite of this was an atypical “stop” decision to the right of the line representing the beginning of the amber phase, indicating that a driver stopped when he/she could have proceeded at a constant speed and legally cleared the intersection. As shown in the figure, there were four instances of this occurring at intersections with CDTs, and zero instances at intersections without CDTs.

Statistical Analysis of Driver Decisions

A comparison of the distributions of driver decisions based on the presence or absence of CDTs is shown in Table 2. A chi square test ($\alpha = 0.05$, df = 3) was performed to determine if the driver decision distributions were significantly different. It was determined that there was a statistically significant difference between the driver decision distributions (p-value = 0.0038).

Table 2. Cross classification of driver action by the presence of pedestrian CDTs

	Driver Action				
	Unchanged Stop	**Stop, Early Deceleration**	**Unchanged Go**	**Accelerated During Go**	**Total**
Pedestrian CDT	17[A] (18.98)[B]	12 (8.86)	48 (43.66)	4 (9.49)	81
No Pedestrian CDT	13 (11.02)	2 (5.14)	21 (25.34)	11 (5.51)	47
Total	30	14	69	15	128

[A] The top number in each cell is the observed value.
[B] The bottom number each cell is the expected value based on total observations.

A cell-by-cell comparison of adjusted residuals between the observed and expected frequencies was also conducted in order to better understand the nature of the data. This was done in order to determine which parts of the distributions were significantly different from the expected values; any adjusted residuals with an absolute value of about two or three indicated a significant difference between the observed and expected observations for the given cell (Agresti 1996). As can be seen in Table 3, the adjusted residuals for the number of vehicles that accelerated in the last 400 ft. upstream from the intersection were found to be significant. Therefore, it can be concluded significantly fewer instances of drivers accelerating in the last 400 ft. upstream from the intersection were observed when CDTs were present. Also, a weaker association exists where drivers were more likely to begin to decelerate before the beginning of the amber phase when pedestrian CDTs were present.

Table 3. Adjusted residuals for testing independence

	Driver Action			
	Unchanged Stop	**Stop, Early Deceleration**	**Unchanged Go**	**Accelerated During Go**
Pedestrian CDT	-0.86	1.84	1.60	-3.13[A]
No Pedestrian CDT	0.86	-1.84	-1.60	3.13[A]

[A] A strong indication that the observed values in these cells are significantly different from the expected values.

CONCLUSIONS

Drivers were found to drive less aggressively at intersections equipped with CDTs compared to similar nearby intersections on the same corridor that did not have CDTs. The proportion of drivers that increased their speed on the intersection approach was significantly less at the intersections with CDTs.

Additionally, drivers who stopped at the intersections were more likely to begin decelerating even before the beginning of the amber phase. These two findings seem to dispel the notion that drivers use CDTs to drive more aggressively. Rather, these findings indicate that drivers use the information presented on the CDTs to make better decisions about their ability to reach the intersections prior to the beginning of the red phase, and they are less likely to drive aggressively as a result.

The data analyzed in this research indicate that drivers use the information provided from pedestrian CDTs to improve their driving decisions; even though the CDT information was not intended to be used by drivers, it appears that they are indeed doing so in a way that results in improved driving actions. Additional research is needed to determine if these results can be replicated at other locations and under other conditions. Additionally, an important safety question not addressed from this research is how drivers might use CDT information when departing from an intersection at the beginning of the green phase.

ACKNOWLEDGMENTS

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A Full Bayesian Assessment of the Effects of Highway Bypasses on Crashes and Crash Rates

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ABSTRACT

A common contention is that the construction of highway bypasses negatively impacts the economy of local communities by reducing pass-by traffic for businesses. However, as access to specific business account records is limited, this is difficult to quantify. Another common contention is that a reduction in crashes will be achieved. The actual impact of highway bypasses in the United States has not been assessed.

This study seeks answers to the following questions with the use of a full Bayesian analysis:

- Do bypasses in Iowa affect crash frequencies and rates on the bypassed highway?
- Do bypasses in Iowa introduce a reduction of overall crash frequencies and rates, or do they merely shift crashes from the highways through the communities to the bypasses with no significant overall reduction?

The results strongly suggest that the construction of bypasses in Iowa increases traffic safety both on the main road through town and on the sum of the main road and the bypass

Key words: Bayesian—crashes—crash rates—highway bypasses—Iowa

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1. INTRODUCTION

Over the past decades, several highway bypasses have been constructed throughout Iowa. Building bypasses affects a variety of elements such as traffic crashes, urban traffic densities via redirection of through traffic, and control of environmental pollution (Elvik and Vaa 2004).

Highway bypasses around rural communities in heavily traveled transportation corridors are perceived as a highly cost-effective method of improving traffic flow along non-interstate transportation routes. However, the bypassing of a central business district raises concerns among merchants and residents over possible adverse impacts on their businesses (Leden et al. 2006).

The common contention that highway bypasses negatively affect the economy of local communities by reducing pass-by traffic for businesses has been widely investigated (Andersen et al. 1993; Blackburn and Clay 1991; Buress, 1996; Hartgen 1991; WisDOT 1998; Srinivasan and Kockelman 2002). Analyzing the impacts of highway bypasses on the economy of small communities is limited by the lack of community level data for areas with population of less than 2,500.

The application of standard empirical techniques to assess the effect on local economies is limited but can be overcome by using local retail sales tax information. However, the conclusions of such analyses are only supported with site visits, surveys, and quasi-experimental results (Rogers and Marshment 2000). Furthermore, surveys of opinions held by residents of the bypassed areas regarding the bypass impact are limited and largely anecdotal and not amenable to statistical analysis (Sabol 1996).

The main aim of extending and improving road systems is to increase mobility and reduce transport costs. The effects of road design and equipment on accidents varies from one measure to another. However, highway bypasses are reported as measures that reduce the number of accidents. (Elvik and Vaa 2004)

Road planners perceive the construction of a highway bypass as a solution for heavy traffic load through towns or business districts (Srinivasan and Kockelman 2002). The expected outcome of redirecting traffic is a decrease of the negative effects of congested traffic, including frequency of accidents. Since crash data in Iowa are available for several years before and after such interventions, with the use of statistical tools it is possible to quantify the effects on traffic safety of constructing highway bypasses.

2. LITERATURE REVIEW

Forkenbrock et al. (1990) suggest that highway traffic passing through small rural towns is often slowed by congestion, traffic control devices, and poor geometry. They also suggest that rerouting long-distance commuters around small towns improves safety and reduces travel times.

The central business district of towns often assembles a mixture of pedestrians, cyclists, and motor vehicles. The accident rate in towns and cities, therefore, is usually higher than in rural areas. The high traffic volume in towns causes both environmental problems and an increased risk of accidents (Elvik and Vaa 2004).

Eagan et al. (2003) affirm that road construction and automobile dependency have been associated with community severity (i.e., the creation of a physical barrier running through the community that reduces access to local amenities and disrupts social networks), increased disturbance among residents, and social inequalities.

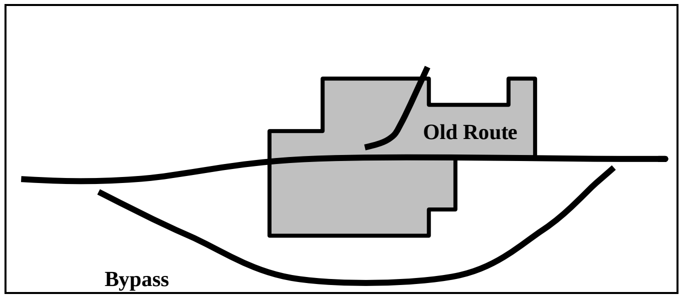

Figure 1. Typical highway bypass around a rural town

For much of the 20th century, transportation planners sought to improve transportation system efficiency by constructing bypasses (Sabol 1996). A highway bypass is a route that splits off and passes along the fringe of a town or city to circumvent all or most of the portions of the town or city that are developed and then ties back into the older route from which it has originated on the other side of town (Figure 1). The new route may run for a longer distance and incorporate more than one town. A highway improvement that redirects through traffic off an existing route to avoid the central business district can be considered a bypass (Sabol 1996).

When bypasses are built, the long-distance traffic is shifted outside towns and cities. This separation of local and long-distance traffic decreases the traffic volume on the main road, making it easier to introduce traffic calming measures (Elvik and Vaa 2004). Normally bypasses are built without direct access roads. The connection between bypasses and actual roads are made using high standard junctions or interchanges. Bypass roads are designed for a speed limit of at least 50 mph.

Sabol (1996) analyzed over 190 reports on the construction of bypasses in 47 states and 6 provinces in the United States. The following list summarizes the reasons for constructing bypasses cited in such reports:

- Relief of traffic congestion in the bypass community (54 sites)
- Rerouting of traffic (27 sites)
- Enhanced access to tourism resources or “downtown” (8 sites)
- Noise reduction (5 sites)
- Traffic safety improvement (4 sites)

According to Snyder and Associates, Inc. (1999), one of the reasons bypasses are built in Iowa is to concentrate a major portion of annual construction budget on the corridors between major businesses and industrial centers, also known as the commercial industrial network (CIN). In a study on highway bypasses Snyder and Associates (1999) report that:

- 64% of rural highway traffic is on the interstate and CIN system
- 82% of rural semi-truck traffic is on interstates or CIN (20% uses CIN)
- 80%+ of Iowans live within 10 miles of the interstate or CIN
- Interstate and the CIN account for only 32% of the total rural highway miles.

In many states, departments of transportation constantly evaluate the investment of resources on major road construction projects (Comer et al. 2000). Table 1 presents a list of nine safety improvement candidate locations where the construction of a bypass or some other form of relief route has been considered by the Iowa Department of Transportation (Iowa DOT).

Table 1. Safety improvement candidate locations in Iowa, 2006

Needs	Route	County	Location
New Route	30	Carroll	Carroll Bypass
New Route	71	Clay	Spencer Bypass
New Route	30	Crawford	Denison Bypass
New Route	63	Davis	Bloomfield Bypass to Floris Road
New Route	61	Des Moines	Burlington Bypass
New Route	71/86	Dickinson	From S. of Milford to IA. 9 (Bypass of Lakes Area)
New Route	30	Harrison	Missouri Valley Bypass to E. of Logan
New Route	30	Linn/Cedar	Mount Vernon/Lisbon Bypass
New Route	75	Plymouth	Hinton and Merrill Bypass

Horwood et al. (1965) analyzed the aggregate results of 45 of the 70 bypass studies from the early 1960s in the United States. Such summary of the state of bypass studies has not been duplicated nor supplanted by later research; however, the aggregated variables and results were used to derive general trends of economic impacts for those studies and not for traffic safety purposes (Comer 2000).

"From a systemwide perspective, bypasses around many of [the] rural trade centers can provide substantial benefits of reduced travel times, congestion, and accidents relative to costs" (Iowa DOT 1992, p. 34).

A meta-analysis of studies in various European countries addressing the effects of bypasses concludes that the construction of bypasses, on average, decreases the number of injury accidents by approximately 25% (Elvik and Vaa 2004). The percentage includes accidents on both the old road network and the bypass. In one of the studies on 20 bypasses in Norway, Elvik et al. (2001) observed a 19% decrease in injury accidents. The decrease was significant when a fixed effects model was used in the analysis, giving a 95% confidence interval from 5% to 30%. However, when a random effects model was employed, the decrease was rejected by significance testing, with a confidence interval between -35% and 0.4%.

According to Elvik and Vaa (2004), the effects of bypasses on the number of accidents can vary from place to place. Several factors have been found to affect accident rates. The first factor is the accident rate on the main road through town where the bypass is built. The higher the number of accidents on the main road through town, the larger the decrease in the number of accidents will be after the bypass is built. The proportion of traffic transferred to the bypass also influences the effect of the bypass. If a larger volume of traffic is shifted to the bypass, then a greater decrease in the number of accidents is to be expected. A greater decrease in the number of accidents can be attained if speed reducing measures are implemented on the main road through town that is replaced by the bypass. Finally, the design of junctions built between the old road and the bypass also influences the accident rate.

Elvik and Vaa (2004) also found that, on average, the accident rate on the old main roads increases, possibly because of the increase in speed due to the lower traffic volume. The speed limit increase on the newly constructed bypass road also raises a concern in accident severity; however, Elvik and Vaa (2004) report a Norwegian study (Amundsen and Hofset 2000) and a Danish study (Andersson et al. 2002) that assert that accident severity on the old main road did not change after a bypass road was opened to traffic.

The construction of a bypass has both mobility and environmental effects. Bypass roads increase the mobility for both long-distance and local traffic. The decrease of traffic will also make it easier for cyclists to cross roads in towns as long as the speed limits do not increase. Environmental effects include (Elvik and Vaa 2004):

- Reduced traffic noise, vibrations, air pollution, and barriers to local travelers due to the reduction in traffic volume
- Improved opportunity for introducing environmental measures in a town
- Fewer vehicle emissions due to lower traffic congestion

Eagan et al. (2003) summarized the results of studies assessing the effects of new roads on injury prevalence rates. Five of these studies addressed out-of-town bypasses with the use of before-and-after comparison of police injury reports. These bypass studies showed a general decline in the incidence of injury accidents after the opening of the new bypasses. Only two of the studies reported a statistically significant decline, and both studies were published in the 1960s (Leeming 1969; Newland and Newby 1962).

All of the bypass studies mentioned by Eagan et al. (2003) compared the incidence of injury accidents on main roads through town in the before period with the incidence of injury accidents on both old roads through town and new bypasses in the after period. The studies were all performed in European countries, two of which were in Denmark (Andersson et al. 2002; Jørgensen 1991), one in Norway (Elvik et al. 2001), and two in the United Kingdom (Leeming 1969; Newland and Newby 1962).

Eagan et al. (2003) conclude that out-of-town bypasses reduce injuries on the main road, but there is not enough evidence of the effects on secondary roads. Of the five bypass studies, only those performed in Denmark included adjacent secondary roads in the analysis of injury accidents, and each study detected statistically insignificant decreases.

Also in their review Eagan et al. (2003) found 12 bypass studies that revealed a general decrease in disturbance and community severance among residents of bypassed towns. The largest decrease in through traffic and thus in disturbance was generally experienced in smaller towns. The review shows that the disturbance level increased in the rural areas surrounding the newly constructed bypasses, thus the traffic noise was merely shifted from one place to another.

The most common methodology used for bypass studies are before-and-after analyses of previously bypassed towns and comparisons of bypassed towns to non-bypassed control towns (Comer et al. 2000). In order to obtain results that can be generalized, the sample biases need to be negligible, the before-and-after data need to be compared, the models used need to be sophisticated enough to extract meaningful information from the data and to account and control for other relevant factors, and control sites need to be included in the dataset. All cities receiving highway improvements could in fact have certain characteristics that differ from those cities not receiving such improvements (Srinivasan and Kockelman 2002).

Srinivasan and Kockelman (2002) performed a multivariate regression analysis to model the economic impacts of bypasses on small- and medium-sized communities. The study involved 23 cities in Texas, bypassed between 1965 and 1990, and 19 non-bypassed cities chosen as control cities. District traffic maps from the Texas Department of Transportation were used to infer the year when traffic first appeared on the bypasses, the year the bypasses opened, and the number of years since opening. The results of the study confirm the findings of the aforementioned review by Eagan et al. (2003) that smaller cities experience a higher decrease in traffic after the construction of a bypass. Another important finding was that relief routes that work better from a traffic standpoint have a greater impact on local per capita sales.

Otto and Anderson (1995) note a problem that emerges when performing before-and-after studies on effects of bypasses. Since the highway bypass construction period is about two to three years, when comparing the experimental data to that of control groups (i.e., similar cities that have populations, traffic volume, and distance from metropolitan areas comparable to the bypass cities), those years of construction add bias to the statistics used to measure the data. This lag leads to an overestimated sample mean. Such means would not represent an approximation of the population mean.

Pawlovich (2003) affirms that over the past ten years fatal and injury crash rates declined in Iowa. He also claims that it is difficult to isolate the cause of such reductions in crash rates due to the wide variety of safety efforts that have been implemented over time and that better methods for determining effective strategies need to be developed. Crash rate is defined as the ratio of crashes to Annual Average Daily Traffic (AADT), for the period of interest, normalized to 10^6 vehicles. Because of improvements in statistical computing techniques, Carriquiry and Pawlovich (2004) suggest that hierarchical Bayesian models can be used to analyze data from before-and-after studies and avoid the disadvantages of the standard regression techniques.

Pawlovich et al. (2006) used a hierarchical modeling approach on a problem similar to the one addressed in this work. They evaluated the effectiveness (in terms of improved safety) of a four-lane to three-lane conversion at various signalized intersections in Iowa. They found that the hierarchical modeling approach provided the flexibility to account for potentially relevant effects in safety beyond the potential effect of the intervention. An approach similar to the one used in Pawlovich et al. (2006), has been adopted in this work.

The paper is organized as follows: data details are described followed by a description of the exploratory analysis and modeling methodology. Analytical results are then presented and followed by interpretation of the results and a summary and conclusions.

3. DATA

This study evaluated several bypassed communities in the state of Iowa. The crash database was constructed by the Iowa DOT and was utilized to obtain traffic safety information over several years before and several years after the intervention (the construction of the bypass). The bypassed sites were compared to six cities that were scheduled to be bypassed but did not receive the intervention prior to 2005. The treated and untreated sites are distributed over 25 towns and cities in Iowa as presented in Table 2.

Most sites were observed over 24 years between 1982 and 2005, and three sites, Site 14, Site 17, and Site 18, were observed for 22 years between 1984 and 2005. Sites 1 to 19 are the bypassed sites and will be referred to as treatment or treated sites. Sites 20 to 25 were the reference sites and will be referred to as non-treatment or untreated sites throughout this work. Since several bypasses incorporated more than one town before tying back to the older route from which they originated, some of the sites include traffic safety information for a stretch of road along multiple towns (i.e., Sites 10, 16 and 17).

One concern in observational studies is sampling bias. If the sites chosen to receive a bypass had a worse safety record than those that did not receive a bypass, then the effect of the intervention on safety will be overestimated. Treated and untreated sites in this study were compared in terms of safety and other characteristics during the years preceding the intervention, and it was found that the untreated sites appear to be safer on average than the treated sites. Anticipating that the Iowa DOT allocates its resources where the perceived is greatest, this bias is expected.

Because of the potential selection bias it is necessary to interpret the results cautiously. Sites 20 to 25 have been used as reference sites in this study with the understanding that the estimated positive impacts of the bypass on safety (if any) will likely be overstated. Selection bias is difficult to overcome without the benefit of a designed experiment where sites are randomly allocated to the intervention and control group. The hope of this study is to ameliorate somewhat the effects of selection bias by considering crash information over a period of at least 21 years at each site.

Table 2. Information on study sites

Site	Site Name	Road Length	Bypass Length	Road + Bypass (Sum) Length	Completion Year	Treated
1	Sioux City	11.0	9.3	20.7	2001	Yes
2	Alton	5.2	6.8	10.1	2004	Yes
3	Storm Lake	10.9	6.4	17.0	1997	Yes
4	Pleasantville	4.5	4.2	7.4	2003	Yes
5	Prairie City	4.2	5.6	9.4	1998	Yes
6	Monroe	4.3	7.4	10.6	1999	Yes
7	Otley	1.9	1.9	3.1	1999	Yes
8	Pella	7.2	10.8	16.3	1995	Yes
9	Oskaloosa	5.7	7.8	13.4	1998	Yes
10	Swedesburg/Olds	5.1	5.3	9.7	1999	Yes
11	New London	5.7	6.8	11.0	2000	Yes
12	Blue Grass	3.1	5.9	8.4	2001	Yes
13	Marion	3.8	5.2	9.0	1998	Yes
14	Waverly	7.7	10.9	17.1	1999	Yes
15	Charles City	8.2	13.7	21.1	2000	Yes
16	Rudd/Mason City/Nora Springs	27.0	33.8	60.8	1999	Yes
17	Jesup/Raymond/Evansdale	23.8	27.3	47.9	1986	Yes
18	Denver	2.0	4.0	5.8	1996	Yes
19	Marshalltown	6.6	9.6	15.5	1998	Yes
20	Le Mars	5.9	N/A	N/A	N/A	No
21	Seney	2.7	N/A	N/A	N/A	No
22	Sheldon	8.6	N/A	N/A	N/A	No
23	Ashton	3.2	N/A	N/A	N/A	No
24	Eddyville	4.4	N/A	N/A	N/A	No
25	Hospers	4.3	N/A	N/A	N/A	No

The Iowa DOT provided data for each site covering the time period of the study. These data included site length (miles), crash frequency (crashes per month), traffic volume (average annual monthly traffic), vehicle miles traveled (monthly vehicle miles traveled), and monthly crash rate (monthly crash frequency per vehicle miles traveled). These data were provided separately for the through road, the bypass, and the sum of the two.

Using transportation maps from the Iowa DOT, in conjunction with plots of the monthly average daily traffic (MADT) values for each site, it was possible to infer the opening year of each bypass and the number of years since opening for each data year. Figure 2 shows a plot of the MADT for Site 1 on the main road through town. The vertical line indicates a drastic drop in traffic volume that coincided with the year of opening of the bypass around that community as indicated by the transportation maps. Figure 3 presents the MADT through town up to the opening of the bypass, indicated by the vertical line, and the MADT on the bypass after the intervention. A careful examination of the plots reveals a large increase in MADT at Sites 5 and 17 after the construction of the bypass. These increases are expected to affect the crash frequencies and the crash rates at the respective sites.

These plots also revealed a constant pattern at each site indicating peaks of traffic during the summer months and lower traffic volumes during the winter months. Such patterns suggested the need of seasonal effects to be accounted for when analyzing crash patterns, at least in Iowa.

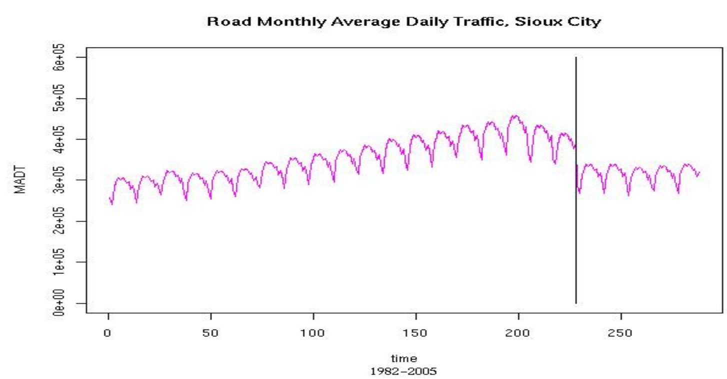

Figure 2. MADT over time on the main road of Site 1

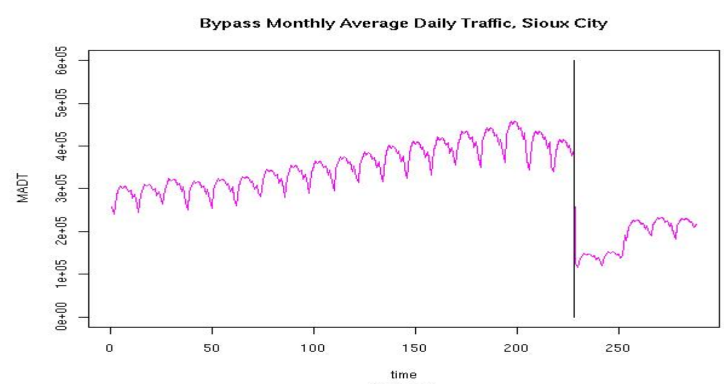

Figure 3. MADT over time on main road up to the intervention and on bypass afterwards at Site 1

4. METHODOLOGY

4.1. Exploratory Data Analysis

A literature review failed to reveal substantial documentation to answer the following questions:

1. Do bypasses affect accident frequencies and/or rates on the bypassed communities?
2. Do bypasses reduce overall accident frequency and/or rates, or do they merely shift the accidents from the highway through the community to the bypass with no significant overall reduction in frequencies and/or rates?

This study employs a full Bayesian approach (Gelman et al. 2004) to answer questions one and two above. An initial exploratory analysis was performed in order to determine the form of the regression model.

Figures 4, 5, and 6 present the number of crashes per month and per mile (i.e. monthly crash frequency per mile) on the road, the main road, and then the bypass after the intervention, and the sum of the road and the bypass (sum) respectively at Site 1. The vertical line in the plot marks the time at which the bypass was completed. The solid line in the graph is a smooth estimate of the number of crashes over time. The smooth curve was obtained by fitting a non–parametric local polynomial regression with optimal bandwidth (Simonoff 1996).

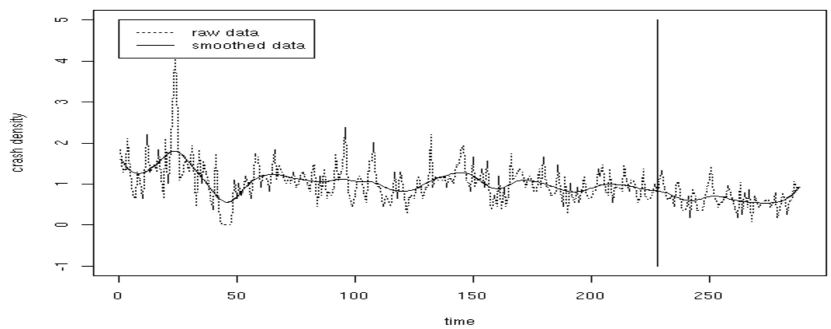

Figure 4. Observed monthly crashes per mile on the main road at Site 1

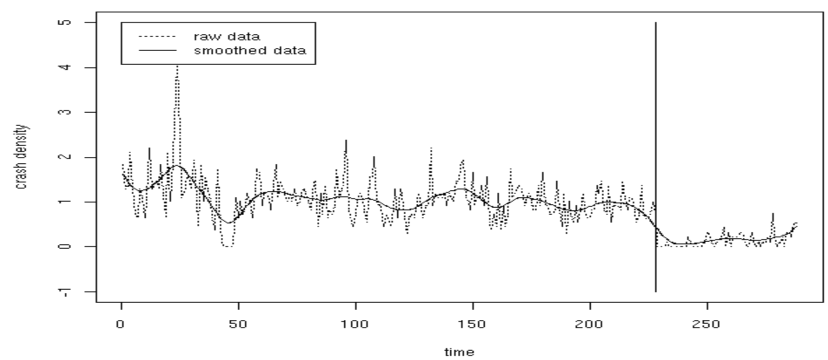

Figure 5. Observed monthly crashes per mile on the main road and then the bypass at Site 1

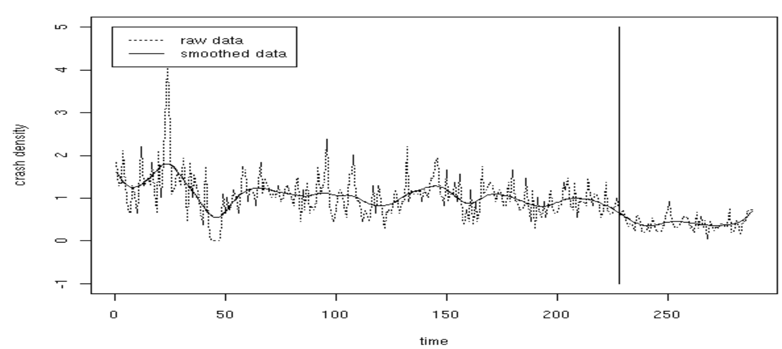

Figure 6. Observed monthly crashes per mile on the sum at Site 1

These plots also reveal the distinct impact of seasonal effects on the number of crashes. It is necessary to account for seasonality in the model for number of crashes.

Figure 7 is a representation of the smooth curves obtained from the crash rates on the road through town (before), the bypass (after), and the sum of the road and bypass at Site 1. The number of crashes per mile decreases after the intervention, both on the road through town and on the sum of the road and the bypass, even though the traffic volume at most sites increased over time.

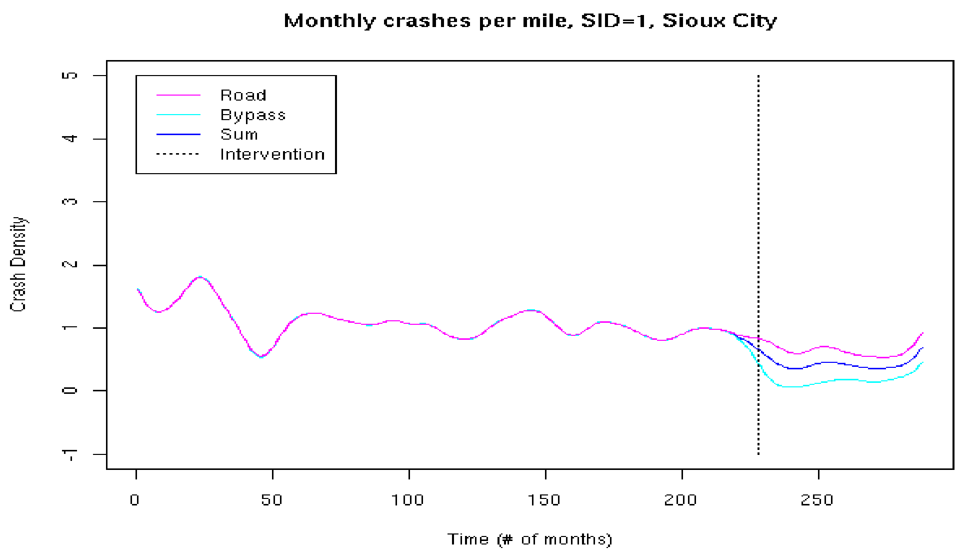

Figure 7. Smooth curves of all crash rates at Site 1

Distances along the old and new road were obtained with the use of the Geographic Information System software ArcView GIS 3.2 (Environmental Systems Research Institute, Inc., Redland, CA). The distances were measured from the point where the bypass branches off the old road to the point where it rejoins the old route. When calculating the distances on the sum, the portion of road at the junction of the old and new road has been taken into account in order to avoid double counting of the same stretch of road.

Based on the exploratory analysis, and the random nature of crash events, a Poisson regression model with a log-link function to associate the Poisson mean to a set of covariates was found to be appropriate. The Poisson-log normal model is essentially equivalent to a marginal negative binomial model on crash frequency. Thus, the overdispersion that is typically observed in the distribution of crashes is accounted for in the model. A set of trigonometric functions was included in the model to account for the seasonal effects on crashes.

The number of crashes per year per mile observed at each site has been calculated for the years preceding the bypass construction and following the bypass construction on Sites 1 through 19 on the main road through town. Table 3 contains such crash frequencies as well as their difference and observed percent reduction. The same values were computed and compared on the sum of the crash frequencies of the road through town after the intervention, and the bypass are also contained in Table 3.

Table 3. Observed average crashes per year per mile before and after the bypass construction

Site	Crash Frequency on Road Before Intervention	Crash Frequency on Road After Intervention	Difference Road After-Road Before Intervention	% Reduction Road After vs. Road Before Intervention	Crash Freq. on Sum (Road After Intervention + Bypass)	Difference Sum-Road Before Intervention	% Reduction Sum vs. Road Before Intervention
1	12.8	7.7	-5.1	40.1	4.9	-7.9	61.5
2	1.8	1.2	-0.7	36.9	0.6	-1.2	67.6
3	9.6	4.6	-5.0	52.0	3.3	-6.3	65.8
4	2.5	2.1	-0.4	17.9	1.7	-0.8	31.4
5	3.0	1.5	-1.5	48.6	1.0	-2.0	67.6
6	2.8	0.9	-1.9	66.7	0.6	-2.2	77.7
7	1.5	1.6	0.1	-7.2	1.6	0.1	-4.0
8	10.7	6.1	-4.7	43.4	3.6	-7.2	66.9
9	16.4	15.1	-1.3	7.7	7.1	-9.4	57.0
10	1.5	0.9	-0.6	38.4	1.0	-0.5	33.7
11	2.9	2.9	0.0	0.8	2.2	-0.7	25.6
12	6.4	2.2	-4.2	65.9	1.5	-4.9	75.9
13	44.87	44.9	0.03	-0.1	25.0	-19.9	44.3
14	10.1	8.1	-2.0	19.8	4.3	-5.8	57.3
15	6.1	5.2	-0.9	14.7	2.7	-3.5	56.5
16	11.8	10.9	-0.8	7.2	5.5	-6.3	53.6
17	1.7	1.3	-0.4	22.9	1.4	-0.2	14.5
18	6.1	4.1	-2.0	32.6	2.8	-3.2	53.1
19	7.5	4.0	-3.4	46.0	2.8	-4.7	62.7
20	11.2	NA	NA	NA	NA	NA	NA
21	1.1	NA	NA	NA	NA	NA	NA
22	3.8	NA	NA	NA	NA	NA	NA
23	1.5	NA	NA	NA	NA	NA	NA
24	3.6	NA	NA	NA	NA	NA	NA
25	1.4	NA	NA	NA	NA	NA	NA

5. RESULTS AND INTERPRETATION

A Bayesian analysis has been performed and a hierarchical Poisson model was fitted to the crash frequencies. In the model the log mean was expressed as a function of time periods and seasonal effects. A random effect corresponding to each site was also included in the model.

5.1. Expected Crash Frequencies and Crash Rates

In order to quantify the difference in crash frequency during the before and after periods, the posterior distribution of the annual frequency per mile as well as the posterior distribution of the percent reduction in crash frequency was computed for each site. The posterior distributions were estimated for each of the sites during the year preceding the construction of the bypass and for the years following the bypass construction at Sites 1 through 19 (the treatment sites).

The posterior distributions of expected annual crash frequencies per mile on the main road for the untreated sites and for all treated sites over the years preceding and following the bypass construction are presented in Figure 8. This figure shows the effect of the bypass construction on the main road through town.

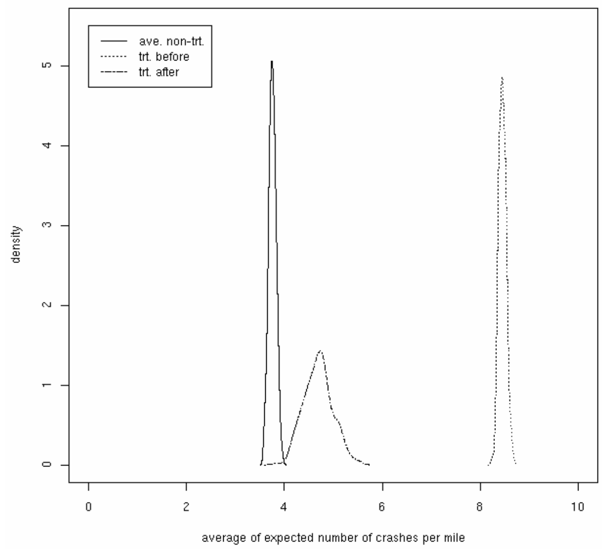

Figure 8. Posterior distributions of the expected annual crash frequencies per mile on main road

Figure 8 reveals that there is a pronounced reduction in expected annual crash frequency per mile in treated sites after the bypass construction. The posterior distributions are narrow, indicating that the posterior mean is a reliable summary of the distribution of likely values of expected crash frequencies.

Traffic volumes also appear to be rising over time at all sites; therefore, the posterior distributions of the expected annual crash frequencies on the main road per site per mile were recomputed normalizing each site to a 10^6 AADT, obtaining the expected annual crash rate per mile. Figure 9 shows the computed posterior distribution of expected annual crash rates per mile for all treated and untreated sites over the years preceding and following the bypass construction.

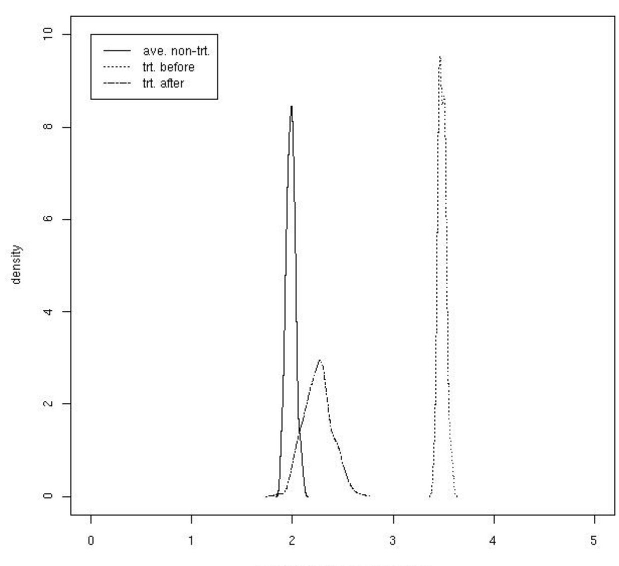

Figure 9. Posterior distributions of the expected annual crash rates per mile on the main road

The posterior means and credible sets for the average expected crash frequencies were computed on the main road before the construction of the bypass and the sum of the average expected crash frequencies on the main road through town and the bypass after the construction of the bypass. Also, the difference in expected crash frequency (main road before - sum of road after and bypass) and the percent reduction in the expected annual crash frequency per mile at each bypassed site between the after and before periods were computed.

Figure 10 presents the posterior distribution of the expected annual crash frequencies per mile for all treated and untreated sites over the years preceding the bypass construction and the sum of the crash frequencies of the road and the bypass after the construction.

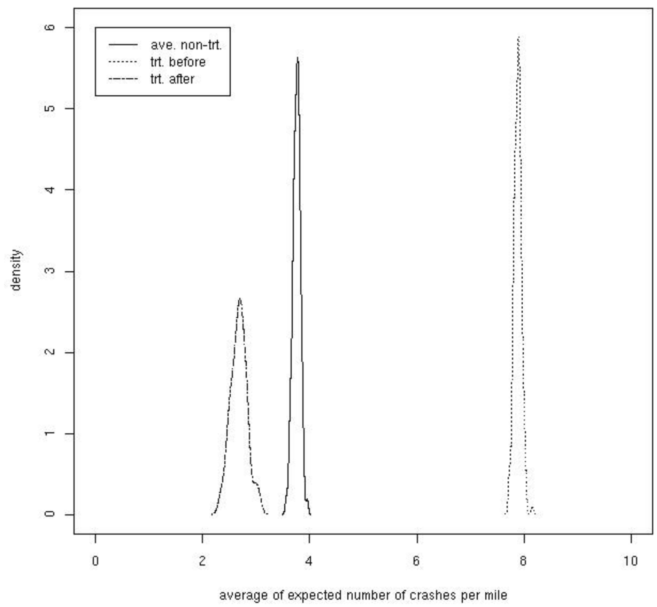

Figure 10. Posterior distributions of the expected annual crash frequencies per mile on the sum

From the figures it can be noted that the reduction in expected annual crash frequency per mile on the sum is more pronounced than it was on the main road after the bypass construction and is lower than before the intervention.

The posterior distributions of the sum of the expected annual crash frequencies of the main road and the bypass per site per mile were recomputed normalizing each site to a 10^6 AADT, obtaining the expected annual crash rate per mile.

Figure 11 shows the computed posterior distribution of expected annual crash rates per mile for all treated and untreated sites over the years preceding the bypass construction and the sum of the crash rates after the construction.

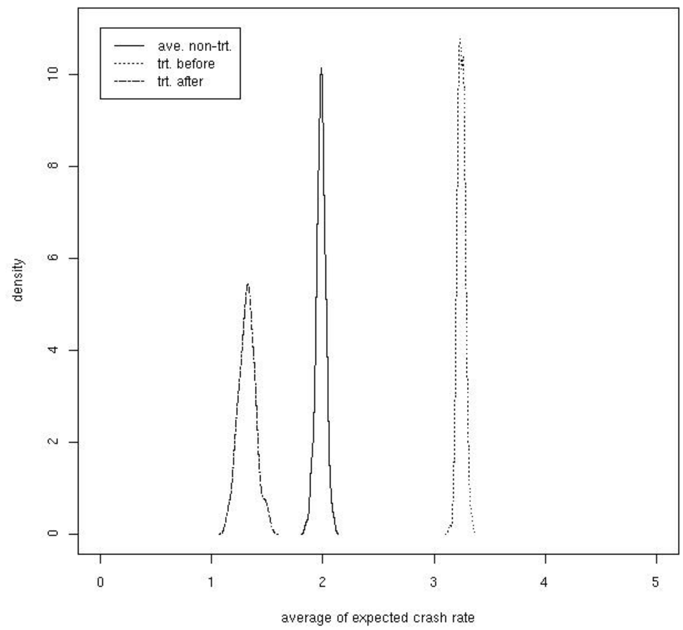

Figure 21. Posterior distributions of the expected annual crash rates per mile on the sum

6. CONCLUSIONS AND DISCUSSION

The traffic safety problem has been examined for many years with the use of various techniques. Classical statistical methods have been generally used due to their widespread understanding and ease of use. The inexpensive access to powerful computers and the development of advanced statistical techniques in recent years have allowed for Bayesian statistics to be applied to traffic safety.

This research adopted a Bayesian approach to assess the impacts of the construction of highway bypasses on the traffic safety of the combination of local and long-distance travelers driving through the bypassed location and the safety of the road travelers residing in the bypassed location. Additionally the impact on traffic safety has been assessed on the old road network consisting of the main road through town, and on the new road network consisting of the main road through town and the newly constructed bypass.

The study consisted of two parts. The first part analyzed monthly crash data for 25 sites in Iowa over a period between 1982 and 2005. Nineteen of the sites were bypassed during the study period while six sites were not bypassed. The un-bypassed sites served as reference sites. A preliminary analysis suggested the use of a hierarchical Poisson model fitted to the crash frequency observed at each site. The model took into account seasonal effects, treatment, time, and interactions for treatment and time. The association between monthly crash rate and explanatory variables was estimated with the use of a random effect included in the model. Proper but non–informative priors have been used for all parameters, and the calculations have been carried out using Markov Chain Monte Carlo methods.

The posterior means obtained were used to calculate the expected annual accident frequencies and rates at each study site. The expected annual accident frequencies and rates were compared for the periods before and after the bypass construction. Subsequently the expected annual accident frequencies and rates at each site were compared to the average of the accident frequencies and rates of the reference sites. The estimated parameters and the results from the posterior predictions suggested that the model was reasonable and fitted the data well.

Moreover the results indicated that the construction of highway bypasses in Iowa appears to be associated with an increase in traffic safety by the reduction of the number of crashes both on the old and new road networks at least in the state of Iowa and on the type of roads considered in the study. The crash frequencies on average were reduced 50% on the old road network and 62% on the new road network. The crash rates on average were reduced 33% on the old road network and 59% on the new road network.

In order to forecast the expected crashes for each of the 25 study sites, predictions were computed for each season of two selected years following the available data range: 2006 and 2008. The expected crash rates and expected number of crashes per mile during those years decrease more rapidly at bypassed sites than at reference sites.

The main objective of the second part of the study was to assess the safety of citizens of the bypassed communities and analyze data on 23 out of the 25 sites over the years between 1994 and 2005. The time range was dictated by the lack of information on driver's provenance prior to 1994. Two of the original 25 sites were excluded from this part of the analysis due to the fact that in one site the bypass was built before 1994 and there were no accidents found involving local citizens in the other site. Five of the study sites were not bypassed and served as reference sites. For all 23 sites in this part of the study only the accidents involving residents of the sites were considered. The same approach and model used in the first part of the study was also employed here, and expected crash frequencies and rates were calculated from the posterior means obtained from the analysis.

The results of this second analysis also suggested that the model was a good fit for the data. The expected crash frequency on the old road network after the construction of the bypass on average decreased 38%. The crash rate, however, on average increased 20%. On the new road network the average crash frequency decreased on average 47%. The average crash rate on the new road network decreased on average 10%.

The increase in accidents on the old road network may be attributed to many variables, among which a reduction in traffic volume on the main road is one.

6.1. Recommendations

The results of this research are reassuring and suggest that they could be incorporated by the Iowa DOT in the decision process for selecting new bypass locations. Using available data for safety improvement locations, a preliminary and Bayesian analysis similar to those implemented in this work can produce results for forecasting the expected decrease in crash rates at those sites.

In order to assess the cause of the increase in accidents involving local citizens on the old road network, it is necessary to collect more detailed data. Traffic speeds should be surveyed on the main roads before and after the construction of a bypass in order to assess if they increase with the reduction of traffic volumes. Observational data should also be collected at certain sites in order to study driver behavior. It may be necessary to determine if the increase in accidents is related to behavioral issues. After the bypass is in place and the traffic volume decreases downtown, some drivers may feel safer and behave differently, causing them to be less cautious in their driving.

Another aspect that may lead to determining the root cause of this increase may be studying the accident severity on the main road. Severity is often related to traffic speed. If the severity of accidents decreases or remains the same on the main road, this may suggest that speeds have not increased.

This research has highlighted the impact of weather on crash frequencies and rates. Future reports should also collect information on road conditions and traffic volume by season. Road conditions may also need to be included in the police accident reports to help better understand if the increase of accidents in the colder seasons is due to an increase in traffic volume or to the road conditions.

More aspects of the impact of the construction of bypasses can be examined. This research was performed on road sections that extend for a few miles. Future research should investigate the accidents at the junction of the bypass and the road through town both at the point where the two roads split off and at the point where they tie back. This could also be done by segmenting the roads and looking at stretches of ¼ mile in length. It may also be of interest to investigate the impact of the construction of bypasses in Iowa on injury accidents by isolating those types of accidents from the crash databases.

An aspect that may improve before-and-after studies such as those described in this thesis would be to define appropriate matching criteria and identify control sites to match each bypassed site. This approach would allow for further comparison of treatment and control sites.

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A Hybrid Approach for Determining Traffic Demand in Large Development Areas

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ABSTRACT

Trip distribution is an important element of traffic impact analysis and is typically performed using expert judgment. Impacts are very sensitive to this step, and for large, mixed-use developments with many parcels, trip distribution can be very time consuming and potentially error prone. Regional trip distribution, on the other hand, is generally computed using a gravity model as part of a three or four step modeling process, which is efficient but that does not allow for the introduction of expert opinion, and outputs must generally be smoothed prior to use for traffic or geometric design. We present a hybrid approach that first obtains trip distribution patterns from a regional (TransCAD) model. Origin-destination (OD) pairs and graphical highway network are then imported into an Excel spreadsheet using a comprehensive set of VBA macros. The paper discusses additional macros developed that (1) implement a driveway-spread method to assign peak hour trips, (2) allow for quick, easy, and intuitive revision of flow patterns, and (3) export network and intersection flows in a traffic program (Synchro) for operational analysis and simulation. We provide a real-world case study where the tool was applied within a rapidly growing community (Johnston Iowa, 2,600 -acre development area with 40 sub-zones). The hybrid approach is expected to greatly reduce time and errors associated with manual trip distribution and site impact analysis.

Key words: land use—operational analysis—trip assignment—trip distribution—trip generation

INTRODUCTION

Traffic forecasts are key inputs to planning and investment for large development areas. Roadway designers require reliable and sufficiently detailed forecasts to ensure acceptable road performance initially and over the project's design life. However, traffic demand analysis for large development areas is full of challenges given the uncertain trip generation and distribution potential of undeveloped or redeveloping land as well as the operational nature of adjacent arterials (Hawkins et al. 2007). Manual techniques typically applied to small sites for impact analysis are not well suited for larger developments due to labor requirements and potential errors due to the vast number of calculations and tabulations that must be made and kept track of. More automated methods, such as travel demand network (or four-step) models are similarly unsuited due to their coarse, aggregate nature, and more especially because they do not allow for the ready introduction of expert judgment. This judgment, at the local level, can be superior to the sometimes arbitrary and opaque distribution and assignment decisions made by the computer algorithms.

PROBLEM STATEMENT

For determining traffic demands in large development areas, traditional site impact studies are expensive if not impractical (Ruehr 2000). This is partly because a great number of distribution percentages must be estimated for a large number of parcels or land uses. Here is an example. For a development with 15 external stations and 40 internal land uses (as described in the land use plan), trip distribution for ingress/egress trips for morning and afternoon peak hours requires as many as 2,400 estimates. If distributions between internal zones are taken into account, the number of total distribution pairs can be as many as 8,640. Manual distribution is, therefore, impractical, particularly if estimates must be revised or repeated for various land use scenarios.

For truly large developments, analysts may consider the application of a regional travel model to estimate distribution patterns for the study area. However, these models must probably be refined to include more detail for the site in question (so-called focus or sub-area models). Subdivided zones should reflect the land use plan, which may be quite detailed (parcel level). Regional models are often calibrated only for daily volumes so peak hour adjustments must also be made, as peak hour volumes, by direction, are required for design and traffic purposes. Finally, regional trip generation is typically based on employment and demographics, as these variables are easier to forecast than land use activity levels required as inputs to site-specific trip generation. Community development departments will likely have detailed estimates of land use by type and acreage, at least for current and short-term forecasts. Therefore, the conversion of land use to population/employment-based trip generations must be accomplished before subdivided zones can be seamlessly included in regional models.

Post processing and repackaging of traffic estimates is required prior to use by traffic and design engineers. Regional model auxiliary programs can build sub-area networks and report the origin-destination (OD) flow table for a specific study sub-area. However, as these programs are not developed specifically to support sub-area traffic studies, they may be cumbersome or inflexible to use. For example, it is not easy to quickly review and modify zonal flow patterns thought or known to be incorrect. It is also not easy to set multiple paths (or split trips among multiple paths).

For sub-area study, the analyst should examine land use locations and layout in each zone to determine the percentages of zonal trips to be loaded onto driveways. Trip assignment thus starts at driveways rather than at centroids as is the case in regional models. Regional planning packages lack a convenient way to implement this kind of driveway-spread assignment.

METHODOLOGY

This paper presents an approach that combines the salient features of manual and computer methods currently used to analyze traffic impacts at much smaller and at much larger scales. This hybrid approach takes advantage of trip distribution outputs from a regional travel model, utilizing them as surrogate patterns for distribution of directional, zonal peak-hour trips in the study area. The proposed approach also makes extensive use of the familiar Excel spreadsheet for data input, manipulation, revision and display. An easy-to-use and intuitive set of worksheets was developed, serving as the platform for integration of regional travel modeling, expert judgment, and traffic operational analysis. An application of this approach is presented for the western growth area of Johnston, Iowa.

The hybrid approach bridges a gap between land use and demographic-socioeconomic data based trip generation and enables the use of distribution outputs from a regional travel model. Many Excel VBA macros were developed to implement trip generation, distribution, and assignment for large-scale developments. Some of the macros graphically translate the sub-area network into the spreadsheet and calculate trip generation and distribution percents for each zone in the study area. Others facilitate splitting trip distribution by driveway and assign trips to the network. Still other macros report turning volumes and percents after assignment and display flow pattern by color-coded lines.

Some of the macros facilitate rapid revision of traffic assignment. Others create a sub-area network for Synchro and export intersection volumes to Synchro via Universal Traffic Data Format (UTDF). Synchro may then be used to test different traffic controls, explore the mitigation of traffic impacts from land development, and run simulations for public meetings. A flow chart depicting the hybrid impact analysis approach is shown in Figure 1.

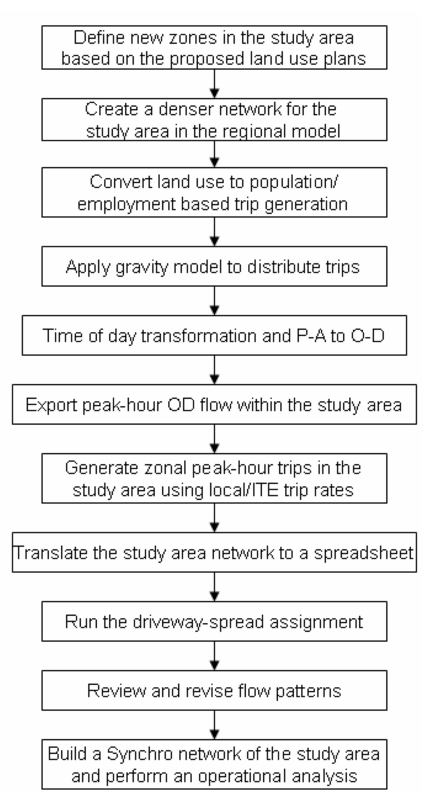

Figure 1. The hybrid approach to traffic demand analysis

CASE STUDY

Located northwest of Des Moines, the city of Johnston is one of the fastest growing communities in Iowa. The city is preparing to improve public roadways in its western portion. Private investment is expected to continue in this area on a large scale. Given the large tracts of undeveloped land (green field), establishing the number of lanes and size of intersections was a considerable challenge. In addition to general roadway design parameters, city staff wanted to know future roadway costs for input to a capital improvement plan. They also wanted to know future traffic conditions in order preserve the appropriate amount of right-of-way and to be better prepared to negotiate with both residential and commercial developers in this yet underdeveloped portion. Each of these required a reliable and sufficiently detailed traffic demand study considering land use impacts at the major driveways, intersections, and turning lanes level.

The case study began with the collection of land use plans and estimation of future trips. The specific uses and acreages of anticipated development within the area were provided by the city. The 2,600 acre study area was subdivided into 40 analysis zones. Subdivision was dependent on the mix of land uses and the ingress/egress routes assumed, and complex areas were subdivided more to allow for higher levels of detail in the analysis. Figure 2 shows the land use information in the study area.

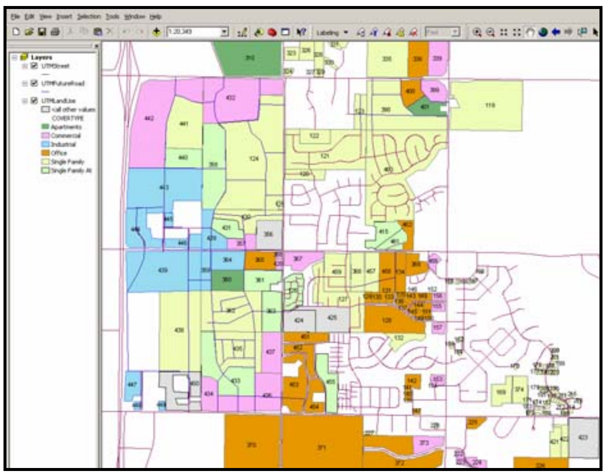

Figure 2. Land uses in the study area

Peak-hour trip generation was calculated using trip rates, pass-by percents and the numbers of development units. Where local rates or pass-by percents were not available, national average values from the *ITE Trip Generation Handbook* (ITE 2003) were used.

In order to provide a baseline trip distribution and reduce potential input errors, the Des Moines area 2030 TransCAD model was used. To acquire distribution percents for each of 40 zones in the development area, the original six traffic analysis zones in the Des Moines model were subdivided into the 40 study area zones. The sub-area roadway network was also updated and densified. Figure 3 shows the revised model network in the study area.

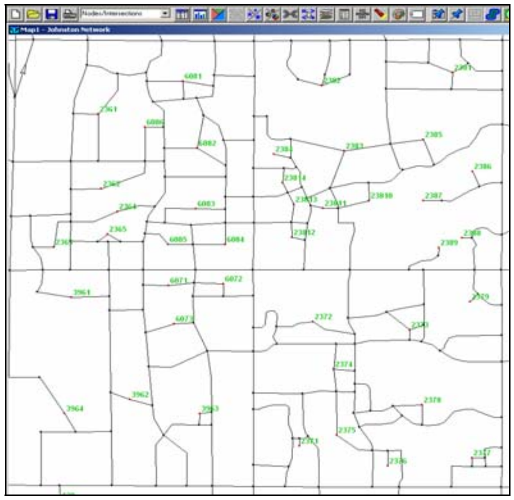

Figure 3. The revised model network in the study area

Before peak-hour distribution patterns could be identified, two limitations of the regional model had to be addressed:

1. Conversion of Land Use to Population and Employment

The 2030 regional model was developed as a 24-hour (daily) model. Daily trips are generated based on estimates of population and employment. Before integrating the subdivided zones with the regional distribution model, land use information for these zones had to be converted to corresponding population and employment numbers. For residential zones, the percentages of household sizes (1, 2, and 3+) for single-family housing, townhouse/condominium, and apartments were estimated considering local demographic characteristics. Regional trip production rates were then applied to these household estimates based on projected income levels in the area.

For business, commercial and industrial land uses, ITE trip rate curves were used to convert land use to employment numbers. For example, 140,000 sq. ft. of general office building generates 1,500 daily trips, which is equivalent to the daily trips generated by the same type of building with 400 employees. The 400 employees were then considered as non-retail employment numbers. Figure 4 illustrates this process. Similarly, retail employment and school enrollment can also be estimated in this way. These conversions and calculations were completed by macros written in the spreadsheet.

Next, the regional trip production and attraction models were applied to the sub-area zones producing daily trips by purpose, including home-based work (HBW), home-based other (HBO), non home-based (NHB), and commercial vehicle (CV).

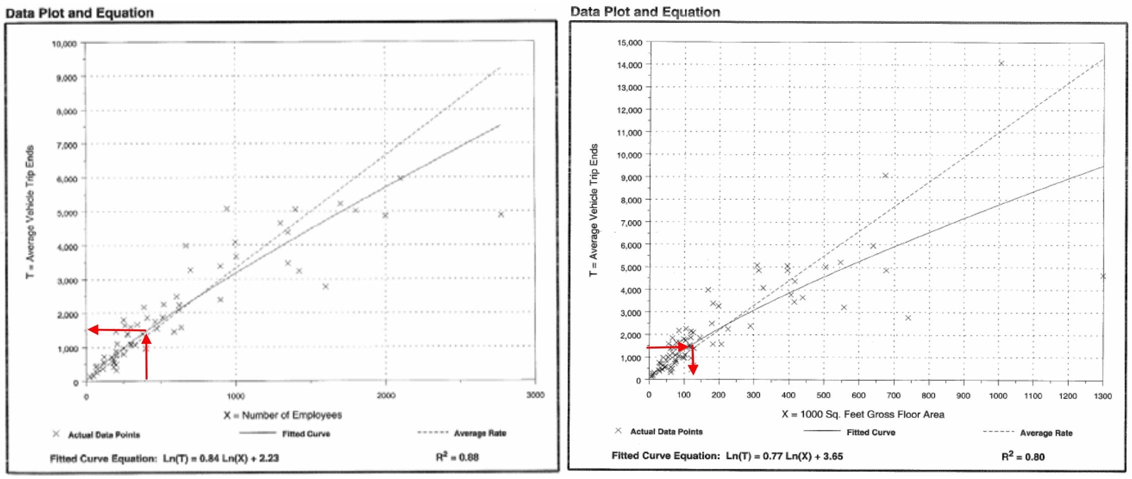

Figure 4. The conversion of land use acreages to employments

2. Time-of-Day Transformation

To convert 24 hour trip generation to peak-hour, it is most helpful to have a local survey. As no such survey was available in our case study, default peak-hour factors from NCHRP Report 187 were used to decompose regional daily production-attraction (P-A) trip table to peak-hour matrices. After the time-of-day transformation, a P-A to OD conversion was conducted to get peak-hour OD matrices.

The last step to obtain zonal distribution patterns for the study area was creating a sub-area OD matrix. TransCAD has a procedure for performing this step (Caliper Corporation 2005). After specifying a cordon line that circumscribes the study area, TransCAD reported this matrix. The final result is the sub-area peak-hour distribution pattern.

Next, the TransCAD network in the area was automatically translated into Excel for further analysis. The macros developed for this process also support the driveway-spread assignment, which provides output at a level of detail more conducive to traffic analysis and design. It also allows the analyst to incorporate local knowledge and expert judgment. An example of the configuration of driveway usage for a zone is illustrated in Figure 5.

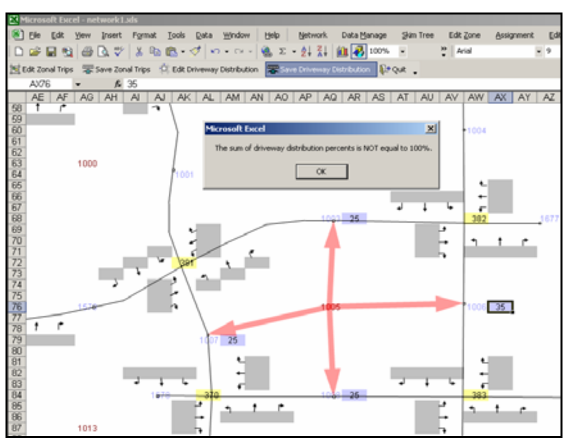

Figure 5. The configuration of zonal driveway usage

The spreadsheet macros also allow quick, easy, and intuitive turning movement analysis and flow pattern modification. For instance, Figure 6 shows traffic flow leaving a centroid through one of its driveways. Upon inspection, analysts can directly make changes on the reported turning percents and revise the flow pattern.

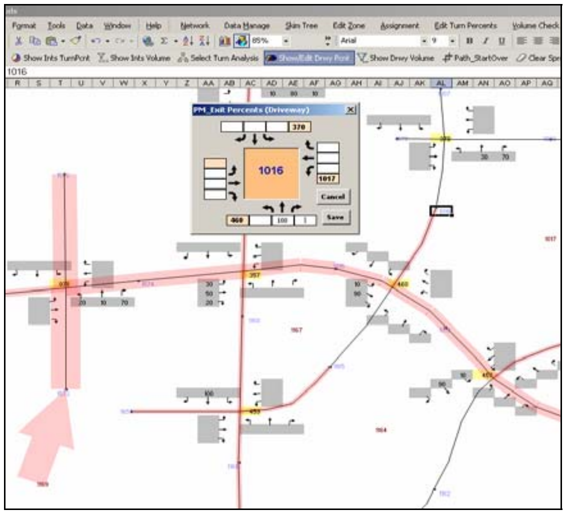

Figure 6. Reviewing and revising traffic flow pattern

The principal goal of this study was to evaluate how road facilities handle traffic under major land development and growth and to forecast intersection level of service. For the Johnston case study, the spreadsheet automatically created the study area's network for Synchro using the node-link data from TransCAD. It also translated volume data to Synchro for more additional operational analysis. The network created by the spreadsheet tool is displayed in Figure 7. Intersections on major roadways and accesses to employment centers and schools were analyzed in the Synchro network.

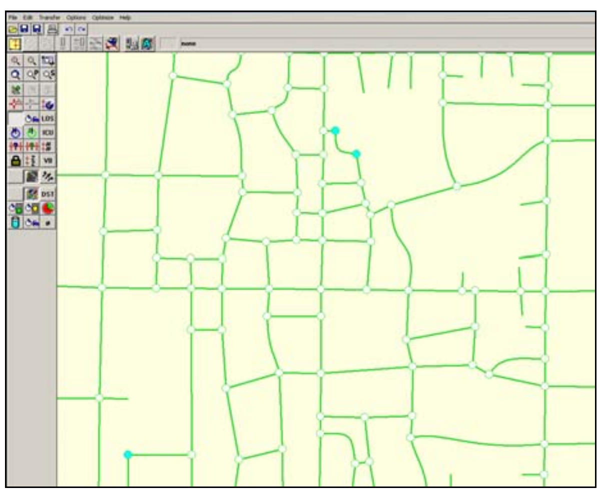

Figure 7. The sub-area network in Synchro

CONCLUSIONS

The hybrid approach introduced in the paper integrated a land use-based approach with population/employment-based trip generation, and trip distribution outputs from a regional travel model for initial trip assignment in a large development area. Compared to previous manual efforts in similar areas conducted by the authors, the tool greatly reduced the time and errors associated with traditional manual trip distribution and site impact analysis.

The use of the familiar Excel spreadsheet is an important feature of the hybrid approach because it serves as a transparent tool accessing the best features of regional travel models, expert judgment, and traffic operational analysis. While the chief benefits of the described approach lie in its ability to provide outputs most useful for traffic engineering and design (as well as to save time and reduce error), the methodology developed in the study has potential for enhancing sub-area peak-hour travel modeling in general. It is recommended that future efforts address the integration of the analysis steps and facilitate the testing of different land use scenarios.

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Field and Laboratory Studies on High-Mast Lighting Towers in Iowa

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ABSTRACT

Recent failures of high-mast lighting towers in several Midwestern states have raised questions as to the robustness and safety of the existing inventory of similar structures. Failure of these structures is very critical, as they are typically located adjacent to interstates or other high-speed highways. The potential exists for these fracture-critical structures to fall across multiple traffic lanes or adjacent property. Forensic studies on cracked poles in the state of Iowa and elsewhere have revealed fatigue to be the primary cause of the failures.

Interestingly, there is little data on the actual response of these structures in terms of natural wind, long-term cyclic stresses, and general dynamic properties. Believed to be the first of its kind, the Iowa Department of Transportation has sponsored a comprehensive field testing and long-term remote monitoring program that has focused on two poles in near Clear Lake, Iowa. The results of the field studies suggest that additional fatigue cracking can be expected at other poles in the future. In order to address this issue, retrofit strategies have been developed and are currently going through static and fatigue tests to establish the fatigue resistance and long-term viability of the design. The laboratory tests, which have just begun, have been augmented with detailed finite element analytical studies on the base plate connection. This paper will report on the initial results of the field studies and provide suggested strategies for retrofit, inspection, and maintenance of these fracture-critical structures.

Key words: fatigue—field testing—high-mast tower

BACKGROUND

Following the November 12, 2003, collapse of a 140 ft. high-mast lighting tower along I-29 in Sioux City, Iowa, an intensive investigation into the cause of failure was carried out (Dexter 2004). Subsequently, two high-mast towers near Clear Lake, Iowa, were field-tested and monitored for one year, from October 2004 through November 2005. One of the towers was retrofitted with a steel reinforcing jacket at its base, while the other remained as originally designed (Connor and Hodgson 2006).

An additional phase of testing has been undertaken to further study the behavior of the reinforcing jacket and is the subject of this preliminary report. The as-built tower located near Clear Lake, Iowa, was retrofitted in June 2006 with a modified reinforcing jacket (designed and installed by Wiss, Janney, Elstner, and Associates). Static and dynamic load tests were performed, and long-term monitoring of the tower is currently underway. This report highlights the preliminary findings of this current phase of work.

Objectives of the Current Study

The current study was initiated to quantify the stresses induced in critical details on the reinforcing jacket and the tower itself through the use of field instrumentation, testing, and long-term monitoring.

Strain gages were installed on the both the tower and the reinforcing jacket. Additional strain gages were installed on two anchor rods. Tests were conducted with and without the reinforcing jacket installed. Data were collected from all strain gages during static load testing and were used to study the stress distribution of the tower caused by known loads, both with and without the reinforcing jacket. The tower was tested dynamically by first applying a static load and then quickly releasing the load, causing the tower to vibrate freely. Dynamic properties of the tower, such as modal frequencies and damping ratios, are obtained by reducing the data.

Summary of the Field Testing Program

Installation of all sensors and load testing were conducted during the week of June 26, 2006. The tower is located at the I-35/US 18 interchange near Clear Lake, Iowa. It is denoted tower number 1 of the interchange. (This tower was termed the “as-built tower” in reference [Connor and Hodgson 2006].)

A series of static and dynamic loading tests were conducted. These tests were conducted by statically loading the towers with a cable fixed at one end and connected to the tower approximately 35 ft. above the base. The load was subsequently released rapidly to allow the tower to vibrate freely. These dynamic, or “pluck,” tests produced a free decay vibration signature that is used to extract both the natural frequencies and damping characteristics of the high-mast tower.

In addition to the load testing, a 12-month long-term monitoring program is currently underway to quantify the response of the tower under natural wind loading. During the long-term monitoring period, ambient vibration data are recorded (for 15 to 30 minutes) when wind speeds and/or tower stresses exceed predetermined trigger levels. Additionally, stress-range histograms are continuously developed. Furthermore, wind speed and direction are continuously monitored.

INSTRUMENTATION PLAN AND DATA ACQUISITION

The following sections describe the sensors and instrumentation plan used during the static/dynamic testing and the long-term monitoring programs.

Strain Gages

Strain gages were placed at predetermined locations. All strain gages installed in the field were produced by Measurements Group, Inc. and were 0.25 in. gage length, model LWK-06-W250B-350. These gages are uniaxial weldable resistance-type strain gages. Weldable-type strain gages were selected due to the ease of installation in a variety of weather conditions. The welds are point- or spot-resistance welds about the size of a pin prick. The probe is powered by a battery and only touches the foil that the strain gage is mounted on by the manufacturer. This fuses the foil to the steel surface. It takes 40 or more of these small welds to attach the gage to the steel surface. There are no arc strikes or heat-affected zones that are discernible. There is no preheat or any other preparation involved other than the preparation of the local metal surface by grinding and then cleaning before the gage is attached to the component with the welding unit. There has never been an instance of adverse behavior associated with the use of weldable strain gages, including their installation on extremely brittle material such as A615 Gr75 steel reinforcing bars.

These strain gages are also temperature compensated and perform very well when accurate strain measurements are required over long periods of time (months to years). The gage resistance is 350 ohms, and an excitation voltage of 10 volts was used. All gages were protected with a multilayer weatherproofing system and then sealed with a silicon-type compound.

Accelerometers

Two uniaxial accelerometers were used for the dynamic tests only. The accelerometers were manufactured by PCB Piezotronics, Inc. (model 3701G3FA3G). This accelerometer has a peak measurable acceleration of 3 g.

These accelerometers are termed capacitive (or DC) accelerometers. The primary component of these sensors is an internal capacitor. When subjected to acceleration, the sensor outputs a voltage in direct proportion to the magnitude of the acceleration. They are specifically designed for measuring low-level, low-frequency accelerations, such as those found on a bridge or a high-mast lighting tower.

Anemometer

An anemometer is used to measure wind speed and direction and is installed atop a 30 ft. wooden telephone pole directly adjacent to the high-mast tower. The anemometer (model number 5103) is manufactured by R.M. Young, Inc. and is a propeller-type anemometer. Both wind speed and wind direction are measured.

Data Acquisition System

A Campbell Scientific CR9000 data logger was used for the collection of data during all static and dynamic testing and continues to be used for the long-term monitoring phase. This logger is a high-speed, multichannel 16-bit data acquisition system. The data logger was configured with digital and analog filters to assure noise-free signals.

The data logger is enclosed in a weather-tight box adjacent to the tower. Remote communications with the data logger are maintained through a satellite internet connection. Data collection is performed automatically. The satellite link is also used to upload new programs as needed. Data are collected and reviewed periodically to assure the integrity of the data.

Instrumentation Plan

A total of 25 strain gages were applied to the tower. Of these, 12 were installed on the existing tower, 9 were installed on the reinforcing jacket, and the remaining 4 gages were installed on two anchor rods. Key drawings are presented in Figure 1.

On the existing tower, a set of four strain gages was installed 90 degrees apart at a section 6 ft. above the base plate. Two strain gages were placed on the tower 180 degrees apart approximately 53 in. above the baseplate (just below the top of the reinforcing jacket). Two strain gages were placed on the tower 180 degrees apart 12 in. above the baseplate. Finally, a set of four strain gages was installed 90 degrees apart adjacent to the base weld. All of these strain gages were oriented vertically. The inside surface of the reinforcing jacket was ground out at strain gaged locations so pressure would not be applied to the strain gage after the bolted jacket connection was fully tightened.

On the reinforcing jacket, two strain gages were installed 180 degrees apart approximately 53 in. above the baseplate (just below the top of the reinforcing jacket). Two strain gages were installed 180 degrees apart approximately 21 in. above the baseplate. Finally, five strain gages were placed around the perimeter of the reinforcing jacket adjacent to the base weld. Again, all of these strain gages were oriented vertically.

Two anchor rods on the north side of the tower were instrumented. On each anchor rod, two strain gages were installed 180 degrees apart on the free length of the anchor rod between the concrete foundation and the underside of the base plate.

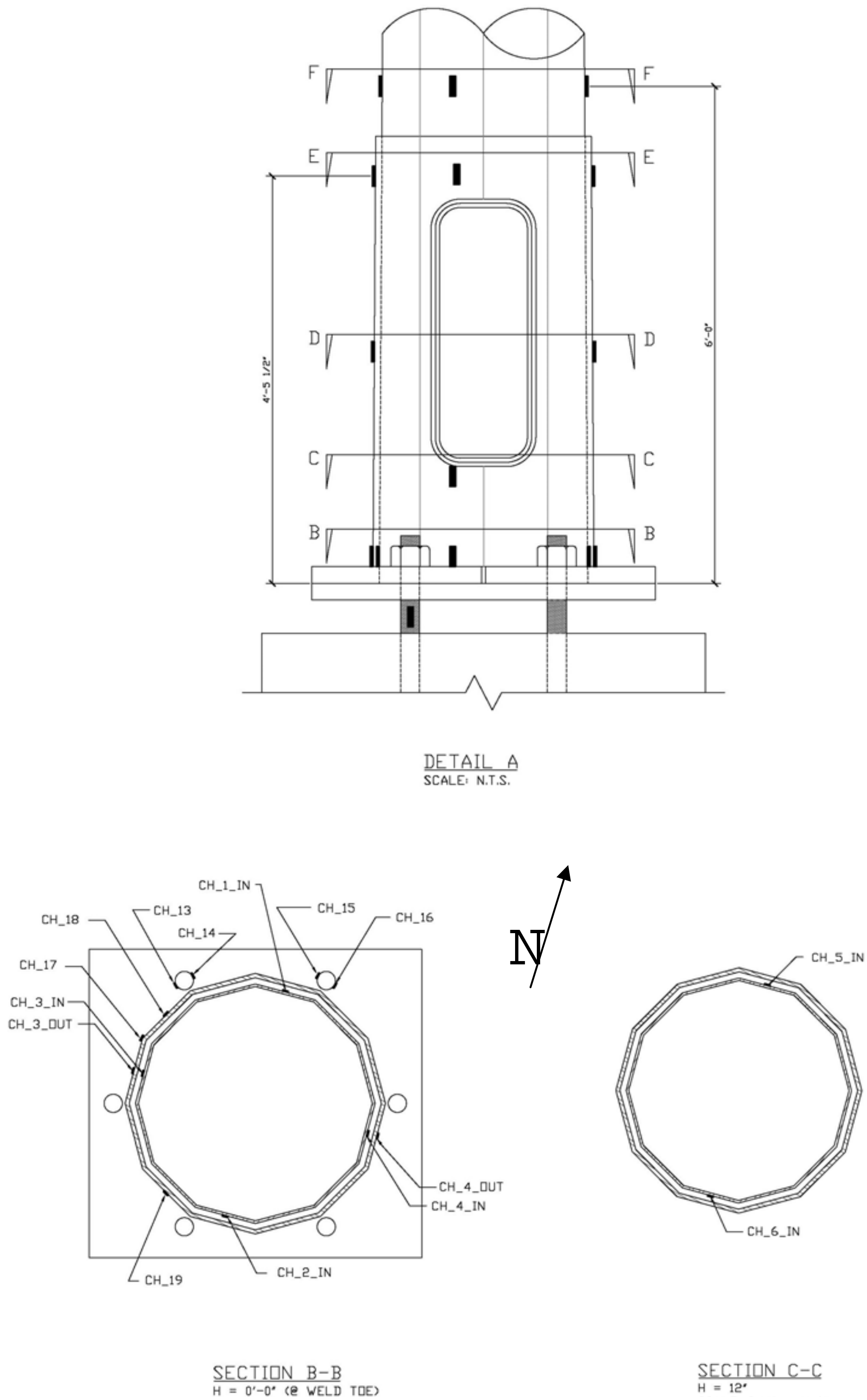

Figure 1. Section drawings showing strain gage locations

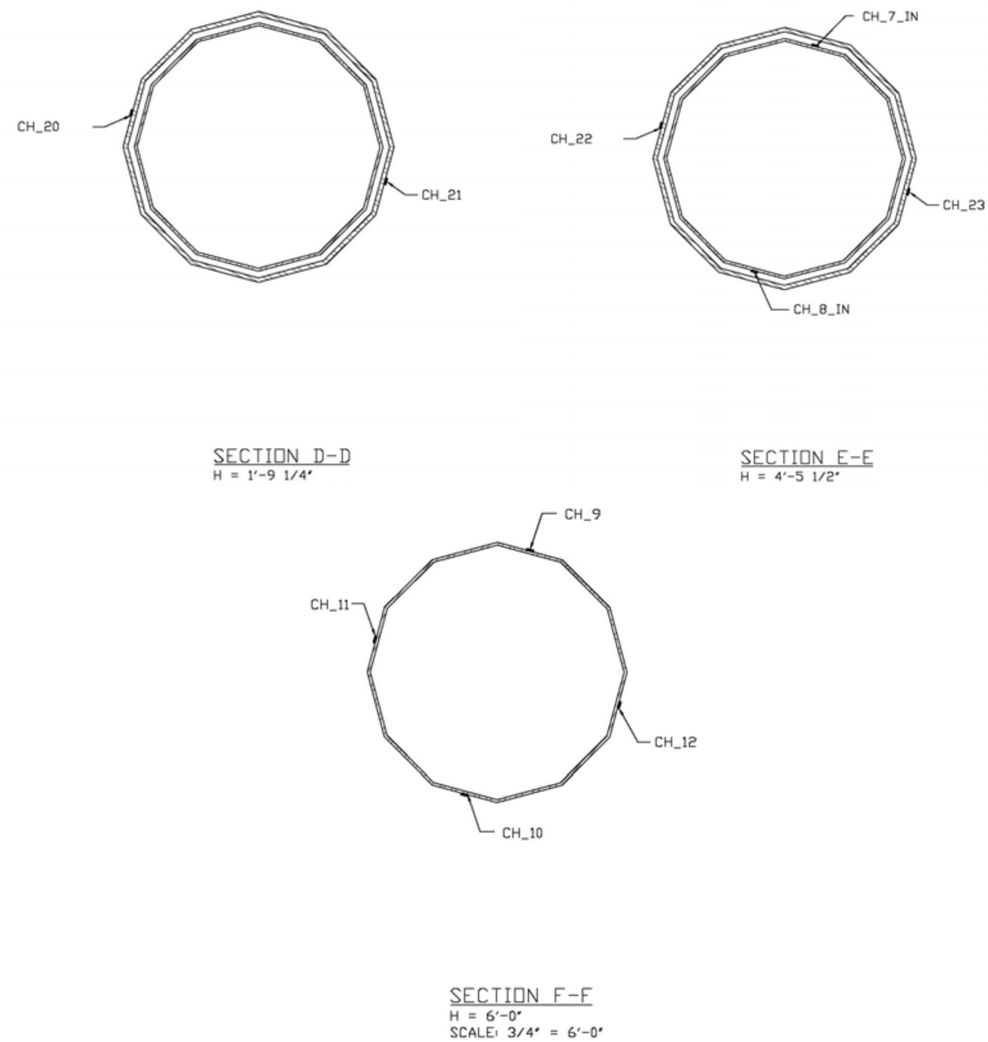

Figure 1. Section drawings showing strain gage locations (continued)

RESULTS OF STATIC TESTING

Static tests were performed on the high-mast tower, both with and without the reinforcing jacket in place. In both cases, the load was applied in both the north and west directions. Tests were repeated multiple times. The load was applied using a nylon sling wrapped around the tower at a height of approximately 35 ft. The load was applied using a come-along connected to the sling with a wire rope. The other end of the wire rope was connected to the back of a heavy truck at ground level. This is shown schematically in Figure 2. As a result of the inclination of the cable, a lateral force (causing bending) and a downward force (causing compression) are applied to the tower.

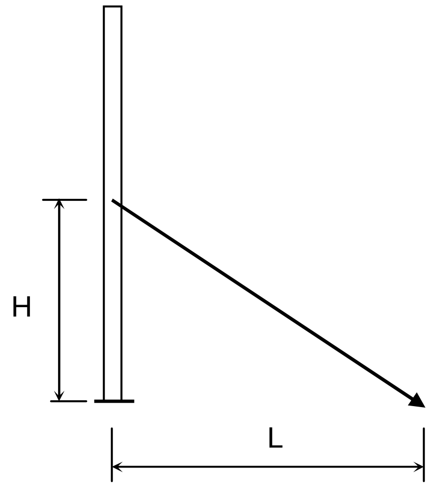

Figure 2. Schematic drawing of static pull tests with key dimensions

Presented in Table 1 are the peak stresses measured in all strain gages on the as-built tower (no reinforcing jacket) for pull tests in the north-south directions, respectively. Also shown in the table are the calculated moment and stress at a section 6 ft. above the baseplate. This is compared to the measured bending stress, calculated by taking the half the difference between the peak stresses measured at the two gages on opposite sides of the tower in line with the load. The measurements from strain gages CH_9, 10, 11, and 12 are used for this calculation. For north-south loading, the bending stress is equal to (CH_10 – CH_9)/2. For east-west loading, the bending stress is equal to (CH_12 – CH_11)/2. It can be seen that there is good agreement between the calculated and measured bending stress. Similar agreement was observed in the east-west pull tests.

Table 2 contains the peak measured stresses in all strain gages on the retrofitted tower (with reinforcing jacket) for the pull tests in the north-south direction. Again, it can be seen that there is good agreement between the calculated and measured bending stress.

In the jacket-reinforced tower, it can be seen that, at the base of the tower section (CH_1_IN, CH_2_IN, CH_3_IN, and CH_4_IN), significant stress levels remain in the tower, though they are markedly reduced from the as-built case.

Table 1. As-built tower, stresses measured during static north-south pull tests

Data Channel	Location	NS Pull				Unit
		Pull_1	Pull_2	Pull_3	Pull_4	
CH_1_IN	N face tower at base	-2.60	-3.39	-6.40	-6.07	ksi
CH_2_IN	S face tower at base	2.23	3.28	6.53	6.51	ksi
CH_3_IN	W face tower at base	-0.19	-0.17	0.15	0.05	ksi
CH_4_IN	E face tower at base	-0.09	0.12	-0.12	0.29	ksi
CH_5_IN	N face tower 12” above base	-1.41	-1.74	-3.25	-2.97	ksi
CH_6_IN	S face tower 12” above base	1.24	1.95	3.86	3.89	ksi
CH_7_IN	N face tower 53.5” above base	-1.43	-1.77	-3.35	-3.02	ksi
CH_8_IN	S face tower 53.5” above base	1.16	1.84	3.62	3.75	ksi
CH_13	NW anchor rod	-0.81	-0.98	-1.60	-1.51	ksi
CH_14	NW anchor rod	-1.72	-2.22	-4.01	-3.87	ksi
CH_15	N anchor rod	-1.21	-1.46	-2.83	-2.52	ksi
CH_16	N anchor rod	-2.14	-2.79	-5.39	-5.07	ksi
CH_9	N face tower 6’ above base	-1.30	-1.59	-3.09	-2.72	ksi
CH_10	S face tower 6’ above base	1.02	1.57	3.13	3.17	ksi
CH_11	W face tower 6’ above base	-0.11	-0.07	0.18	0.21	ksi
CH_12	E face tower 6’ above base	-0.14	0.09	-0.04	0.27	ksi
	Load	0.78	1.06	1.83	2.02	kips
	Moment @ 6' above baseplate	19.82	26.82	46.44	51.40	k-ft
	Calculated Stress	1.18	1.59	2.76	3.05	ksi
	Measured Bending Stress	1.16	1.58	3.11	2.94	ksi
	Meas./Calc'd Stress	0.99	0.99	1.13	0.96	ksi

H= 35’, L = 63.6’

Table 2. Retrofitted tower, stresses measured during static north-south pull tests

Data Channel	Location	NS Pull Pluck_A	NS Pull Pluck_B	NS Pull Pluck_C	Units
CH_1_IN	N face tower at base	-2.48	-0.99	-1.70	ksi
CH_2_IN	S face tower at base	2.93	4.34	3.05	ksi
CH_3_IN	W face tower at base	0.61	2.01	1.01	ksi
CH_4_IN	E face tower at base	0.49	1.97	1.02	ksi
CH_5_IN	N face tower 12” above base	-0.66	0.76	-0.08	ksi
CH_6_IN	S face tower 12” above base	1.62	3.11	1.99	ksi
CH_13	NW anchor rod	-2.59	-1.01	-1.86	ksi
CH_14	NW anchor rod	-2.56	-1.54	-2.23	ksi
CH_15	N anchor rod	-1.10	0.36	-0.50	ksi
CH_16	N anchor rod	-5.44	-4.39	-4.67	ksi
CH_9	N face tower 6’ above base	-2.56	-1.06	-1.93	ksi
CH_10	S face tower 6’ above base	3.05	4.92	3.47	ksi
CH_11	W face tower 6’ above base	0.53	2.09	1.00	ksi
CH_12	E face tower 6’ above base	0.37	2.01	0.97	ksi
CH_3_OUT	W face jacket at base	0.41	1.64	0.70	ksi
CH_4_OUT	E face jacket at base	0.38	1.77	0.90	ksi
CH_17	WNW face jacket at base on bend	-1.26	0.08	-0.87	ksi
CH_18	WNW face jacket at base	-1.22	-0.29	-1.03	ksi
CH_19	SSW face jacket at base	3.11	4.74	3.36	ksi
CH_20	W face jacket 21.25” above base	0.30	1.78	0.69	ksi
CH_21	E face jacket 21.25” above base	0.18	1.46	0.55	ksi
CH_22	W face jacket 53.5” above base	0.29	1.36	0.59	ksi
CH_23	E face jacket 53.5” above base	0.36	2.02	0.70	ksi
	Load	1.95	1.98	1.89	kips
	Moment @ 6' above baseplate	49.56	50.22	47.99	k-ft
	Calculated Stress	2.94	2.98	2.85	ksi
	Measured Bending Stress	2.81	2.99	2.70	ksi
	Meas./Calc'd Stress	0.95	1.00	0.95	ksi

H= 35’, L = 63.6’

PRELIMINARY RESULTS OF LONG-TERM MONITORING

Stress-range histograms were recorded every ten minutes using the rainflow cycle counting algorithm (Miner 1945). These histograms were generated for nine selected strain gages. A stress-range histogram is basically a tally of stress cycles of predetermined ranges. Every ten minutes, the data acquisition system updates the tally. A fatigue evaluation of the towers was performed using the stress-range histograms, which were truncated at a level equal to approximately 1/4 of the constant amplitude fatigue limit (CAFL) of the detail in question per AASHTO. That is, all cycles with stress ranges less than the truncation level were removed from the histogram prior to calculation of the effective stress.

In addition to the stress-range histograms, stress time history data were recorded when predefined trigger events occurred. These trigger events occurred when wind speed and stress events at selected locations exceeded various levels. When a trigger event was detected, data were recorded from all sensors for a predefined length of time. The stress time history data were used to assess the validity of large stress-range cycles recorded in the stress-range histograms and to understand the wind phenomena that caused them.

Finally, average wind data were recorded continuously, on five-minute intervals. During each interval, the data logger records the average and maximum wind speed and the average wind speed.

A total of nine strain gages are currently being monitored, as identified in Table 3 and as shown on the instrumentation plan Figure 1. A fatigue life estimate was performed for each of the gages listed in Table 3, using the stress-range histograms developed for the period between November 8, 2006, through February 13, 2007.

Table 3. Summary of strain gages for which stress-range histograms were developed

Strain gage	Location
CH_1	N side on tower; at base weld centered on face
CH_3IN	W side on tower; at base weld centered on face
CH_3OUT	W side on jacket; at base weld centered on face
CH_9	N side on tower; 6’ above baseplate centered on face
CH_11	W side on tower; 6’ above baseplate centered on face
CH_14	NW anchor rod
CH_16	N anchor rod
CH_17	W side on jacket; at base weld on bend
CH_23	E side on jacket, 4’-5 1/2” above baseplate

The strain gages on the tower beneath the reinforcing jacket adjacent to the full-penetration groove weld (strain gages CH_1 and CH_3IN) are considered Category E, due to the fact that the backing bar is not welded to the base plate by a full-penetration weld. (It is noted that all fatigue categories cited herein are per current AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Supports). The strain gages on the reinforcing jacket adjacent to the base weld (CH_3OUT and CH_17) are considered Category E as a full-penetration groove weld. The strain gages on the anchor rods (CH_14 and CH_16) are considered Category D based on the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Supports, Table 11-2. Strain gage CH_23 on the reinforcing jacket some distance above the baseplate is considered Category B as a bolted connection. Strain gages above the jacket on the tower (CH_9 and CH_11) are also considered Category B as a bolted connection.

Table 4. Summary of fatigue life calculations (November 8, 2006, through February 13, 2007)

Strain gage	Assumed category (CAFL)	S_{Rmax} (ksi)	Cycles > CAFL #	Cycles > CAFL %	S_{Reff} (ksi)	Cycles/ day	Remaining life (years)
CH_1	E' (2.6)	8.5	10,849	0.21%	1.0	54,288	17
CH_3IN	E' (2.6)	9.0	9,738	0.15%	1.0	65,225	15
CH_3OUT	E (4.5)	11.0	574	0.02%	1.6	29,732	18
CH_9	B (16)	13.0	0	0.00%	6.0	27	infinite
CH_11	B (16)	12.5	0	0.00%	5.8	25	infinite
CH_14	D (7)	21.6*	621	0.04%	2.2	16,465	30
CH_16	D (7)	16.0	2,796	0.11%	2.4	25,651	11
CH_17	E (4.5)	25.8*	58,238	0.83%	2.2	72,665	4
CH_23	B (16)	8.0	0	0.00%	5.0	1	infinite

*Maximum stress cycle determined from review of time-history data

Review of the stress-range histograms revealed that very high-stress cycles were recorded in a number of the strain gages, in particular gages CH_14 and CH_16 on the anchor rods and CH_17 at the bend line on the west face of the reinforcing jacket. The effect of these higher stress cycles are reflected in the relatively low fatigue life estimates presented in Table 4. However, it should be noted that the strain gages at the base were installed adjacent to the weld to at the baseplate weld. As a result, the strain gages in their current position are located in an area of high stress gradient. Since the AASHTO S-N curve is calibrated to be used with nominal stresses, the life estimates presented in Table 4 will likely underestimate the actual fatigue life. (It is noted that the fatigue categories used for this evaluation will be updated based on the results of the Iowa Department of Transportation –funded laboratory testing of retrofitted high-mast towers, currently underway at Purdue University.)

The highest stress cycles recorded during the monitoring period were the result of a wind storm that occurred on November 10, 2006, at approximately 7:10 a.m. The average wind speed at the time was only approximately 20 mph, with gusts up to 33 mph. The winds were from the north (note, the data in Figure 3 is an uncorrected wind direction; 58 degrees must be added to each measurement to correct to the compass directions). The stress cycles resulting from this wind event caused a second peak in the stress-range histograms at many locations where gages were installed.

Though the stress magnitudes and corresponding fatigue life estimates may be alarming, the frequency of occurrence of this wind event must be evaluated through continued monitoring. It must be noted that the fatigue life estimates are based on the assumption that the number and magnitude of stress cycles measured during the three-month period presented are the same for all three-month periods for the remainder of the tower life. This is most likely not the case, as this event may have been a rare occurrence. This will be evaluated using the remaining long-term data.

At this time, the authors believe that the tower was excited by an initial gust (see Figure 3) that caused the tower to begin vibrating in its first mode. The first mode frequency for this pole is approximately 0.3 Hz (or a period of vibration of 3.3 seconds). This means that the time it takes for the pole to start at a positive peak stress, vibrate to peak negative stress, and return to peak positive stress is 3.3 seconds.

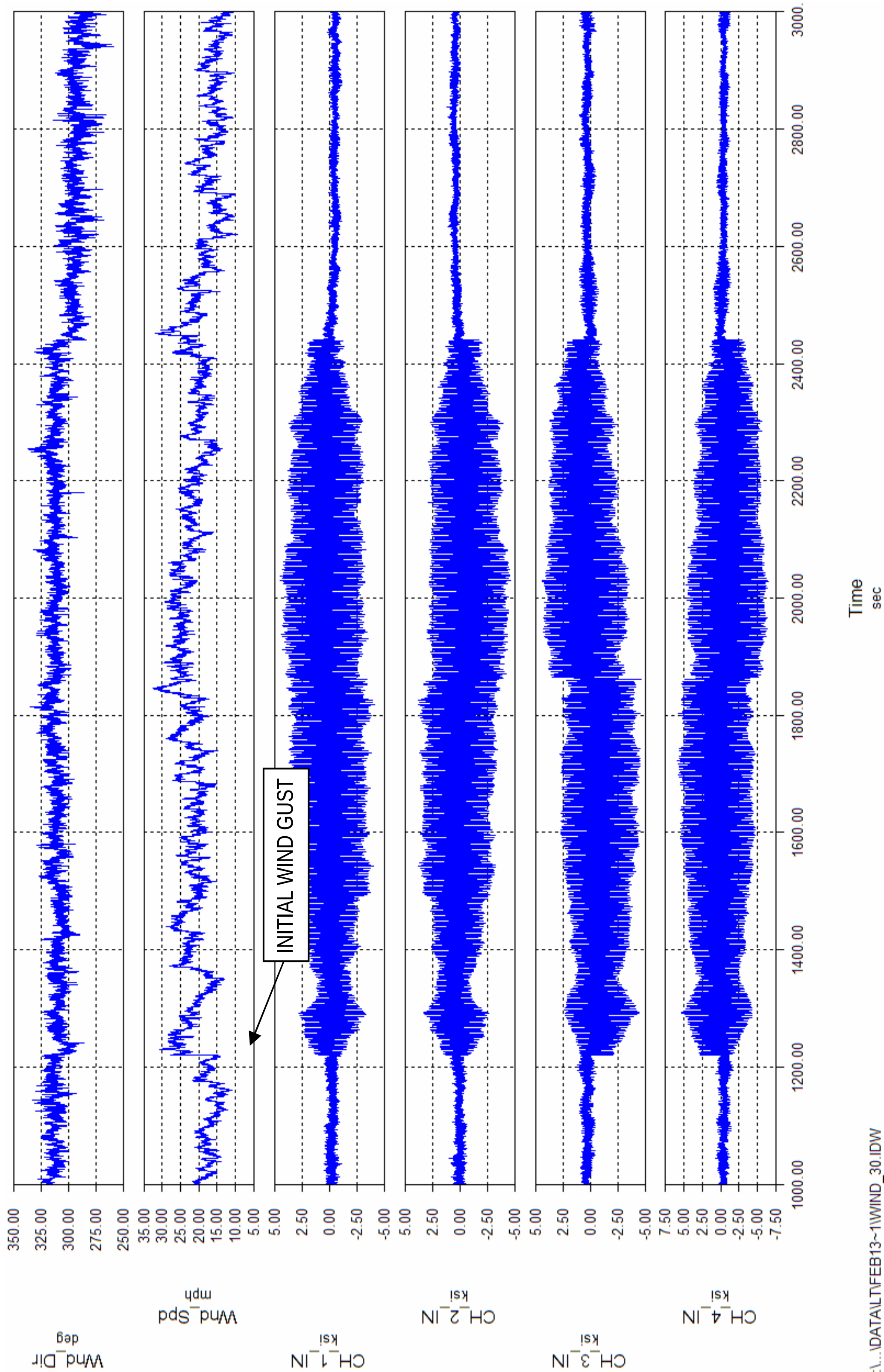

Figure 3. High-stress event, November 10, 2006, 7:10 a.m.

This observed vibration is most likely not vortex shedding, since the critical velocity for vortex shedding in the first mode is significantly less than the observed 20 mph present during the event. The wind gusts appear to have been in phase with the first-mode natural frequency of the tower, causing the magnitude of vibration to increase. This will be studied further as part of this project to fully assess the nature of this phenomena and its effect on the fatigue performance of the retrofitted and unretrofitted high-mast towers.

Figure 4 contains a close-up stress time history for the four strain gages on the tower above the reinforcing jacket (6 ft. above the base plate). It can be seen that the time it takes the stress to vary from peak positive to peak negative and back to peak positive stress is approximately three seconds. Furthermore, strain gages CH_9 and CH_11 are in phase. Strain gages CH_10 and CH_12 are also in phase with each other, but collectively out of phase by 180 degrees with CH_9 and CH_11. Note that the magnitudes are roughly equal. This indicates that the neutral axis of bending lies on a 45 degree line running between CH_9 and CH_11 on one side and CH_10 and CH_12 on the other (i.e., running northeast-southwest). A similar plot for the four gages on the tower at the base weld (beneath the reinforcing jacket) is shown in Figure 5. It can be seen that the same direction of bending is causing the measured stresses.

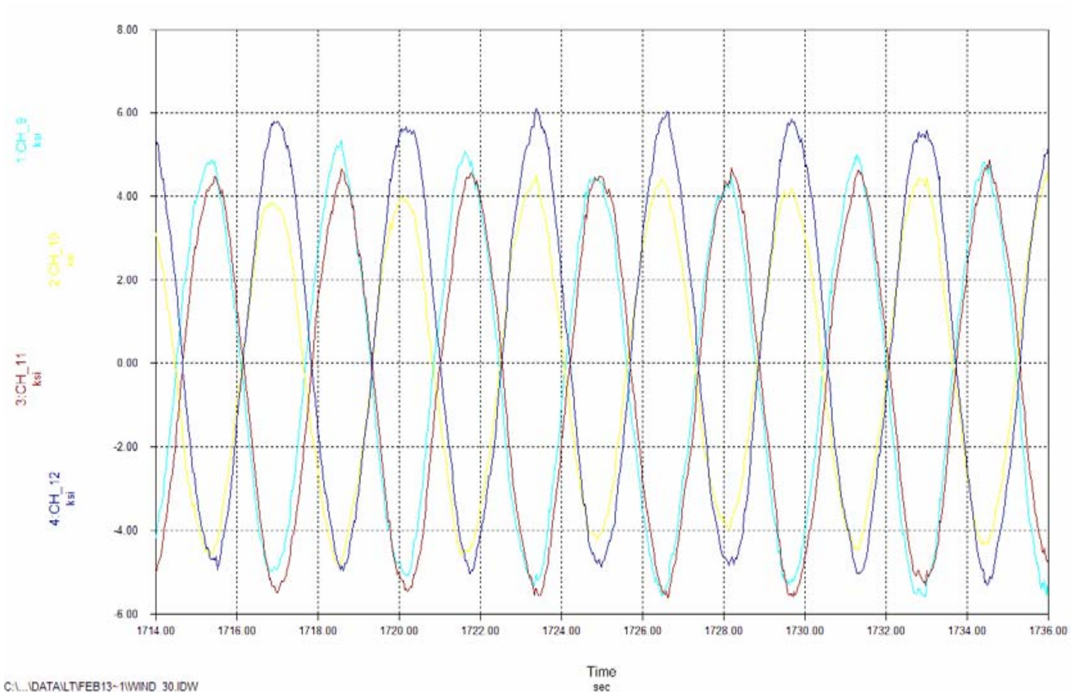

Figure 4. High-stress event, November 10, 2006, 7:10 a.m.; strain gages six ft. above the baseplate

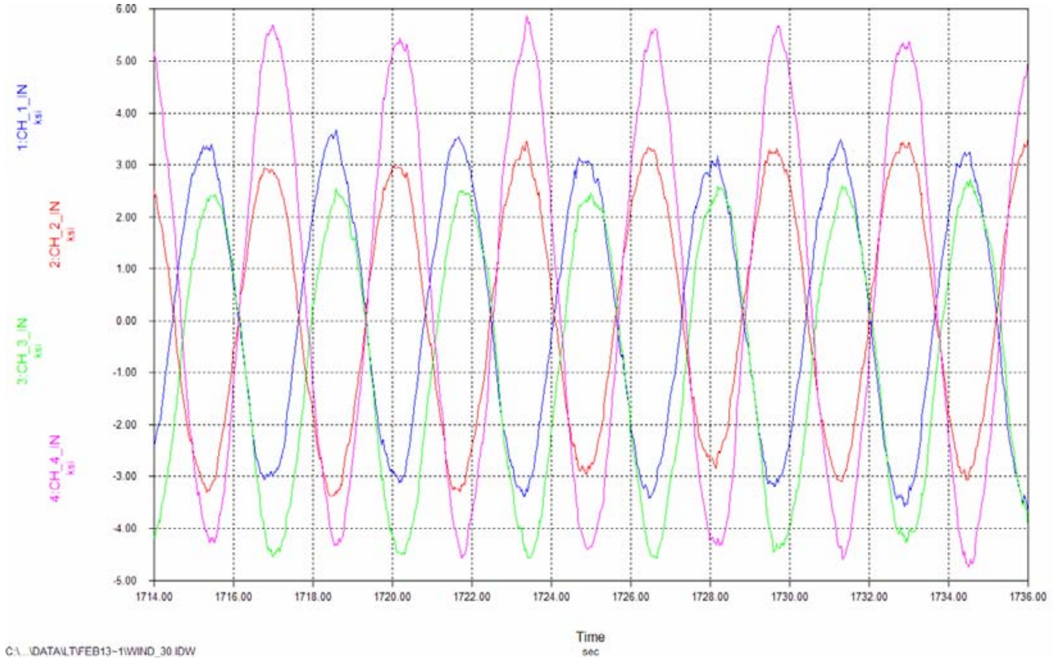

Figure 5. High-stress event, November 10, 2006, 7:10 a.m.; strain gages on the tower at the base weld

FIELD INSPECTION OF HIGH-MAST LIGHTING TOWERS

Based on the results of this study and the observed causes of other failures of high-mast lighting towers, it is clear that these structures should be regularly inspected. There is no doubt that these are fracture-critical structures and that collapse can be extremely hazardous. Below are some recommendations related to the inspection of these towers:

- The bottom sections of high-mast towers should be inspected visually at the base weld and handhole on a regular interval. The interval between inspections should be related to the susceptibility of the structure to fatigue cracking. This susceptibility can be determined by evaluating the pole using current specifications. Recent inspection experience by several owners indicates that cracks in weathering steel towers can be very difficult to detect visually. The use of magnifying glasses, grinders, and magnetic particle/dye penetrant testing will likely be required to ensure a thorough inspection.
- The risk of anchor rod fatigue is also critical. If it is established that there is a potential for anchor rod cracking, ultrasonic testing of the anchor rods should be performed during every inspection. In addition, special attention should be given to ensuring that both the upper and lower anchor nuts are tight. It would be prudent to check existing structures using a torque wrench or other method.
- Weathering steel poles are particularly sensitive to the collection of debris inside and outside of the tube. Inspections should also ensure that leaves, soil, or other debris are not building up on the outside of the pole and that there is ample clear space around the base. Inspections of the inside of the base section must also be conducted to ensure corrosion is not taking place on the inside of the tube wall. Furthermore, inspections must also remove any debris buildup from the inside of the pole. Removal of grout between the base plate and the footing is beneficial in helping keep the area dry and free of debris. Special attention must be given to section loss or any poles located adjacent to the highway that may be susceptible to salt splash.

CONCLUSIONS

The following preliminary conclusions are made:

1. Static pull tests were performed on the high-mast tower in both the as-built and jacket-reinforced condition. The stresses in the tower were reduced by the presence of the jacket, but not eliminated, and in fact they are still significant.
2. Through the long-term monitoring, which continues, a large number of high-amplitude stress cycles were measured in the tower and reinforcing jacket. Initial fatigue life estimates are believed to be conservative due to the reasons cited in this paper. Revised detail categorization will be based on the results of the ongoing laboratory fatigue investigation of high-mast towers at Purdue University.
3. During the monitoring, an unusual and likely infrequent wind event that occurred in November 2006 induced most of these high-amplitude stress cycles. This type of behavior was not observed during the long-term monitoring of the same tower conducted in 2004–2005. It is surprising that the peak wind speed recorded during this event was only 33 mph. However, the frequency of the in-line wind gusting appears to have matched the natural frequency of the tower, thus magnifying the vibration amplitudes. The frequency of occurrence of this type of wind event must be evaluated. (Currently, the presence of these cycles in the spectra will skew the life estimates if the event rarely occurs.)

ACKNOWLEDGEMENTS

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Self-Centering Bridge Piers with Structural Fuses

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ABSTRACT

An innovative structural system for pier columns is currently being investigated through a series of laboratory experiments. The columns and connections under investigation are comprised of precast concrete segments to accelerate construction. In addition, some of the columns being investigated employ elastic elements to self-center the columns against lateral loads and structural fuses to control large lateral deflections and expedite repair in the event of a catastrophic loading event.

At the time of publication, two cantilever columns with varying component materials and connection details have been tested in the laboratory and two more are in preparation for testing. The columns are subjected to axial and cyclic, quasi-static lateral loads. After sustaining significant damage, the self-centering columns are repaired by replacing the structural fuses and retested to failure to investigate the effectiveness of the repair.

Of the columns tested to date, the first with a socket connection at the base and no intermediate joints or post-tensioning behaved similarly to a conventional concrete column as expected. The second, a segmented column tested with elastic post-tensioning and structural fuses, experienced a premature failure due to cracking of a weld at a steel collar and subsequent bolt pull-out. Alternate detailing to avoid this failure mechanism is planned for the remaining tests.

Key words: post tensioning—precast concrete columns—rapid bridge construction—structural fuses

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PROBLEM STATEMENT

The objective of this research is to accelerate bridge pier construction through the use of precast columns in order to reduce construction costs, decrease traffic delays, improve work zone safety, and minimize environmental impacts.

Furthermore, it aims to develop a pier system that could endure an extreme loading event such as an impact, severe wind storm, flood, blast, or earthquake. The pier should be able to sustain large deformations, be tough and durable, and be easily repaired.

Precast substructures have been used around the country with varying degrees of success over the past two decades. A concise overview of the existing technology is given in "State-of-the-Art Report on Precast Concrete Systems for Rapid Construction of Bridges" by Hieber et al. (2005) The system examined in this research offers a different design approach and details that have the potential to reduce the construction time and improve structural performance. The basic proposed pier assembly is illustrated in Figure 1. Key features include steel collars at the ends of segments (Figure 2), external reinforcement of segment joints which have bolted connections (Figure 3), and bearing plates between segments to avoid labor-intensive grouting procedures (Figure 4).

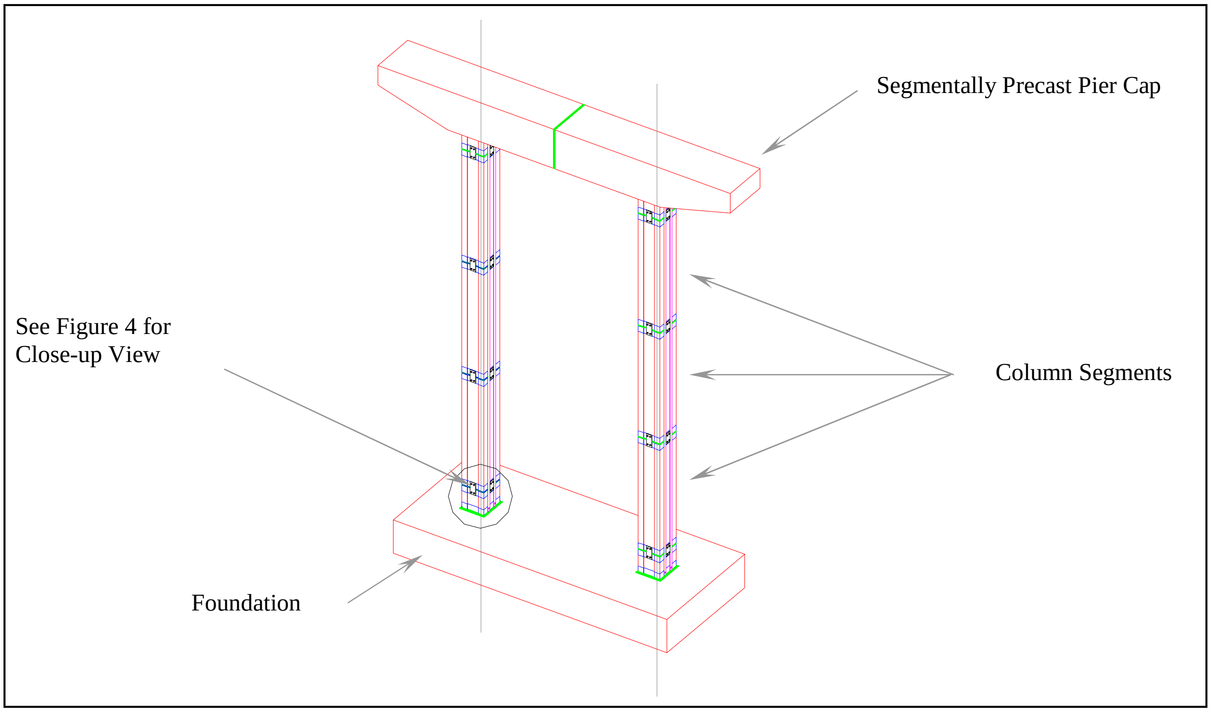


Figure 1. Basic pier assembly (isometric view)

The steel collar assembly to be cast at the ends of each column segment, as illustrated in Figure 2, serves three purposes: reinforcement of the concrete corners to prevent damage during shipping and erection, confinement of the concrete at the ends of the segments to provide additional concrete strength and ductility, and a convenient and aesthetically pleasing means for attaching the exterior reinforcement plates.

A single column segment is illustrated in Figure 3 with steel collars at each end. Both the connections between the foundation and column shaft and the connection between column segments are illustrated in Figure 4. The first (lowest) column segment fits into a socket formed in a cast-in-place or precast pile cap or a spread footing. The annular space at this location is filled with a flowable epoxy grout. The column segments are separated by bearing plates.

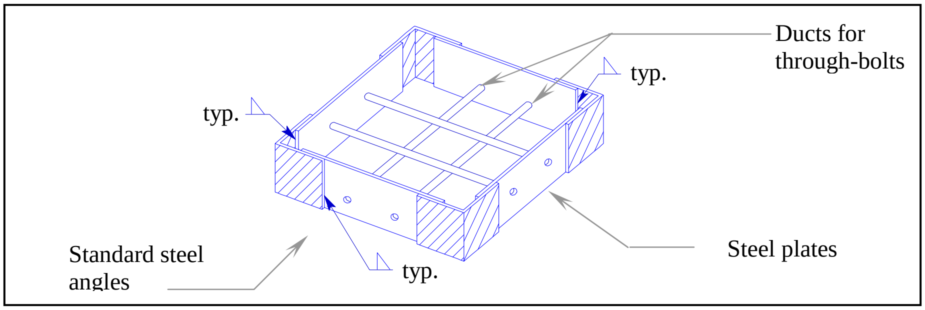


Figure 2. Steel collars at segment ends

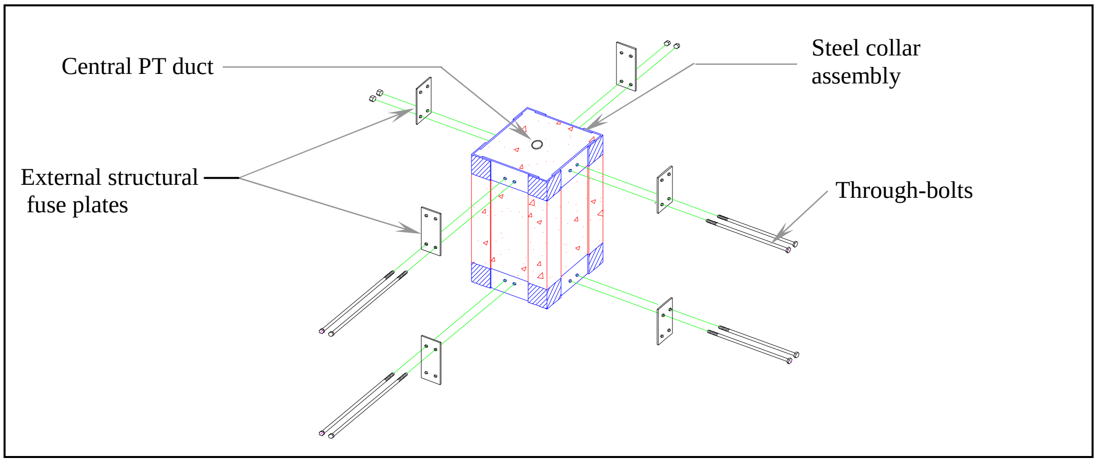


Figure 3. Single column segment with external connectors

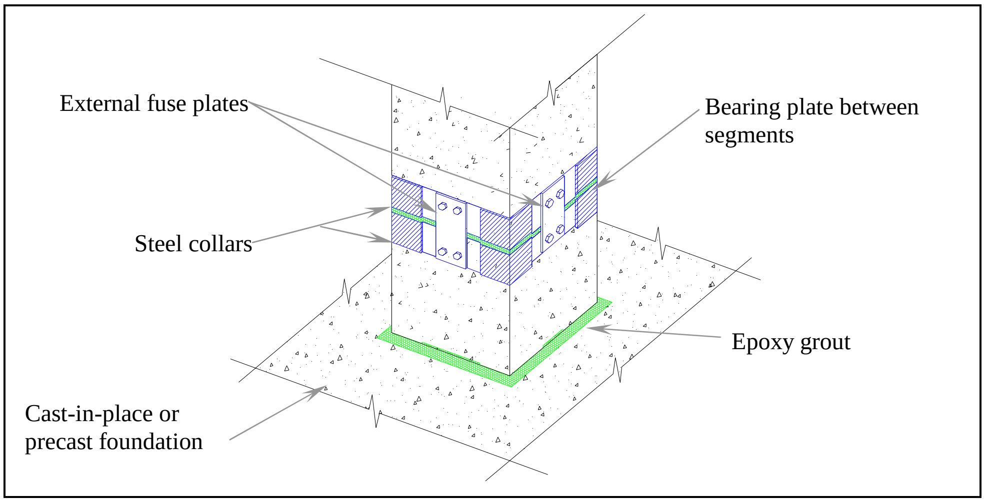


Figure 4. Close-up of typical joint

A possible construction sequence is as follows. Once piles are driven, a precast pile cap is placed or cast with sockets to receive precast column segments. A vertical alignment rod is threaded into an anchor cast into the bottom of the socket, and the first column segment is lowered into place. Once shimmed and leveled, flowable epoxy grout is poured into the annular socket space. A bearing plate is placed at the top of the last segment placed, and the next segment is subsequently lowered over the alignment rod to rest on the bearing plate. The external fuse plates previously described are then bolted into position. This procedure continues until all but the uppermost column segments are in place. The uppermost column segment is epoxied into the pier cap socket and the unit is lowered onto the columns. Once all pieces are in place, cap segments are connected, external plates are secured by fully tensioning the through-bolts, and nuts are threaded onto the top of the alignment rods to post-tension the column. The pier would then be ready to receive the superstructure. This research is placing special emphasis on the connection designs to simplify construction. Although Figures 1–4 schematically illustrate square columns, the connection details could be developed for rectangular or circular columns as well.

The segmentally precast pier provides the economic and aesthetic advantages usually ascribed to any precast concrete system. Because the concrete is cast at a plant rather than in the field, environmental conditions that are crucial to freshly placed concrete may be more closely monitored and controlled. The usual result is higher quality concrete that is more durable over the life of a structure. The precast pieces may be cast early in the project schedule and then be rapidly assembled in the field even during temperature extremes that normally pose problems for cast-in-place structures. Architectural finishes may also be expediently applied in the plant providing a wider range of appearances for the completed structure. By casting the pier columns in segments, shipping and handling costs will be reduced and smaller, lighter equipment will be required for field assembly.

In addition to the economic advantages of accelerated construction, the system can be designed to provide advantages in structural performance under extreme lateral loads arising from events such as impacts, severe winds, floods, blasts, or earthquakes. This behavior may be achieved by designing the easily replaceable external plates as structural fuse plates. The structural fuse plates would serve to concentrate damage in the fuses themselves while leaving the remainder of the structure relatively undamaged. Continuous elastic elements, such as a post-tensioning rod may also be incorporated in the members to provide a self-centering force and minimize residual deformations. By achieving minimal residual deformations, the bridge may remain in service immediately after an extreme loading event. Full repair of the structure, if required, may then be accomplished simply by replacing the structural fuse elements. It is anticipated that the incremental costs may be minimal because such fuse elements can serve the dual purpose of expediting construction and improving structural performance.

RESEARCH METHODOLOGY

Key Variables

Because the greatest uncertainties in this system are associated with the behavior of the segment joints, cantilever column specimens were designed to focus on the behavior of a single segment joint. Test variables to be investigated include the types of bearing plates, fuse plates, and amount of post-tensioning force applied. Figure 5 shows the variables selected for each test. One column was cast as a single unit to examine the behavior of the socket-type base connection and serve as a basis for comparison with the subsequent jointed columns. Columns 1 and 2 have been tested, and Columns 3 and 4 have not yet been tested.

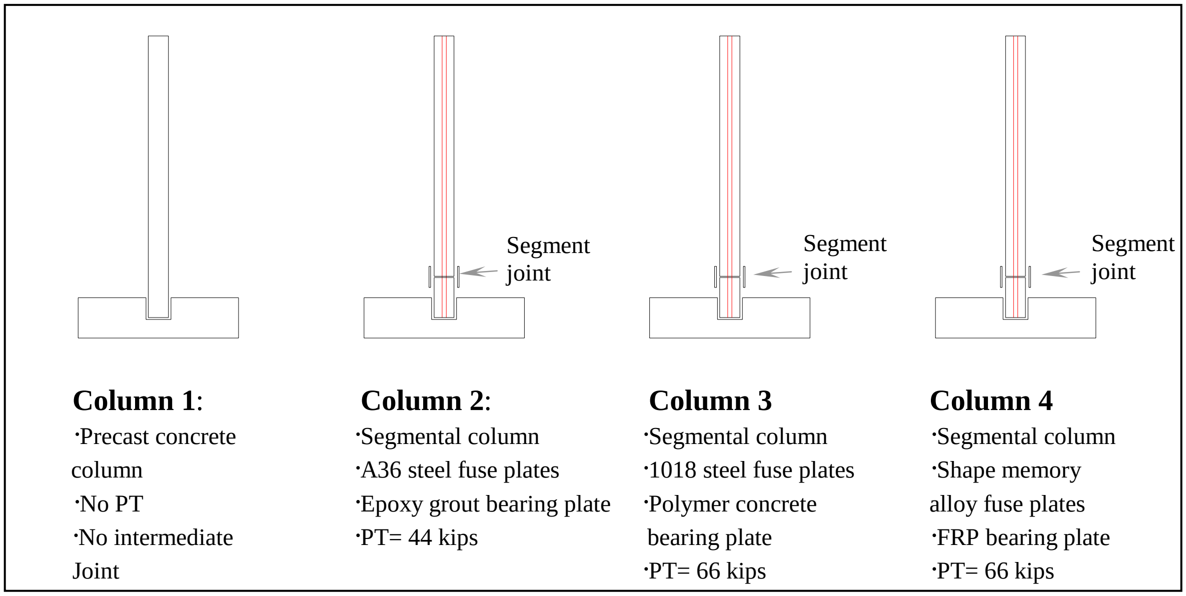


Figure 5. Test columns

Bearing Plates

The bearing plate in the precast pier system is one of the variables being investigated. The bearing plate must be compliant enough to transfer axial stress uniformly between segments but strong enough to withstand high shear, axial, and bending stresses. Several different types of plates were considered with desired properties that included high compressive, tensile, and shear strengths, a modulus of elasticity roughly half that of concrete, high resistance to corrosion, low creep, and relatively low cost. Materials considered were an epoxy grout plate, a polymer concrete plate with a steel reinforcement, a fiber-reinforced polymer (FRP) plate, a lead plate, a neoprene pad, and a cotton duck pad. The following were selected for testing.

- **Epoxy Grout.** Sikadur 32, a high modulus, flowable epoxy was mixed with Grade 37 silica sand in a 1:1 ratio for Column 2. The modulus of elasticity was roughly 1,500 ksi, while the compressive strength of the epoxy-grout was roughly 11.5 ksi. When tested in compression, this material exhibited large plastic strains without cracking.

- **Polymer Concrete.** A steel-reinforced polymer concrete plate is planned in the third column test. In polymer concrete, a high-strength, corrosion-resistant, thermosetting resin acts as the binding agent. Three-eighths in. aggregate will be used in the plate, and a two inch by two in. welded wire grid will be placed in the plate to reinforce against splitting. The modulus of elasticity of polymer concrete is approximately 2,400 ksi, while its compressive strength is approximately 10 ksi.

- **Fiber-Reinforced Polymer.** A carbon fiber-reinforced polymer (CFRP) plate is planned for the fourth column test. An epoxy resin acts as the binding agent for CFRP, with a 90-90 fiber orientation. The elastic modulus for the CFRP bearing plate perpendicular to the fibers is expected to be approximately 2,000 ksi.

Fuse Plates

The fuse plate in the precast pier system is another variable under investigation. Desired properties include low yield stress, large plastic strain region, high resistance to corrosion, large ultimate elongation (~20 to 25%), high toughness, and low cost. Materials being considered include A36 steel, 1018 carbon steel specifically manufactured to yield between 30 and 36 ksi, shape memory alloy, A242 steel, and A588 steel. The following were selected for testing.

- **A36 Steel.** A36 steel plates were used in the second test column. From laboratory tests, the plates were determined to have a yield stress of 42 ksi with an ultimate elongation near 20%.

- **1018 Carbon Steel.** 1018 carbon steel will be used for the third test column. It has a yield stress of 30 to 36 ksi and an ultimate elongation similar to that of the A36 plates.

- **Shape Memory Alloy (SMA).** Shape memory alloy will be used for the fourth test column. It has a yield stress plateau similar to A36 steel, with much less permanent elongation. The yield stress plateau is approximately 50 ksi. Although more expensive than conventional steel, the SMA plates provide the advantages of helping to self-center the column and are less likely to require replacement.

Post-tensioning

The post-tensioning force applied to the columns is the final variable being investigated. The forces applied to each test specimen were noted previously in Figure 5. Higher initial PT force increases the lateral stiffness of the column, increases self-centering force, and reduces cracking in the concrete at the expense of axial capacity of the column.

Specimen Design and Construction

Laboratory size constraints restricted the height of the columns, which in turn established column gross cross section dimensions. Fuse plate and bearing plate dimensions and material properties were then selected. Design dead load and post-tensioning forces were also determined based on these limiting criteria. For simplicity of design and analysis, fuse plates were only used on the tension and compression sides of the columns.

An analytical approach using reinforced concrete design techniques was used for the design and theoretical analysis of the columns and foundation blocks. Force and moment equilibrium and strain compatibility were used to establish predicted values for load-deflection behavior, stresses, and strains in various elements of the columns.

Four identical foundation blocks were cast to provide a base connection for each column specimen. The foundation blocks were designed to restrict rotation at the base of the columns. A socket was used to provide a connection between the foundation block and the test columns. Five ksi concrete and mild reinforcing steel were used for the blocks. Foundation block dimensions are shown in Figure 6.

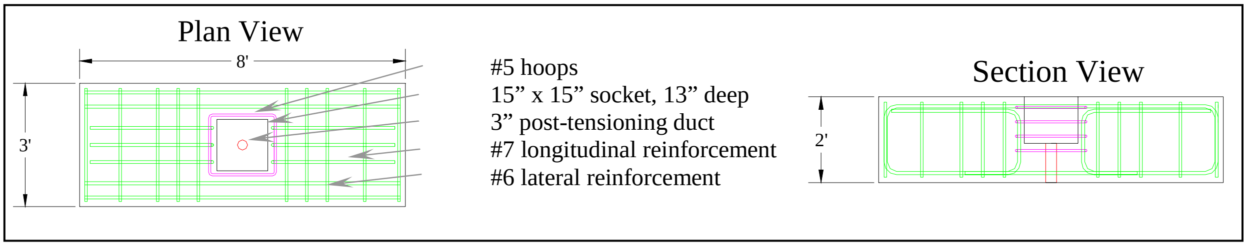


Figure 6. Foundation blocks

Four test columns were cast. The first column was cast as one continuous reinforced concrete element 14 ft. in length. Six #7 mild reinforcing bars were used for longitudinal reinforcement and #3 mild reinforcing bars were used as lateral ties spaced on 12 in. centers. The three remaining columns were cast in two segments. Each column consisted of a two ft. reinforced concrete segment and a 12 ft. reinforced concrete segment. Reinforcement for all segments consisted #7 mild steel bars spaced as in the first column.

The segments of the jointed columns were cast with steel collars on one end. The collars were fabricated with one in. diameter through-ducts for bolts to connect the fuse plates to both column segments (see Figures 2 and 8).

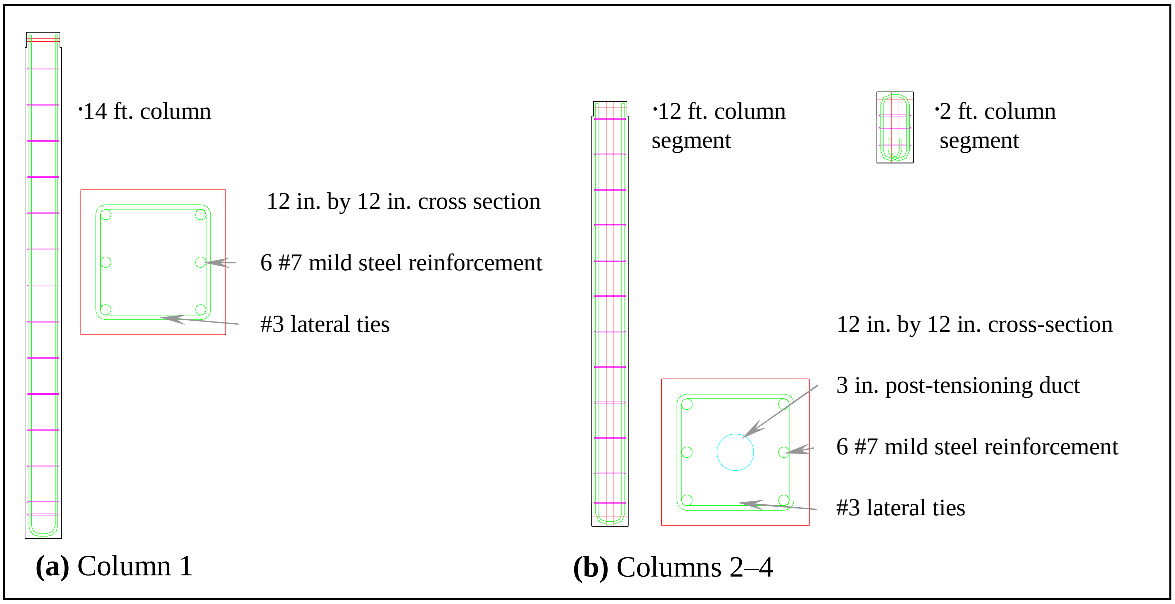


Figure 7. Test columns

Figure 8. Steel collar

Test Setup, Instrumentation, and Procedure

The test was designed to apply a constant, vertical dead load, as well as a cyclic, quasi-static lateral load. Dead weights were hung from a pier cap attached to the top of each column specimen. Each hanging block weighed 18.5 kips, while the pier cap weighed 7 kips for a total of 44 kips. Column base fixity was achieved by grouting the column into the socket in the foundation block and post-tensioning the foundation to the lab strong floor.

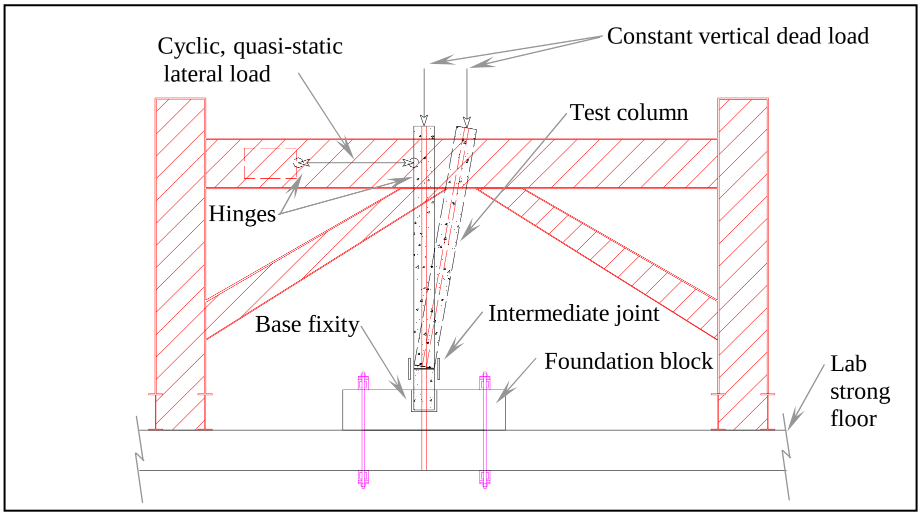


Figure 9. Test setup schematic

For the second test column, fuse plates stabilized the column segments before post-tensioning or dead weight was applied. Oversize holes in the fuse plates allowed for alignment. Once the post-tensioning was applied, the column was stable without the fuse plates. The fuse plates were then set into their final position. Four one in. diameter A490X bolts were used to connect the fuse plates to the column segments. A slip-critical connection was designed for the joint, so direct-tension-indicator washers were used to indicate when the bolts were adequately tensioned. Figure 10 shows the final test setup after the dead weights were applied.

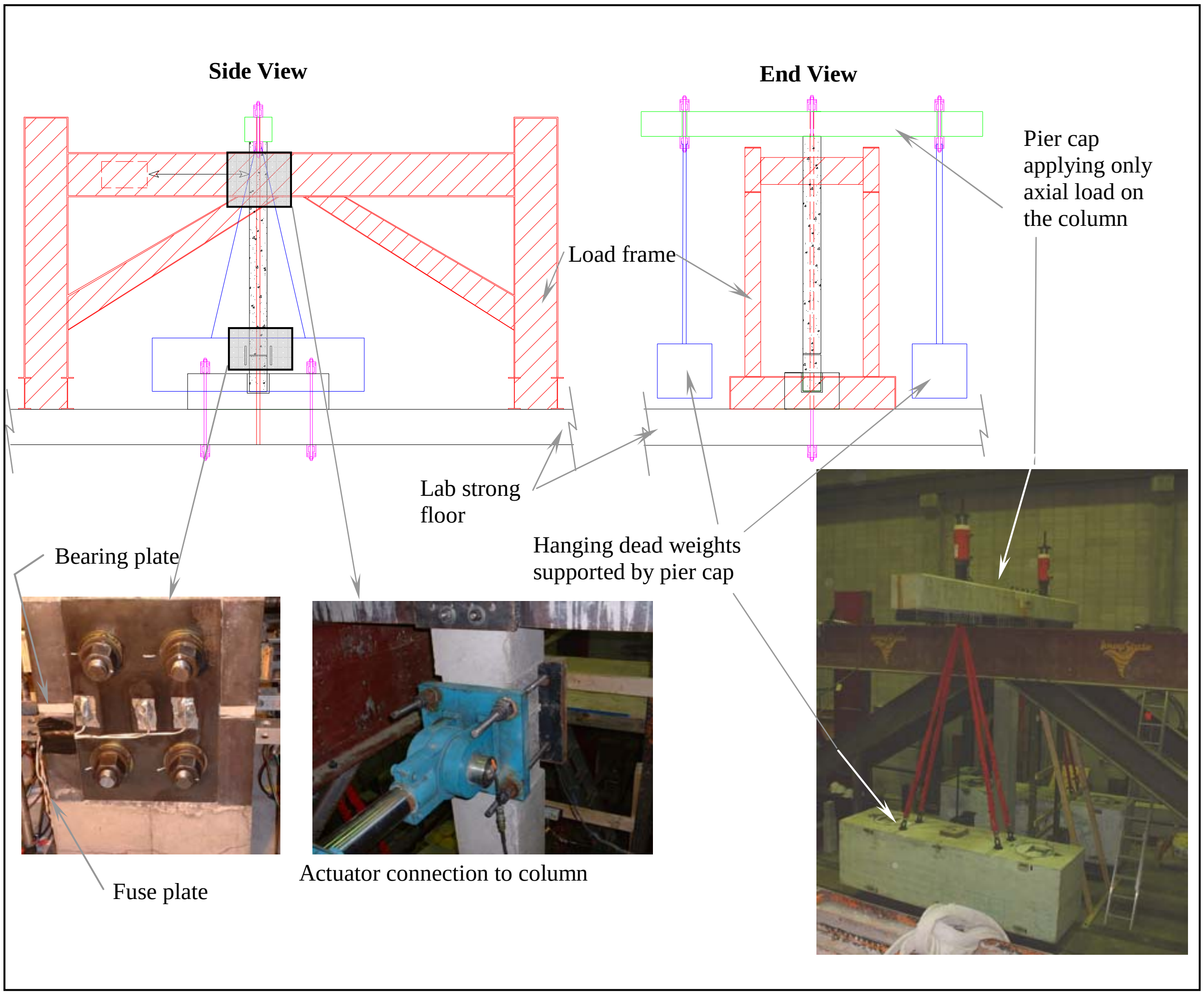


Figure 10. Test setup

Instrumentation

Several types of instrumentation were used to acquire data from the tests. Load cells were used to measure the post-tensioning force and lateral load. Lateral displacement transducers, located 7 in., 15.75 in., 30 in., 60 in., 96 in., and 134 in. (height of actuator) from the base were used to measure column lateral deflections. Tilt of the column was measured by inclinometers located 1 in., 7 in. and 15.75 in. from the base connection. Displacement transducers were used to measure deformations in the segment joint. Four were mounted on both faces perpendicular to the fuse plates, as shown in Figure 11. Strain gauges were bonded on mild reinforcing bars near the base of each column to measure strain. Strain gauges were also used to measure strains in the fuse plates.

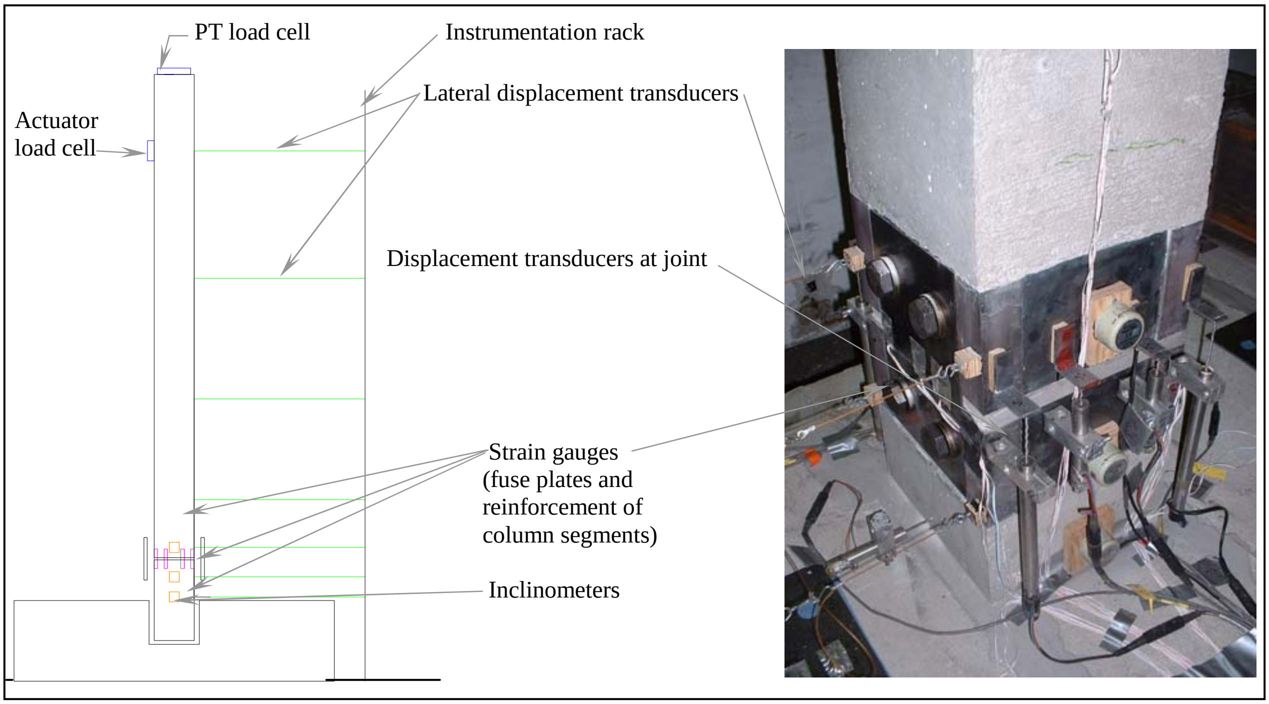


Figure 11. Instrumentation

Testing Procedure

A cyclic, quasi-static lateral load was applied to each test specimen. Loading on the column was displacement-controlled and increased incrementally until failure. Test data were taken at each displacement increment from each of the strain gauges, inclinometers, and displacement transducers using a data acquisition system (DAS). The specimen was examined for damage at each peak displacement of each cycle, and photographs were taken every few cycles. The displacement regimen used in the tests is shown in Figure 12.

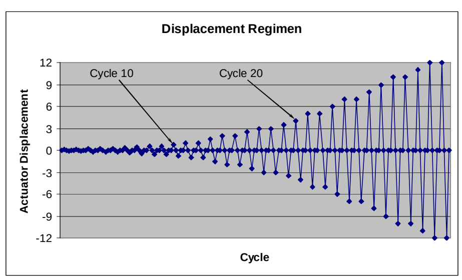


Figure 12. Displacement regimen

The second test column followed a similar procedure with one major modification. Once the column had been displaced 2.5 in., yielding and buckling of the fuse plates was apparent. At this point, the fuse plates were replaced with new plates and the loading regimen was started over and continued until the column failed.

TEST RESULTS

Specimen 1

Lateral loads were applied to a maximum 9 in. displacement in the positive and negative directions. Cracking began near the base and progressed upward beginning on cycle seven (0.40 in. displacement, 2.29 kip load) at an applied base moment of 25.68 ft.-kips. The predicted load for initial cracking was 2.32 kip. The column had the most significant damage on the southwest corner when the mild reinforcement began to buckle. This caused the column to tilt slightly, perpendicular to the axis of loading. When positive displacement was induced, the pier cap began to impinge on the loading frame during the last four cycles causing a significant increase in lateral load. This can be seen in the hysteresis for the Column 1 test in Figure 13.

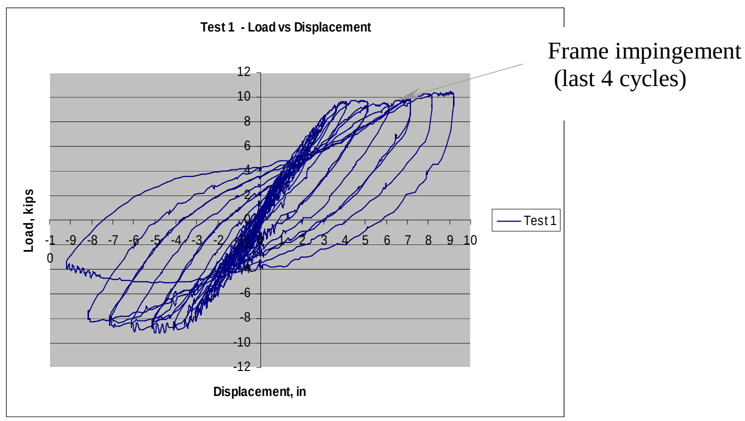


Figure 13. Test 1 hysteresis

Strain gauge data indicated that the longitudinal reinforcement yielded in tension on cycle 15 (displacement 2.5 in., 7.54 kip load) at an applied base moment of 84.20 ft.-kips. The mild reinforcement buckled roughly nine in. above the base. Confinement at the base provided by the socket in the foundation and by a lateral tie three inches above the base prevented buckling from occurring at the base.

LVDT and inclinometer data indicated minimal rotation at the column base. At peak lateral loads the rotation at the base was measured at 0.67 degrees and -0.78 degrees for push and pull cycles, respectively. The epoxy at the socket connection showed no signs of cracking during the test. See Figure 17(a) and (c) for the deflected shape of Specimen 1.

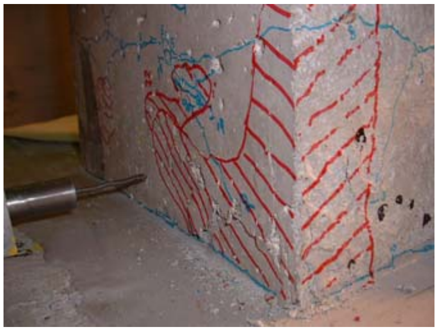
(a)  (b) 

Figure 14. (a) Spall initiation at southwest corner, (b) Column 1 after testing

Test 2 Results

Test 2 consisted of two loading regimens. The first test (referred to as 2a) included cycles to a displacement of 2.5 in., when significant damage had been achieved in the fuse plates. After replacing the fuse plates, the loading regimen restarted and continued until failure of the column (test referred to as 2b).

Cracking at the base of the column began on cycle ten (0.75 in. displacement, 4.45 kip load) at an applied moment of 49.69 ft.-kips. This specimen resisted cracking for three cycles (0.35 in.) more than Column 1 due to the post-tensioning. Minor cracking progressed up the column during the following cycles up to a displacement of 2.5 in., when the plates were replaced.

Yielding of both fuse plates was achieved on cycle 14 (2.0 in. displacement, 6.66 kip load). Noticeable plate buckling also began on cycle 14. At this point, an unforeseen failure mechanism developed when a weld cracked in the collar and the through bolts began to pull out of the upper column segment. After replacing the fuse plates, the loading cycle was repeated up to +/- 6 in. displacement. Only minor cracking occurred in the column during the remainder of the test. Neither segment experienced significant damage or spalling other than bolt pull-out. No mild reinforcement reached the yield stress in the second test.

The hysteresis for test Column 2 (see Figure 15), shows the relatively small residual deflections for the column. Column 2 had an initially stiffer response to loading than Column 1. The evident changes in slope occurred when the fuse plates yielded, and load declined when the collar cracked.

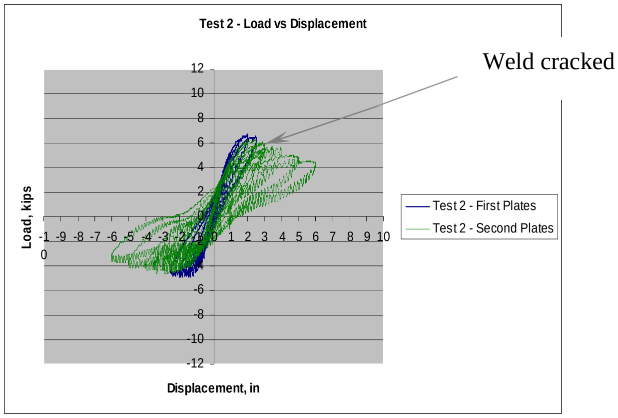


Figure 15. Test 2a and 2b hysteresis

The epoxy grout plate exhibited no evidence of damage until the final cycle, when a small crack was noticed on the northwest corner of the plate. Pinging sounds noticed in the bolts may have indicated some slippage in the slip-critical connection; however, no bearing on the holes was evident after the plates were removed.

In the test on Column 2 there was negligible rotation at its base, as indicated by the LVDT and inclinometer data. These data showed rotation at the base at peak loads of 0.1 degrees and -0.07 degrees for push and pull cycles, respectively. The epoxy at the socket connection showed no signs of cracking during the test. See Figure 17(b) and (d) for the deflected shape of Column 2.

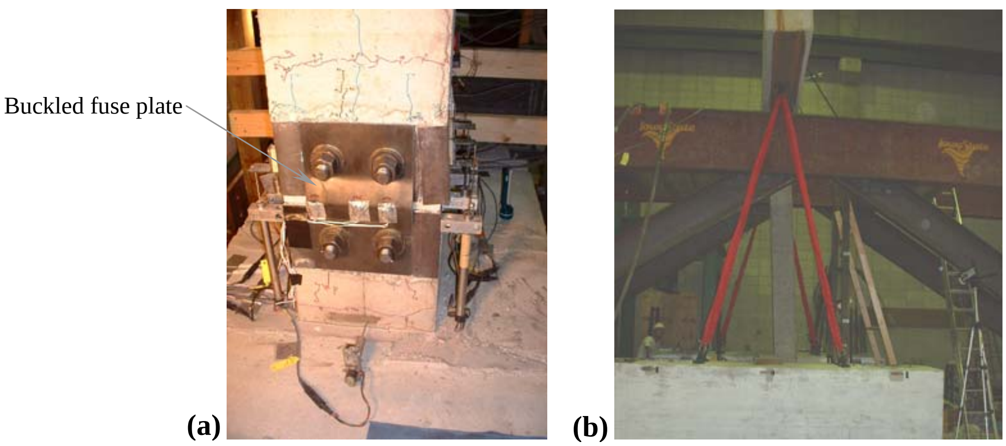


Figure 16. (a) Buckled fuse plate with cracks in column, (b) Displaced column

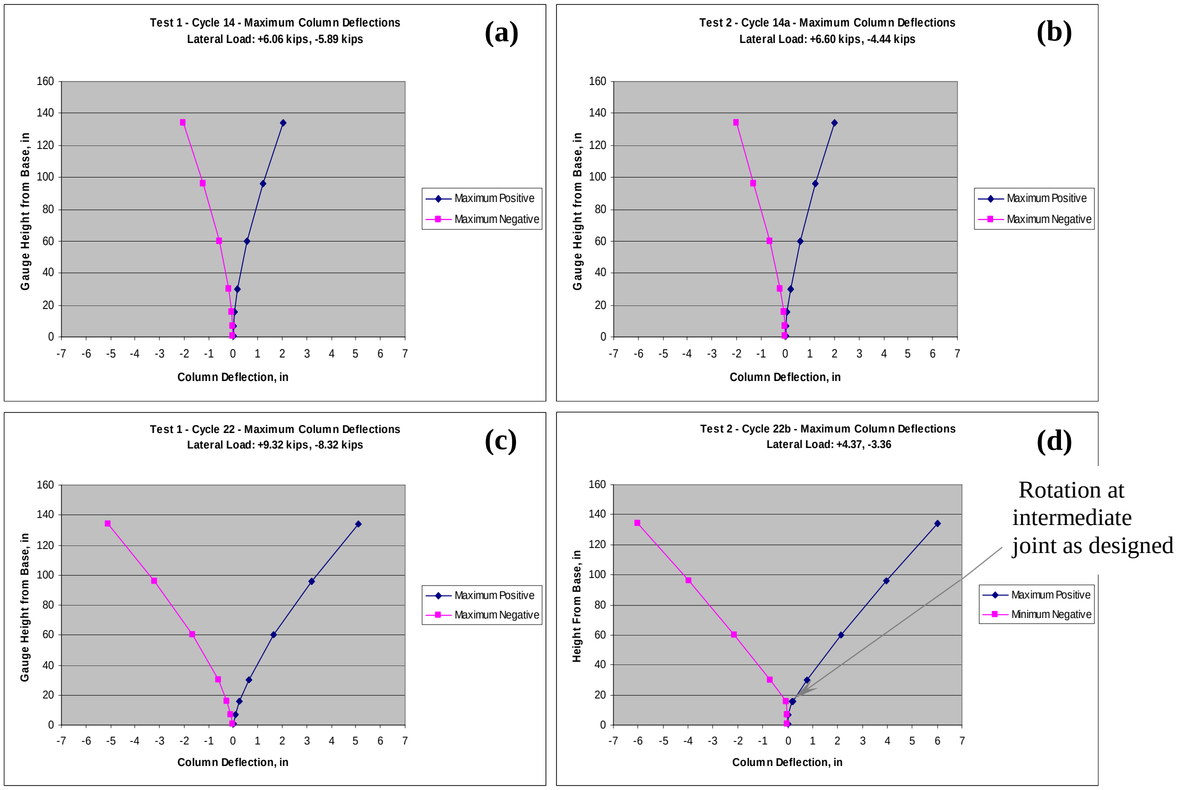


Figure 17. (a) Test 1 deflected shape, cycle 14, (b) Test 2a deflected shape, cycle 14, (c) Test 1 deflected shape, cycle 22, (d) Test 2b deflected shape, cycle 22

SUMMARY

The unjointed Column 1 behaved, as expected, similarly to a conventional concrete column. Predictions of first cracking load, load at yielding of reinforcement, and ultimate lateral load capacity closely matched conventional computations for reinforced concrete columns. Lateral displacements were slightly larger than predicted due to the elastic deformation of the epoxy grout at the socket. The rotations measured at the socket, however, were small, and the connection showed no indication of damage during the test. Furthermore, the socket connection detail was easy to construct with the greatest additional time cost in forming the socket itself.

The Test 2 segmented column displayed much lower lateral load and deflection capacity than Column 1 due to the unforeseen failure of the welded collar and subsequent bolt pull-out. This failure mechanism may be addressed by using perpendicular through-bolts at the collar, specifying heavier welds, or by slight modification of the reinforcement detail at the end of the column. Subsequent tests will address this issue.

Despite the premature failure of the column, initial lateral stiffness was greater and residual deformations smaller than those observed in Column 1. Yielding and buckling of the fuse plates occurred as predicted until cracking of the weld. Cracking in the column was significantly reduced relative to Column 1 since curvature was concentrated in the intermediate joint as designed.

ACKNOWLEDGMENTS

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Effectiveness of Continuous Shoulder Rumble Strips in Reducing Single-Vehicle Ran-Off-Roadway Crashes in Nevada

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ABSTRACT

Single-vehicle ran-off-roadway crashes are of significant concern in Nevada. This paper summarizes some findings of a research project to evaluate of the effectiveness of continuous shoulder rumble strips to reduce such crashes in Nevada. The efforts evaluated safety records on roadways in Nevada on which continuous shoulder rumble strips had been installed. The roadways studied included interstate freeways, U.S. routes, and state routes. Crash data for the period from 1995 to 2003 were used for the analyses. Key data considered in the analyses presented herein include the locations and dates of installation of continuous shoulder rumble strips on roadway segments, crash data, and roadway characteristics. The frequencies and rates of single-vehicle ran-off-roadway crashes were determined for periods before and after the installation of the continuous shoulder rumble strips. Analyses of the data showed that overall the continuous shoulder rumble strips treatment has been effective in reducing the frequency of single-vehicle ran-off-roadway crashes and the corresponding crash rates.

Key words: crash rates—road safety—rumble strips—single-vehicle ran-off-roadway crashes

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INTRODUCTION

Driving safely requires complete attention from the drivers. Fatigue, boredom, and other psychological factors contribute to a lack of attention by drivers. Environmental conditions (e.g., the landscape around the roadway), roadway design characteristics, traffic conditions, and the length (duration) of the drive are factors that affect driver fatigue and boredom. These factors often result in drivers running off the roadway, leading to single-vehicle crashes. Statistics show that a significant proportion of these single-vehicle crashes was fatal (Taylor and Meczkowski 2003). According to the Federal Highway Administration (FHWA), in 2002 ran-off-roadway single-vehicle crashes accounted for more than 120 fatalities and 2,400 injuries in the state of Nevada. The fatalities accounted for more than 30% of total fatalities statewide. Likewise, such injury-related crashes accounted for more than 15% of the total injury crashes statewide. The causes of these crashes include inattentive driving, fatigue, drowsiness, falling asleep, driver distraction, alcohol/drugs, and glare (Haworth et al. 1988). The Nevada Department of Transportation (NDOT) installed continuous shoulder rumble strips along interstate freeways and highways in urban and rural areas of Nevada to alert the drivers and reduce single-vehicle ran-off-roadway crashes. There was a need to evaluate the effectiveness of these continuous shoulder rumble strips in reducing these single-vehicle ran-off-roadway crashes. The types of continuous shoulder strips and the design specifications such as shoulder widths are some factors that might affect the effectiveness of continuous shoulder rumble strips in enhancing safety. This research paper summarizes an evaluation of the effectiveness of continuous shoulder rumble strips in reducing ran-off-roadway single-vehicle crashes in Nevada.

LITERATURE REVIEW

Various types of continuous shoulder rumble strips that have been previously or are currently being used in the United States include the following: milled rumble strips, rolled rumble strips, raised rumble strips, and formed rumble strips. These types of continuous shoulder rumble strips differ in their method of installation, size, shape, and spacing and the noise and vibration they produce (Hickey 1997).

The California Transportation Department (Caltrans) in 1975 added grooved rumble strips to the outside shoulders of a 24-mile section of I-15 next to the Nevada state line. Preliminary results were favorable, and this success led to the installation of shoulder rumble strips in the late 1970s as part of overlaying an additional 130 miles of I-15 and 5 miles of I-40 east of Needles, California. A before-and-after analysis of 1990–1992 crash data on a road network in Utah showed that freeways without shoulder rumble strips experienced a higher rate of run-off-the-road crashes (33.4%) compared to those with shoulder rumble strips (26.9%) (Cheng et al. 1993). An evaluation by Wood (1994) showed that after installation of a sonic nap alert pattern, drift-off-road accidents decreased by 65%. In 1997, Hickey conducted a follow up study to Wood's 1994 observations, in which he added traffic exposure to compare crash volume and crashes per vehicle distance traveled and made adjustments to account for a decline in all crashes during the study years. Perrillo (1998) evaluated the effectiveness of continuous shoulder rumble strips. Griffith (1999) extracted data from California and Illinois and estimated the safety effects of continuous rolled shoulder rumble strips on freeways. The results from this analysis estimated that continuous shoulder rumble strips reduced single-vehicle ran-off-roadway crashes on average by 18.3% on all freeways (with no regard to urban/rural classification) and 21.1% on rural freeways. Chen (1994) performed an analysis of milled, rolled, and formed shoulder rumble strips at 112 locations on two interstate highways in Virginia. The Pennsylvania Department of Transportation researched milled rumble strip patterns that were found to be safe and effective for bicyclists and motorists on nonfreeway roads (Elefteriadou 2000). Spring (2003) conducted a study on the effectiveness of rumble strips in the state of Missouri. The study recommended that the rumble strips only be used on rural roadways and on urban highways in cases where the ran-off-roadway crash history exceeds the acceptable values. Hauer (1997) presents discussions on conducting before-and-after studies for road safety. Building upon the aforementioned items from the literature, this paper reports some findings from a study to evaluate the effectiveness of continuous shoulder rumble strips deployed along roadways in Nevada.

METHODOLOGY

Based on the literature review, metrics were adopted to evaluate the effectiveness of continuous shoulder rumble strips in Nevada. The metrics used in the analyses include those based on individual roadway sections as well as for composite sections. Specific metrics include frequencies and rates of single vehicle ran-off-roadway crashes. A before-and-after study approach was used to evaluate the effectiveness of the rumble strips. The analysis began with the identification of individual roadway segments. The individual segments served as the basic units for the analysis. Data pertaining to the individual sections of a roadway were aggregated to evaluate the roadway. The analysis was based on single-vehicle ran-off-roadway crashes in Nevada recorded by law enforcement agencies.

Data Identification, Collection, and Analysis

The data required to support the analyses consisted of information pertaining to crashes, roadway design, and operational characteristics. Data elements of interest in this regard included the following:

- Road network data (for locations with continuous shoulder rumble strips)
    - Functional classification of roadway
    - Identification of roadway segments
- Rumble strip data
    - Rumble strip installation location
    - Date of rumble strip installation
    - Type of rumble strips
    - Shoulder width
- Single-vehicle ran-off-roadway crash data

NDOT maintains 5,400 centerline miles of highways in the state of Nevada. At the commencement of this study, more than 1,455 centerline miles (both directions accounted for on divided roadways) of these roads had been treated with continuous shoulder rumble strips. These roadway segments consisted of the following types of roadways: interstates, U.S. routes, and state routes. They included roadways distributed across the state of Nevada. The continuous shoulder rumble strips considered for the analyses spanned installations from the year 1998 to 2004. Of these, 1,017 centerline miles had rumble strips installed during the year 1999.

For the purposes of performing the analyses in this study, the roadway sections with continuous shoulder rumble strips were divided into smaller segments. A total of 370 segments were thus identified for the study. The sections with continuous shoulder rumble strips were located on different functional classes or types of roadways, such as interstate freeways, U.S. routes, and state routes. The before-and-after analysis required data for periods prior to and following the construction of continuous shoulder rumble strips on each segment to be evaluated. Among the 370 roadway segments identified for evaluation, 64 segments had their continuous shoulder rumble strips installed in the year 2003 or 2004. Thus, they did not have any after condition data. The remaining 306 roadway segments had at least one year of crash data for each of the before and after conditions with respect to the continuous shoulder rumble strips' installation. These segments, which account for 1,303 centerline miles of roadway, were considered for evaluation and analyses in this study. The segments range in length from less than one mile to several miles, depending on roadway characteristics. A geographic information system layer was developed to identify segments based on the dates of installation of the rumble strips.

Key data pertaining to the road segments with continuous shoulder rumble strips were obtained from NDOT. These included the following: the date of continuous shoulder rumble strips' installation, traffic volumes, crash data, posted speed limits, shoulder width, and section lengths. In order to study the effectiveness of continuous shoulder rumble strips, it was necessary to compile and evaluate data related to single-vehicle ran-off-roadway crashes that occurred on the road segments of interest for time periods before and after the installation of rumble strips. Such crash data for the roadway network in Nevada for the years from 1995 to 2003 were obtained from NDOT. This amounted to a total of 33,117 single-vehicle ran-off-roadway crashes, for an average of 3,680 crashes/year during the nine-year period under consideration. About 2.3% (772) of the 33,117 crashes resulted in one or more fatalities. About 35.7% (11,812) of the crashes involved human injuries or fatalities. The remaining 62% (20,532) of the crashes involved property damage. The 4,173 single-vehicle crashes reported in the year 1998 was the highest number of crashes in any year during the analysis period; the 2,817 crashes for 2003 was the lowest number of crashes. The 33,117 crashes were geographically located with reference to the road network using a geographic information system program.

ANALYSIS

Computation of Safety Indicators

A comparison of the number of crashes during the period before the continuous shoulder rumble strip treatment and after the continuous shoulder rumble strip treatment was a good indicator of the effectiveness of continuous shoulder rumble strips. To account for a variety of factors, which might have a bearing on the increase/reduction of the number of crashes, the analyses were carried out based on two indicators of safety for each segment:

- Crash frequency, in Crashes/Year
- Crash density, in Crashes/Mile/Year

Computation of Crash Frequency

Crash frequency was computed by dividing the total number of crashes recorded on each segment during the before or after period by their respective duration expressed in years. It was computed using the following equations:

$$Crashes/Year_{\,before} = \quad C_{ij\ before}/P_{i\ before} \qquad (1)$$

$$Crashes/Year_{\,after} = \quad C_{ij\ after}/P_{i\ after} \qquad (2)$$

where,

$C_{ij\ before}$ = Total number of crashes recorded on segment *i* in year j during the before period
$C_{ij\ after}$ = Total number of crashes recorded on segment *i* in year j during the after period
$P_{i\ before}$ = duration of the before period of segment i in years
$P_{i\ after}$ = duration of the after period of segment i in years

The total number of crashes on each facility was obtained by the simple addition of the number of crashes on each constituent segment. Since different roadway segments have different construction periods, it was implied that the before periods and after periods were also different for many of these roadway segments. In order to compute the crash rate in terms of crashes/year, a weighted average of the before and after period was computed separately, as described next:

$$\text{Average before period } (P_{before}) \quad = \quad L_i * P_{i\ before} / \quad L_i \qquad (3)$$

where,

L_i is the length of each segment
P_i is the before period of each segment

A similar computation was used to obtain the average after period (P_a). The crashes per year for each facility were computed as follows:

$$\text{Crashes/Year}_{before} \quad = \quad \Sigma_i\Sigma_j\, C_{ij\ before} / P_{before} \qquad (4)$$

$$\text{Crashes/Year}_{after} \quad = \quad \Sigma_i\Sigma_j\, C_{ij\ after} / \mathrm{P}_{after} \qquad (5)$$

Computation of Crash Density

The safety indicator in terms of crash density was computed as the crashes/year divided by the length of each segment. To compute the crash rates in terms of crashes/mile/year for each facility as a single unit, the following expression was used:

$$\text{Crashes/Mile/Year}_{before} = \Sigma_i\Sigma_j\, C_{ij\ before} /(\quad L_i * P_{i\ before}) \qquad (6)$$

where,

C_{ij} = Number of crashes of segment i in year j
L_i = Length of Segment i
$P_{i\ before}$ = “before” period of analysis of segment i

RESULTS

Crash Frequency (Crashes/Year)

The first type of analysis was performed by computing the crash rate in terms of the number of crashes that occurred per year during the before and after periods. The number of crashes on each segment was divided by the number of before or after period years to obtain the crash rate in terms of crashes/year. Once the crash rate was computed, the percent change in crash rates was computed to determine if the safety condition on the roadway had improved or deteriorated after the continuous shoulder rumble strip treatment. When the crashes/year values of each of the 306 segments were compared, it was observed that about 66% of the segments showed a decline in the number of crashes/year. These segments accounted for 81% of the total centerline miles of roadway. Likewise, 12% of the segments (about 4% of centerline miles) showed no change in crashes/year, and 23% of the segments (15% of the centerline miles) showed an increase in the number of crashes/year. The results suggested that overall the continuous shoulder rumble strip treatment was effective in reducing the number of single-vehicle ran-off-roadway crashes. The two major interstate facilities in Nevada, I-15 and I-80, recorded 23% and 36% reductions, respectively, in the number of single-vehicle ran-off-roadway crashes/year. The two major U.S. routes, US-95 and US-93, also showed reductions in the crashes/year, registering 32% and 38% reductions, respectively. Although not very high, it was observed that US-6 experienced a slight increase in the number of crashes/year, and SR-160 showed a significant increase in the crashes/year.

Overall, when the effectiveness of the continuous shoulder rumble strip treatment based on individual facilities was considered, all state routes except SR-160 showed improvement after the continuous shoulder rumble strip treatment. A summary of the crash rates before and after installation of rumble strips with their corresponding percent changes is presented in Figures 1 through 3. Figure 1 shows the results for roadways with crash frequencies ranging between 0 and 250 crashes/year, while Figure 2 provides the results for facilities whose crashes/year value varies between 0 and 60 crashes/year. Results of the analyses based on roadway class are shown in Figure 3.

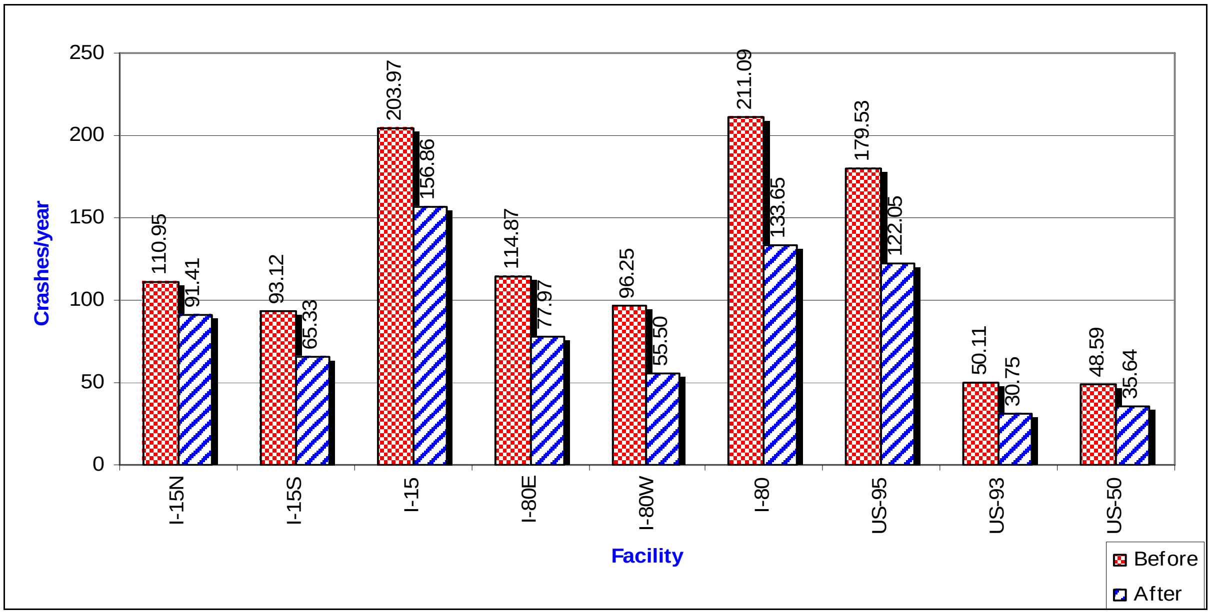

Figure 1. Single-vehicle ran-off-roadway crashes/year before and after continuous shoulder rumble strip treatment

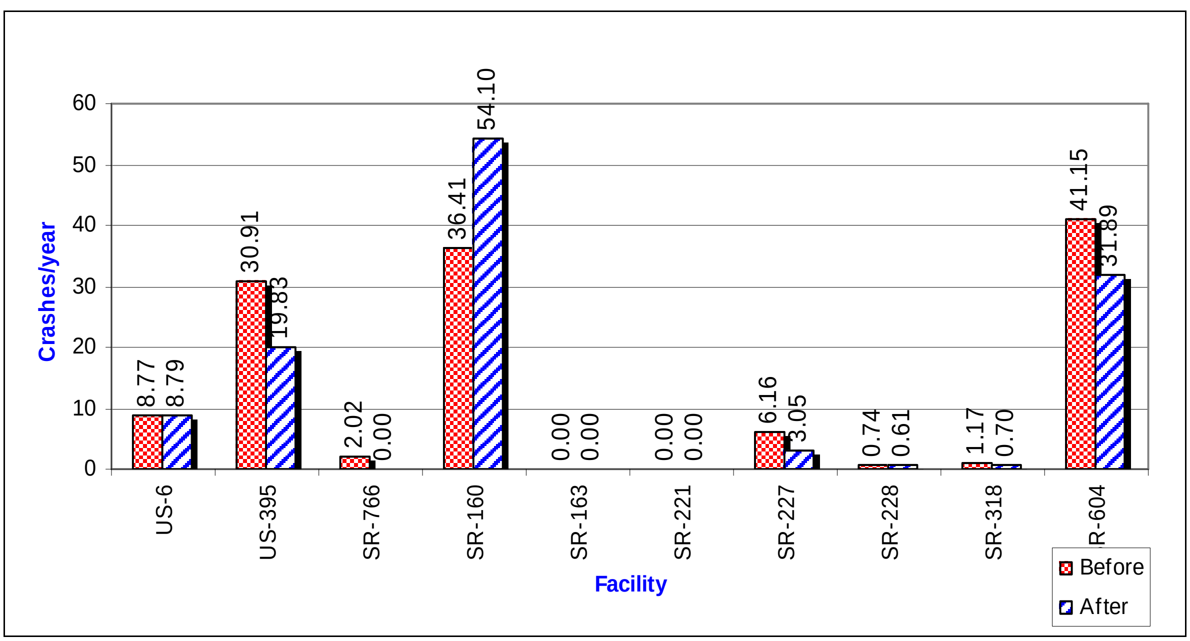

Figure 2. Single-vehicle ran-off-roadway crashes/year before and after continuous shoulder rumble strip treatment

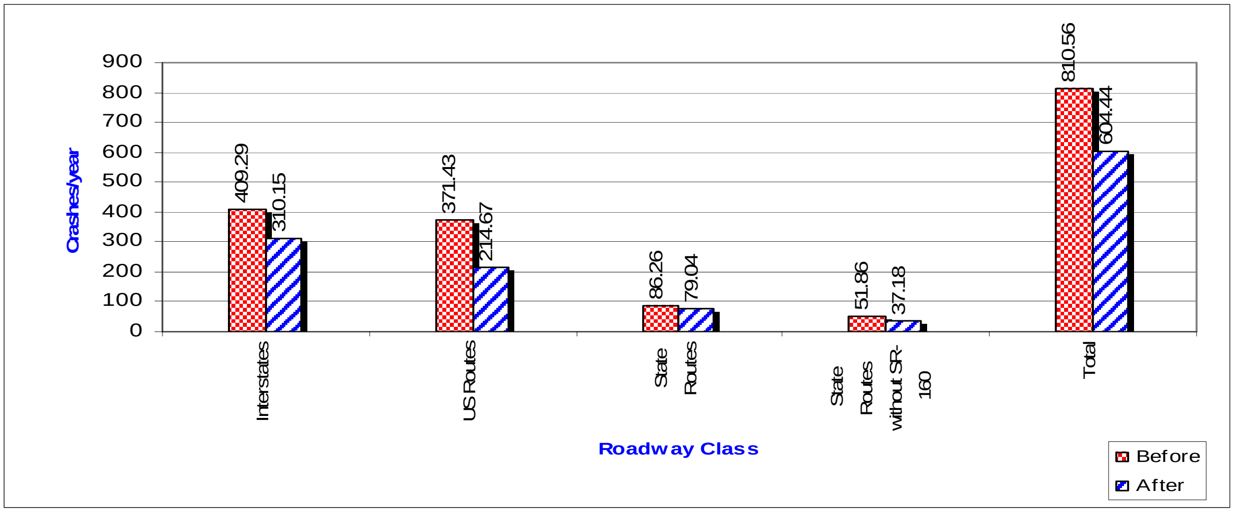

Figure 3. Single-vehicle ran-off-roadway crashes/year before and after continuous shoulder rumble strip treatment

Observations showed that over 142 of the 181 centerline miles of I-15 (78%) with continuous shoulder rumble strip treatment saw a decline in the number of single-vehicle ran-off-roadway crashes/year after the treatment. Similarly on I-80, 281 miles of the 366 centerline miles (about 77%) with continuous shoulder rumble strip treatment recorded a decline in the number of single-vehicle ran-off-roadway crashes after the continuous shoulder rumble strip treatment. On US-95, 320 miles of the 380 centerline miles evaluated (about 84%) showed improvement. Likewise, on US-93, about 91% of the 143 centerline miles studied showed a decline in the number of single-vehicle ran-off-roadway crashes/year after the installation of continuous shoulder rumble strips. Overall, of the 1,303 miles of roadways with continuous shoulder rumble strip treatment, 1,051 miles (80%) experienced a reduction in the number of single-vehicle ran-off-roadway crashes /year. This is a clear indication that, based on crash frequencies, the continuous shoulder rumble strip treatment was effective.

Crash Density (Crashes/Mile/Year)

An evaluation based solely on the number of crashes does not accurately reflect the changes in safety, since it does not address factors related to measures of exposure. If a roadway segment of short length experienced a high number of crashes, then the number of crashes/unit length of roadway would be high. On the other hand, if a longer roadway segment had the same number of crashes, then the number of crashes/unit length of roadway would be lower. Hence, to address such scenarios, crashes/mile was computed for each segment. By computing the number of single-vehicle crashes/mile, segments with relatively high crash densities or concentrations (expressed in terms of crashes/centerline mile) were identified. Once the crashes/mile was computed for the before and after periods, the rates were compared to evaluate the effectiveness of rumble strips in reducing the ran-off-roadway crashes. The results show that overall 201 of the 306 segments studied (i.e., 65.7%) showed a reduction in crashes/mile (i.e., improvements in safety). These segments accounted for 1,051 centerline miles (80.7%) of the roadways studied. Likewise, 36 segments (11.8%) showed no change in the number of crashes/miles. These segments accounted for 4.3% of the centerline miles of the roadways studied. Further, 69 segments (22.6%) experienced increased crash rates after the continuous shoulder rumble strip treatment. They constituted about 15% of the centerline miles studied. Figures 4 and 5 show results based on the number of segments for each facility. Similarly, Figures 6 and 7 show results based on centerline miles for each facility.

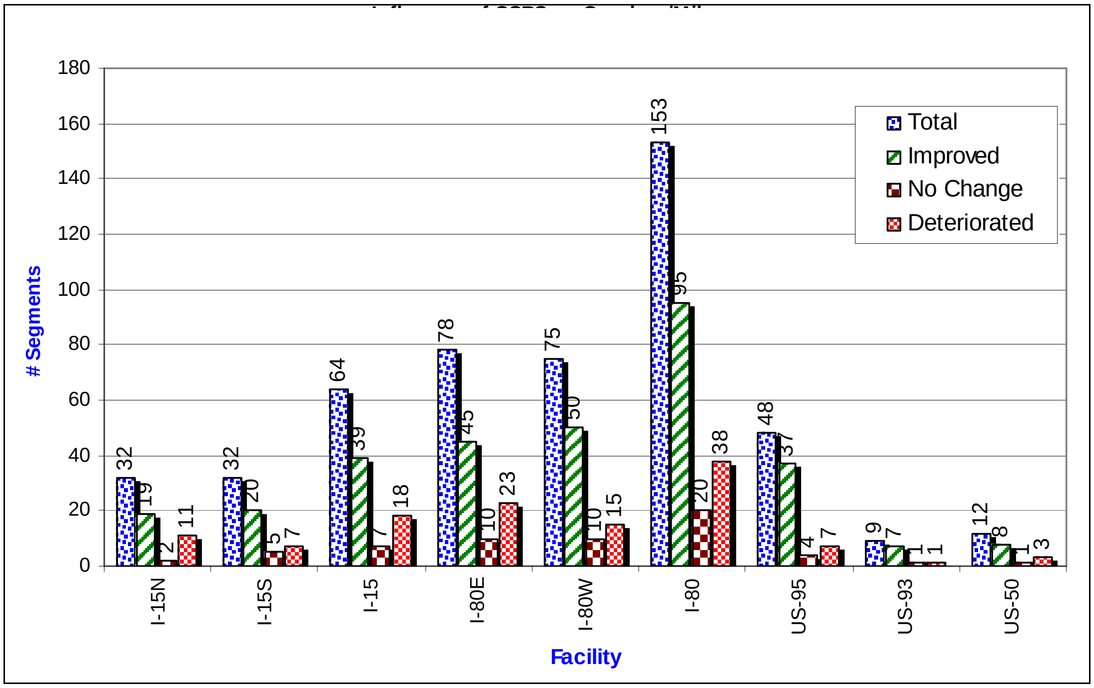

Figure 4. Influence of continuous shoulder rumble strips on crashes/mile/year (#segments)

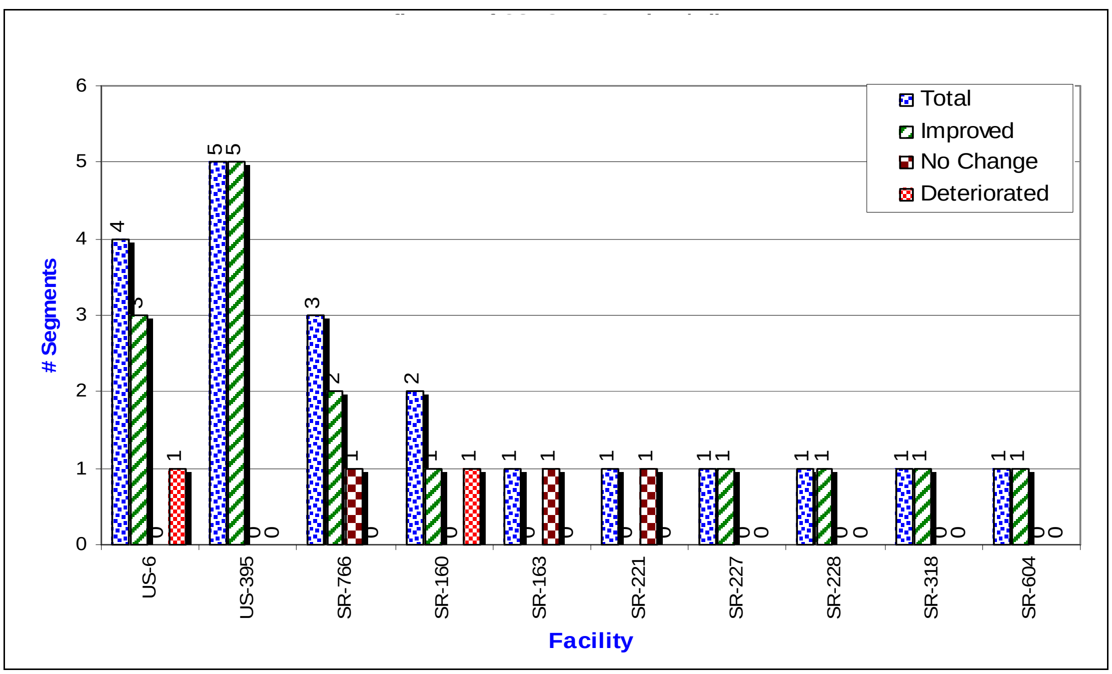

Figure 5. Influence of continuous shoulder rumble strips on crashes/mile/year (#segments)

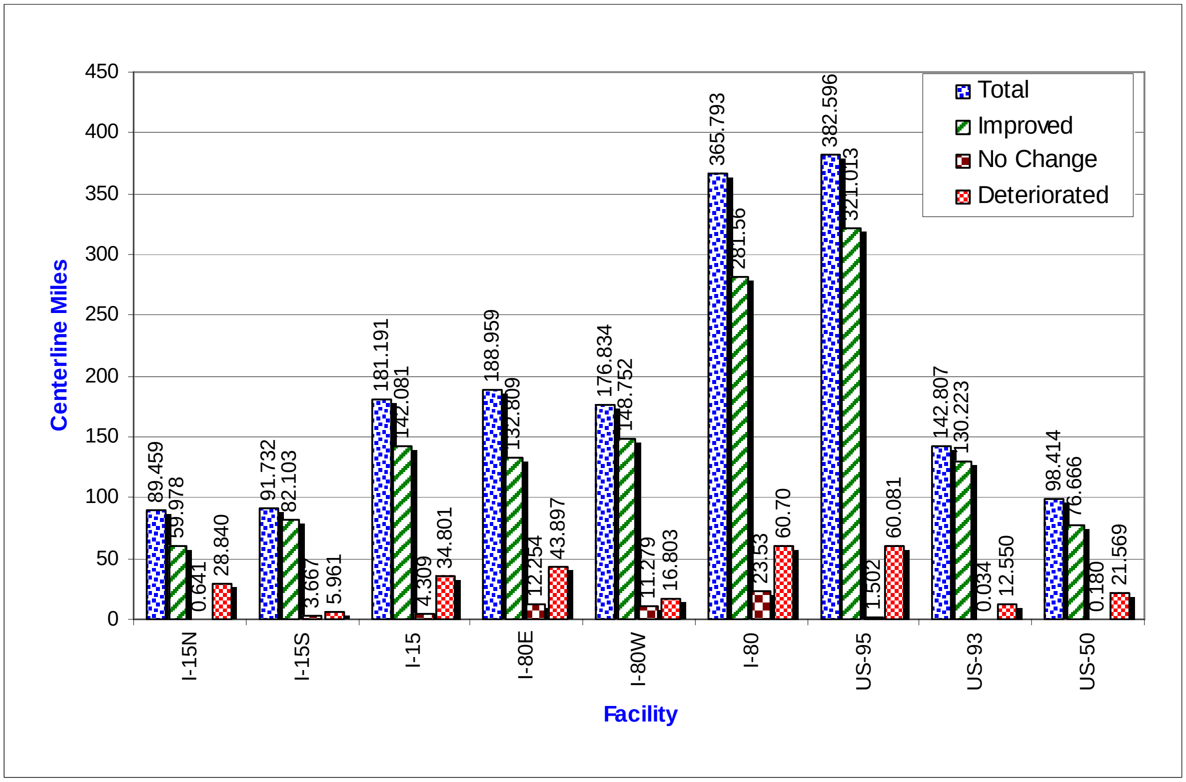

Figure 6. Influence of continuous shoulder rumble strips on crashes/mile/year (centerline miles)

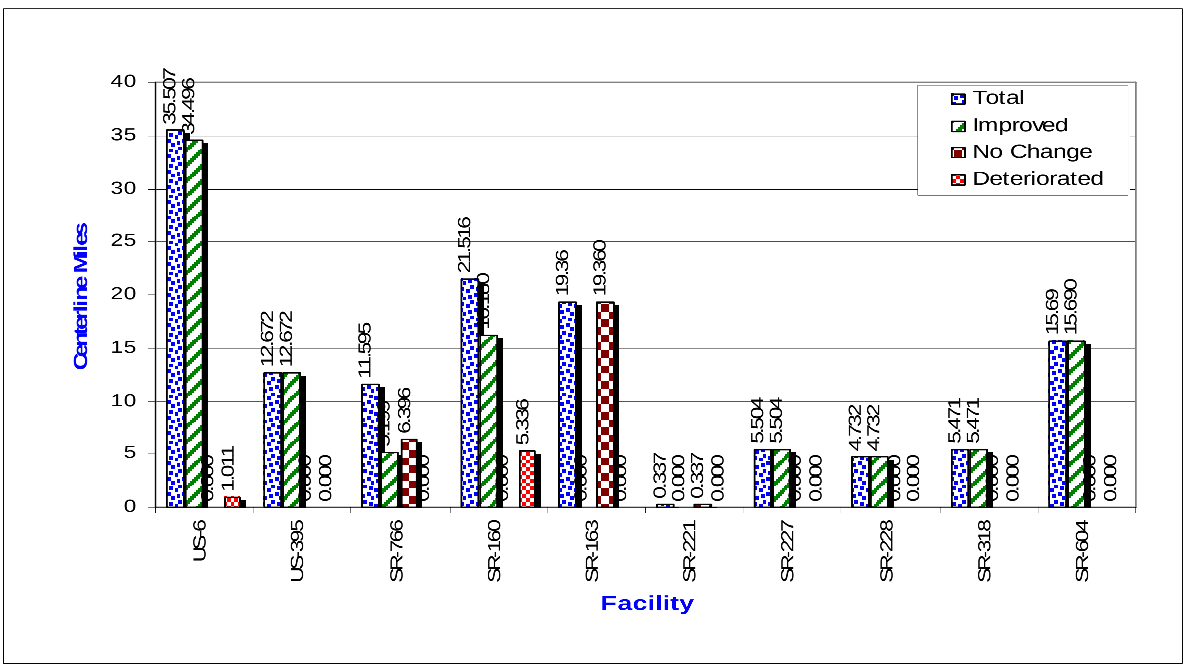

Figure 7. Influence of continuous shoulder rumble strips on crashes/mile/year (centerline miles)

SUMMARY

This paper has summarized a simple evaluation of the effectiveness of continuous shoulder rumble strips in enhancing road safety. The analyses were based on before-and-after studies comparing crash data on roadway segments and roadways before and after the installation of rumble strips. The analyses accounted for the length of individual roadway segments in determining crash rates. The results clearly show that such rumble strips on roadways in the state of Nevada have been effective in reducing the frequency of single-vehicle ran-off-roadway crashes and the corresponding crash rates. However, the analyses did not include information related to traffic volumes or vehicle miles of travel. They, too, need to be included in the analyses. Further, the changes in crash rates over time need account for normal effects, as opposed those that could be attributed to the rumble strips.

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Effect of Unmatched Longitudinal Construction Joints and Pavement Markings on Lateral Position of Vehicles

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ABSTRACT

Motorists generally follow the guidance provided by pavement markings, which are normally marked in coincidence with the longitudinal construction joints, when the joints are necessary. At some locations, however, there may be a mismatch between joints and markings, which may lead the motorists to follow joints instead of pavement markings. In the absence of detailed studies on this topic, an effort was made in this study to evaluate the effects of unmatched longitudinal construction joints and pavement markings on the lateral positioning of vehicles. Sites with such characteristics were identified, and detailed data were collected at one of the sites, using video camera technique to capture movements of vehicles for longer durations. The video tapes were later reduced to extract necessary information. Distance to the centerline of each vehicle, vehicle type, presence of vehicles in the adjacent lane, traffic volume, and vehicle movement were the main data parameters gathered while reducing the data. In addition, two surveys were also conducted to gather the opinions of practitioners.

Statistical analysis was carried out using Student's t-test to see the differences, if any. Several comparisons were made for various types of vehicles traveling under different weather conditions and vehicles going straight and turning right immediately after passing the location. The analysis results indicated that the distance to the center line of vehicles traveling in the target lane was statistically different from the expected lateral positioning of the vehicles if they were not affected by the joints.

Based on the survey and analysis of field data, drivers' lateral position seems to be affected by unmatched construction joints and pavement markings. It might be advisable to make efforts to avoid such occurrences.

Key words: driver confusion—lateral position of vehicles—longitudinal joints—pavement markings

INTRODUCTION

Drivers rely on a complex series of visual cues to safely navigate the roads. Longitudinal lines, transverse lines, arrows, words, symbol markings, and special markings constitute different types of pavement markings that guide the motorists in positioning the vehicles on the roads. Longitudinal lines, such as center lines, lane lines, and edge lines, delineate vehicular paths of travel along the roadway by marking the center of road, lanes of travel, and edges of the pavement, respectively. Pavement edge lines provide a visual guide in confining vehicles to a travel lane. Several factors, such as speed, traffic composition, weather conditions, roadway geometric design features, drivers' physical condition, and personal attributes, may also have an influence on the lateral position of vehicles.

In situations where the pavement is wider than the paving machine, longitudinal construction joints occur. These joints are normally expected to coincide with pavement markings. Sometimes, the joints are induced by sawing to prevent random cracking. In some circumstances, however, pavement markings do not exactly match the construction joints. Under these conditions, motorists may face difficulty in choosing between pavement markings and construction joints as guiding marks for their movement.

The study described in this paper has primarily been conducted to evaluate the effects of unmatched pavement markings and longitudinal construction joints on lateral position of vehicles. The research team undertook the activity of an extensive field study to locate the sites having such characteristics. Detailed field data were collected, and analysis was carried out based on data collected at one site that met the requirements. A questionnaire was also sent to transportation professionals and engineers of the state of Kansas and across the country to solicit their opinions on the unmatched joints and pavement markings. Through the evaluation of effects of mismatched joints and pavement markings on the lateral position of vehicles, this study is expected to provide transportation agencies with guidelines on the placement of longitudinal joints on the pavement.

LITERATURE REVIEW

Even though not much information is currently available on the research carried out on unmatched joints and pavement markings, some studies have been conducted to evaluate the effects of pavement edge lines on lateral position of vehicles. One such investigation was performed by the Missouri State Highway Department in 1969 to study the effect of pavement edge lines on the lateral position of vehicles on rural two-lane highways with widths between 20 ft. and 24 ft (Missouri State Highway Commission 1969). Vehicle placement was measured using an electronic placement tape with a 20-pen graphic recorder. The main finding was that vehicles generally tended to move closer to the centerline under free-flow conditions after applying the edge lines. In 1971, Hassan confirmed the results of the previous study by utilizing a mechanical traffic counter to measure the vehicles' lateral placement on two one-mile sections that were 18 ft. and 24 ft. wide in Maryland (Hassan 1971). The analysis found that, with edge lines, vehicle position was closer to the centerline of the roadway on both sections.

A research study was conducted by Steyvers and De Waard (2000) in the Netherlands using video recording equipment to observe vehicles' position changes before and after edge line markings on four narrow rural roadways with pavement widths between 13.5 ft. and 14.8 ft. (Steyvers and De Waard 2000). It was observed that drivers took a more central position after the edge line markings were incorporated on the road.

Sun et al. (2006) conducted a study in Louisiana to evaluate the effects of pavement edge lines on lateral position of vehicles. (Sun et al. 2006). After thoroughly experimenting with and evaluating several data collection methods, Sun et al.'s research team used air switch devices (also known as road tubes) for collecting large number of samples, as this method was found to be more reliable, less intrusive, and easier to setup in the field. Three traffic counters were used, each connected with at least two tubes for collecting the data. The tubes were fixed in such a manner that the data for vehicles with their right tires touching the one ft. section of roadway next to the pavement edge, vehicles with their right tires touching the roadway section between one and two ft. from the roadway section, number of vehicles crossing the center line, hourly volume, and operating speed of vehicles were obtained. The data were collected at a total of ten sites on Louisiana rural two-lane highways for at least 24 hours before and after implementation of edge lines. It has been found that with the implementation of edge lines the vehicles followed a more centralized path, which indicates that there is an effect of pavement edge lines on the lateral position of vehicles.

Another study has been conducted in Tyler, Texas, to compare the edgeline effects on speed, lateral position, and human perception (Tsyganov et al. 2006). Three two-lane roads with lane widths of 9, 10, and 11 ft. were selected for collecting the data and carrying out the analysis. The lateral position of vehicles before and after the edge line treatments were observed under both the categories of stationary observation design and test driving design. On applying the edge line, drivers traveling on the 9 ft. lane width highways moved their vehicles closer to the roadway edge, with greatest movement on curved sections. While driving on 10 ft. lane width highways, the drivers tended to move slightly towards the center of the road. While traveling on the 11 ft. wide lanes, the drivers moved slightly closer to the centerline under all lighting conditions. These results indicate that as the lane width increases the drivers tend to be closer to the centerline under all lighting conditions upon the implementation of edge lines.

In summary, the majority of past studies confirm that there is a significant impact of the pavement edge lines on the lateral position of vehicles.

METHODOLOGY

Surveys

Transportation professionals and engineers from various agencies across the country and in the state of Kansas participated in two web-based surveys conducted through the Kansas Department of Transportation (KDOT). Participants expressed their views on the operational and safety problems that arise in sites having unmatched joints and pavement markings. The general policies of the corresponding agencies were also obtained from the survey. Thirty American Association of State Highway and Transportation Officials (AASHTO) members responded to the first survey. Transportation officials and engineers from different counties in the state of Kansas took part in the second survey. Mixed responses were received on the unmatched joints and pavement markings. Some departments of transportation (DOTs) preferred to match the joints and markings, whereas a few of them were concerned about the maintenance of the pavement and hence were willing to offset the joints from the pavement edge lines. A summary of the survey results from the AASHTO responses are reported in Table 1.

Table 1. Summary of survey results from the AASHTO responses

State	Operational/Safety Problems due to unmatched joints and pavement markings	General policy of the transportation agency on unmatched joints and pavement markings
Alabama	Operational problems arise if the joints are in the lane as the longitudinal joint (LJ) is typically the weakest area of the lane. They expressed that if wheel path runs along that LJ, failure is most likely to occur. They stress this point in Hot Mix Asphalt (HMA) Lay down training.	It is not a good practice. Their specifications state that 'LJ's in the wearing layer shall confirm with the edges of the proposed traffic lane, in so far as practical.'
Arizona	One of the four regional traffic engineers and one of the maintenance engineers expressed safety concern if the joint falls in the wheel path.	Differing opinion based on pavement types: if asphalt there may be no safety or operational issues; if undoweled concrete there is the issue of vertical misalignment; with doweled concrete there may not be safety or operational issues.
Arkansas	Neither operational nor safety problems.	They try to avoid unmatched joints and pavement markings. However, they have not found that unmatched joints and pavement markings are a problem if the pavement is built correctly.
Colorado	Neither operational nor safety problems.	They have not had a problem as long as the longitudinal joints are within a foot of the pavement marking for HMA. For PCCP, they have placed 14 foot driving lanes with the pavement marking placed at 12 feet. These widened lanes have been observed to perform well with no adverse traffic problems.
Connecticut	Operational and safety problems arise from those situations.	They allow an offset of 6 inches to the joint from the marking in order to avoid the failure of either of them due to the failure of the other.
Delaware	Operational problems arise when the longitudinal joint is in the wheel path.	They try and match the joints and markings so they do not have wheel paths at longitudinal joints. They observed that the wheel path at longitudinal joint causes premature failure and raveling of joints.
Florida	Neither operational nor safety problems.	They felt that the joints in the wheel path create roughness and pavement performance issues.
Georgia	Operational and safety problems arise from such a situation as the drivers perceive the joints as lane markings after the markings are worn.	They felt that unmatched joints are acceptable on ramps and intersections. In the past their agency used unmatched joints and markings in a couple of areas and the results were not satisfactory. They do not prefer to use the unmatched joints within interstate travel lanes or within lane shifts.
Iowa	Operational problems arise from such a situation.	They felt that was not a good practice.
Kentucky	No comments.	No comments.
Louisiana	Operational problems arise from such a situation. They have observed lateral movement of vehicles when markings have substantially faded.	The operational effects of not aligning joints and pavement markings were found to be minimized by reconstructing the portions of the road so as to break up the continuous joint.
Maine	Operational and safety problems arise from such a situation.	As per their specifications, they do not accept unmatched joints and pavement markings.
Maryland	Safety problems might arise from such a situation.	He didn't know of a significant problem.

Table 1. Continued

State	Operational/Safety Problems due to unmatched joints and pavement markings	General policy of the transportation agency on unmatched joints and pavement markings
Michigan	Operational and safety problems arise from such a situation. It is felt that the situation causes confusion to the driver.	Their standards require joint lines and pavement marking to match with rare exceptions.
Mississippi	Neither operational nor safety problems.	Markings were much more visible than the joints and have never caused problems. They offset the joints by one inch from the markings for a better appearance.
Nebraska	Neither operational nor safety problems.	Mismatched joints and markings are less than desirable.
Nevada	Operational and safety problems arise from such a situation. While raining or when the pavement markings are almost worn out, drivers tend to follow joint line thinking it as an edge line.	The agency does not allow unmatched joints and pavement markings.
New Hampshire	Operational problems which lead to a failed pavements.	Not a good idea.
New Jersey	Neither operational nor safety problems.	The agency prefers to offset pavement joints from traffic stripes by approximately 6 inches
New York	Operational and safety problems.	They prefer to keep them together wherever possible. Else, they wish to position them in the middle.
North Dakota	Operational and safety problems. Crash analysis indicates side-swipe crashes in these areas.	They believe that there is a safety problem due to the mismatch.
Pennsylvania	Operational and safety problems. Some accidents have been reported in these locations which have been attributed to confusion in lane assignments.	Avoid unmatched joints and markings at all costs.
South Carolina	Neither operational nor safety problems.	They prefer to match joints and markings.
Tennessee	Neither operational nor safety problems.	No exact comment on the mismatch.
Texas	Neither operational nor safety problems.	Placing pavement markings over the joints is detrimental to the durability of the markings. They were not aware of the safety issues at locations where they had a mismatch.
Virginia	Safety problems arise. Virginia DOT has safety concerns regarding motorcycles traversing joints. They require signs noting wide joints to alert motorists of a possible safety hazard.	For concrete pavements, longitudinal joints need to be located at the edge of the travel lane. For asphalt pavements, they need to be located at the center of the travel lane.
West Virginia	Operational and safety issues arise due to the mismatched joints and markings, especially under wet weather conditions.	Do not allow this to happen if at all possible.
Wisconsin	Operational problems arise under wet weather conditions and also during nights.	From maintenance point of view, it is advisable to offset joints from markings at least by 3 to 4 inches.
Wyoming	Operational and safety problems arise in areas when the pavement markings fade off.	They prefer to match joints and markings wherever possible. They feel that they can mitigate the problem to a certain extent by keeping the joint lines fresh.

The second survey, involving transportation engineers in Kansas, was mainly targeted to gather local opinions and identify suitable sites. After careful evaluation of available information, one site was selected, as it was suitable for detailed data collection (Figure 1).

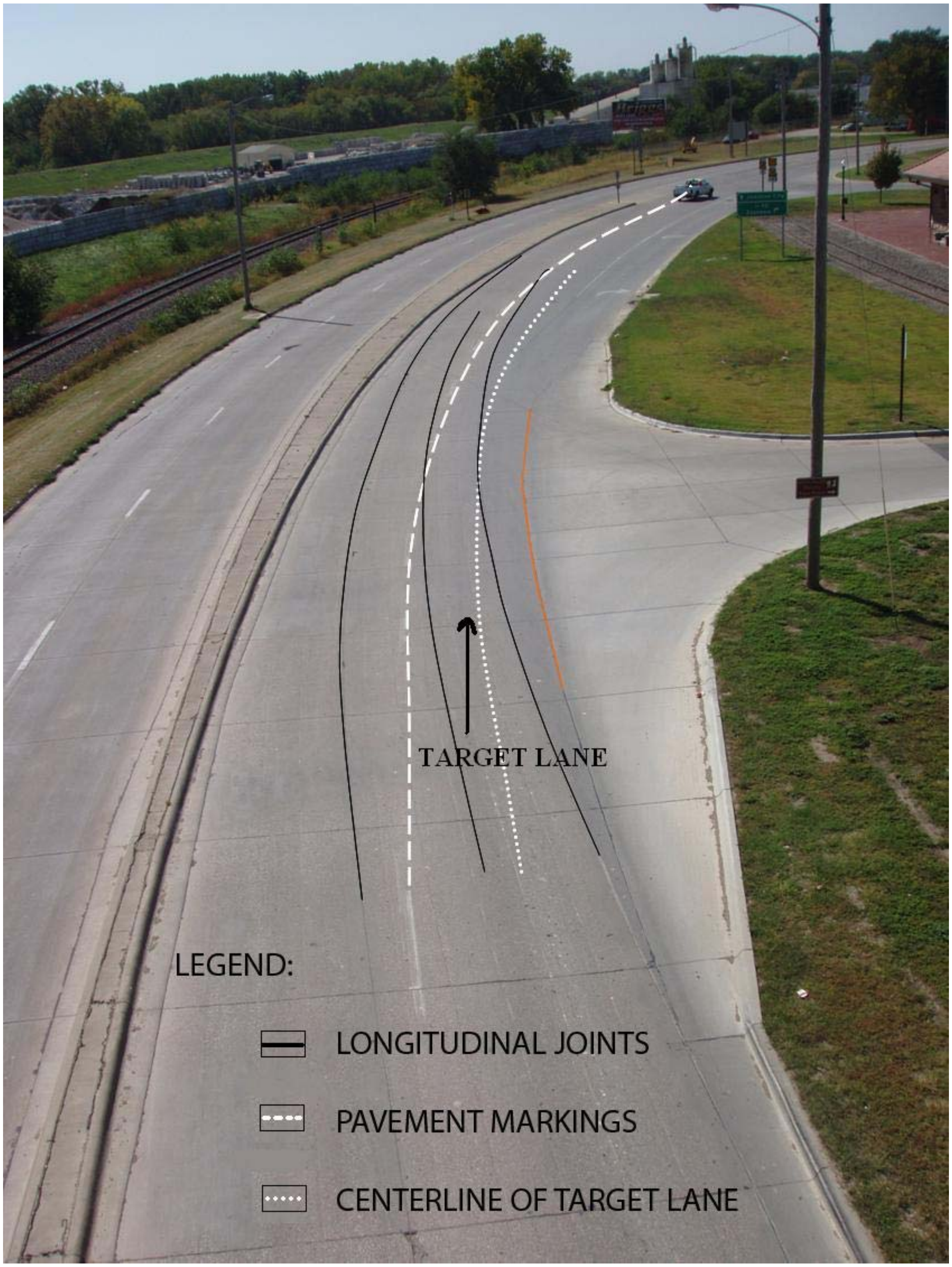

Figure 1. Photograph of the data collection site

Field Study

The site selected for field data collection is a four-lane road having two lanes in each direction. The widths of the two lanes of the road were measured as 12.40 ft. and 11.85 ft., respectively. Longitudinal joints were located at 5 ft. away from edge of the road, as shown in the Figure 1. A video camera (fisheye camera) was set up on a utility pole with a recorder at its bottom in order to capture the movement of vehicles for an extended period of time. Data were collected for longer durations at the site, which had the characteristics of unmatched longitudinal joints and pavement markings. The data recorded were then extracted in the laboratory from the video tapes.

The width of two lanes of the road was measured as 24.25 ft. A scale was marked in divisions of inches that had one inch divided into sixteen divisions. A straight portion of the section of road was chosen and a scale was then set up (fixed) on the television screen, such that its 0 value started at the right curb of the road. The distance to the front right (passenger side) of the vehicle traveling in the target lane from the right curb was measured using this scale. For all vehicles, a similar method was adopted. The widths of the vehicles were also calculated. Thereby, the distance to the centerline of the vehicles from the right curb was computed. These distances were then converted to the real-world dimensions with the help of Excel spread sheets by applying the corresponding scale factor. In addition to the distance to the right passenger side of the vehicle in the target lane, other details, such as the type of vehicle, whether there was a vehicle traveling in the adjacent lane, and the movement of the vehicle right after passing the location (i.e., right turn on to the ramp vs. straight), were extracted from the video tapes. Also recorded was the weather condition at the time of the data collection to see whether it affects driver performance.

Data related to a total of 14,050 vehicles was extracted from the video tapes. Vehicles were classified on the basis of weather conditions and movement. Vehicles traveling under good and bad weather, i.e., rainy, snowy, and wet pavement conditions were observed. From the total vehicles extracted from the video tapes, 8,518 and 5,532 vehicles were observed to be traveling under good and bad weather conditions, respectively. Vehicles that were going straight and those making right turns immediately after the portion of the road that had unmatched joints and pavement markings have been observed as 8,714 and 5,336 vehicles, respectively.

DATA ANALYSIS

The mean distance to the centerline of the vehicles from the right curb of the road was used as the variable for analyzing the data via Student’s t-test using Statistical Analysis Software (SAS). The command “PROC TTEST” computes the t-statistic by using the following formula:

$$t = \frac{X - \mu}{\left(\dfrac{s}{\sqrt{n}}\right)} \tag{1}$$

Where,

t =t-value
X =The mean distance to centerline of vehicles from the right curb of road
μ = Expected position of centerline of vehicles, which is the centerline of the target lane
s =Standard deviation of distance to the centerline of vehicles
n =Number of observations

The null hypothesis is the mean distance to the centerline of the vehicles from the right curb of the road and is the same as the expected position of the centerline of the vehicles in the target lane. The a value has been assumed as 0.05. The SAS software directly gives the probability value, i.e., *p*-value. If the *p*-value associated with the t-test is small ($p<0.05$), there is evidence that the mean is different from the hypothesized value. If $p>0.05$, then the null hypothesis is not rejected and it can be concluded that the mean is not different from the hypothesized value.

The independent group t-test is used for comparing the means of two groups of data. The command "PROC TTEST COCHRAN" was used for analyzing the data by independent group t-test. It reports two t-statistics: one under equal variance assumption and the other under unequal variance. It also reports an *F*-value, which helps identify the type of *t*-test used in analyzing the data. The *F*-statistic is computed to check the equality of variance, which uses the following formula:

$$F^{'} = \frac{\left(larger\ of\ \ s_1^2, s_2^2\right)}{\left(smaller\ of\ \ s_1^2, s_2^2\right)} \tag{2}$$

The SAS program displays the *F*-statistic along with a *p*-value. If the *p*-value associated with the *F*-test is greater than 0.05, the null hypothesis that the variances of two samples are equal can be accepted, and *t*-statistic is computed by pooled method of equal variance by using the following formula:

$$t = \frac{(x_1 - x_2)}{\sqrt{s^2\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}} \tag{3}$$

Where,

t =t-value
x_1 =Mean of the first group
x_2 =Mean of the second group
s^2 =Pooled variance
n_1 =Sample size of the first group
n_2 = Sample size of the second group

The value of pooled variance, s^2, is calculated under the assumption that the population variances for two samples are equal using the following formula:

$$s^2 = \frac{(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2}{(n_1 + n_2 - 2)} \tag{4}$$

where

s_1^2 =Sample variance of the first group
s_2^2 =Sample variance of the second group

If the p-value associated with the F-test is less than 0.05, we reject the null hypothesis and the t-statistic is computed under the assumption of unequal variance by using the following formula:

$$t = \frac{(x_1 - x_2)}{\sqrt{w_1 + w_2}} \tag{5}$$

Where,

x_1 =Mean of the first group
x_2 =Mean of the second group

and where w_1 and w_2 are computed using

$$w_1 = \frac{s_1^2}{n_1} \tag{6}$$

$$w_2 = \frac{s_2^2}{n_2} \tag{7}$$

The Cochran and Cox Approximation or Satterthwaite's Approximation is used for computing the t-statistic under the assumption of unequal variance, in which case SAS output reports both values.

The degrees of freedom for Satterthwaite's Approximation is computed as follows:

$$df = \left(\frac{(w_1 + w_2)^2}{\frac{(w_1^2)}{(n_1 - 1)} + \frac{(w_2^2)}{(n_2 - 1)}} \right) \tag{8}$$

In summary, if the probability value (p-value) for the computed F statistic is greater than 0.05, the method of equal variance is accepted. Otherwise, the t-statistic corresponding to Satterthwaite's or the Cochran and Cox Approximation is used for analyzing the data (Steel and Torrie 1960).

DATA ANALYSIS

A histogram was plotted, with the cumulative percentage of vehicles on the y-axis and distance to the centerline of the vehicles from the right curb of the road on the x-axis. The best fitted curve represented a bell shape, similar to that of normal distribution. As the data came from normal distribution, a t-test was applied for analyzing the data.

Initially, the entire dataset of 14,050 vehicles extracted from the video tapes was analyzed by Student's t-test using SAS software. If the vehicles are assumed to be guided by the pavement markings alone, the expected position of the centerline of the vehicles from the right curb of the road should have been located at 6.2 ft., which is half the width of the target lane, and which is the distance to the centerline of the target lane. The calculated mean and standard deviation of distance to the centerline of the vehicles from the right curb of the road were 7.06 ft. and 1.61 ft., respectively. The null hypothesis has been assumed as the calculated mean distance to the centerline of the vehicles from the right curb of the road is same as half the width of the target lane (6.2 ft.). The *t*-value was calculated using the formula from equation (1), by substituting the values as follows:

X = 7.06 ft.
μ = 6.20 ft.
s = 1.61 ft.
n = 14,050

The value of the *t*-statistic was obtained as 63.72. The probability value has been reported in the SAS output as p<0.0001. As the *p*-value reported in the output is less than 0.05, with 95% confidence, it can be said that the mean distance to the centerline of the vehicles from the right curb of the road is significantly different from 6.20 ft. It implies that the null hypothesis can be rejected. The *t*-values and *p*-values corresponding to the *t*-tests are reported in Table 2.

Table 2. Summary statistics based on type of vehicle

Description	Sample Size	Mean (ft.)	Std. Dev. (ft.)	*t*-value	*p*-value
All vehicles	14,050	7.06	1.61	63.72	<0.0001
Passenger cars	5,878	6.36	1.44	8.36	<0.0001
Vans	4,055	7.42	1.49	51.85	<0.0001
Pick-ups	3,352	7.55	1.48	52.92	<0.0001
Heavy vehicles	765	8.50	1.61	39.55	<0.0001

An independent group *t*-test was carried out to analyze the vehicles traveling under good and bad weather conditions. This test was also applied for vehicles going straight and those taking a right turn. The procedure used for performing the analysis can be explained by considering vehicles traveling under good and bad weather conditions. It has been observed that 8,519 vehicles (n_1) were traveling under good weather conditions, with a mean (x_1) and standard deviation (s_1) of 7.23 ft. and 1.71 ft., respectively. It was also observed that 5,531 vehicles (n_2) were traveling under bad weather conditions, with a mean (x_2) of 6.72 ft. and a standard deviation (s_2) of 1.37 ft. The "PROC TTEST COCHRAN" command computes the folded-form *F*-statistic to check the equality of variances using Equation (2). The *F*-value and corresponding *p*-value were displayed in the SAS output as 1.57 and p<0.0001, respectively. It implies that the method of unequal variance is used for computing the *t*-statistic.

The *t*-statistic was computed under the assumption of unequal variance by Equation (5), taking the values of x_1 and x_2 to be 7.23 ft. and 6.72 ft., respectively. w_1 and w_2 were computed using Equations (6) and (7) as 0.000347 and 0.000339. The *t*-statistic was calculated as 19.62. In addition to this, the "PROC TTEST COCHRAN" also displayed the *t*-value calculated under the assumption of equal variance by substituting the corresponding values in Equations (3) and (4) respectively. However, the *t*-statistic computed under the assumption of unequal variances has been reported as the test value, as the method of equal variance had been rejected by the *F*-test.

As the *p*-value corresponding to the *t*-test is less than 0.05, the null hypothesis, that the mean distance to the centerline of the vehicles under good weather is the same as that under bad weather can be rejected.

Hence, with 95% confidence, it can be said that the mean distance to the centerline of vehicles under good weather conditions is different than that of vehicles under bad weather conditions.

The vehicles were classified into two different categories, vehicles classified on the basis of movement and vehicles traveling under different weather conditions. The summary statistics, i.e., the mean and standard deviation of vehicles traveling under different weather conditions, is reported in Table 3. In addition to these, the *p*-values corresponding to the independent *t*-tests, along with the *F* - statistic and *t* - statistic values, are also reported. The *t*-test has also been applied to the vehicles classified on the basis of movement, and its details are reported in Table 4. Since the *p*-value corresponding to the *F*- statistic for different vehicles under good and bad weather conditions has been found to be less than 0.0001, the method of unequal variance was used for analyzing the data, and hence the *t*-value calculated using this method is reported as the test value. In terms of the *t*-test carried out upon classifying the vehicles on the basis of movement, since the *p*-value associated with the *F*-statistic is greater than 0.05, the method of equal variance was used for analyzing the data. Hence, the *t*-value corresponding to that method is reported as the test value.

All the results were found to be statistically significant, except for the heavy vehicles tested with respect to movement. The computed *p*-value, corresponding to the *t*-test, was found to be greater than 0.05, which implies that the result is not statistically significant.

Table 3. Summary statistics of vehicles under good and bad weather conditions

Description	Weather Condition	Sample Size	Mean (ft.)	Std. Dev. (ft.)	*F*-test		*t*-test	
					***F* value**	**Pr.>*F***	***t*-value**	**Pr.>*t***
All vehicles	Bad Good	5,532 8,518	6.76 7.27	1.37 1.72	1.57	<0.0001	19.62	<0.0001
Passenger cars	Bad Good	2,290 3,588	6.10 6.52	1.23 1.53	1.55	<0.0001	11.70	<0.0001
Vans	Bad Good	1,773 2,282	7.10 7.66	1.26 1.61	1.62	<0.0001	12.47	<0.0001
Pick-ups	Bad Good	1,210 2,142	7.21 7.75	1.26 1.62	1.69	<0.0001	11.09	<0.0001
Heavy vehicles	Bad Good	258 507	8.13 8.79	1.27 1.71	1.82	<0.0001	5.47	<0.0001

Table 4. Summary statistics of vehicles going straight and making a right turn

Description	Movement	Sample Size	Mean (ft.)	Std. Dev. (ft.)	*F*-test		*t*-test	
					***F* value**	**Pr.>*F***	***t*-value**	**Pr.>*t***
All vehicles	Right Straight	5,336 8,714	6.90 7.16	1.61 1.60	1.02	0.34	9.40	<0.0001
Passenger cars	Right Straight	2,221 3,657	6.14 6.49	1.44 1.42	1.02	0.52	9.10	<0.0001
Vans	Right Straight	1,503 2,552	7.16 7.47	1.45 1.51	1.07	0.12	6.07	<0.0001
Pick-ups	Right Straight	1,295 2,057	7.42 7.64	1.46 1.49	1.04	0.46	4.21	<0.0001
Heavy vehicles	Right Straight	317 448	8.58 8.44	1.48 1.69	1.30	0.01	1.19	0.2455

CONCLUSIONS

Based on the survey and analysis of field data, the lateral position of vehicles seems to be affected by unmatched pavement markings and longitudinal construction joints. All the results were found to be statistically significant, except those for the heavy vehicles tested on the basis of movement.

If the position of vehicles is assumed to be guided by pavement markings alone, the mean distance of vehicles would have been 6.2 ft. Since the longitudinal joint was located 5 ft. away from the right curb of the pavement, the drivers would have followed the joints instead of the markings. The mean distance of travel was observed as 7.06 ft., which could be due to the drivers' confusion resulting from the mismatch between longitudinal joints and pavement markings. It should be noted that the detailed data collection was limited to just one site because of the difficulty in identifying more sites with similar characteristics. This research needs to be expanded by identifying and collecting more sites with similar characteristics to make the findings more reliable.

The standard specifications of 35 states have provisions concerning unmatched joints and pavement markings. Some states do not have any information regarding the positioning of longitudinal construction joints with respect to pavement markings. It would be better if the standard specifications of all the states had provisions pertaining to the mismatch.

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Effect of Concrete Pavement Surface Texture on Traffic Safety: Longitudinal or Transverse Tines?

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ABSTRACT

Longitudinally tined (LT) concrete pavement surfaces were shown to produce less noise than transversely tined (TT) ones, making them more desirable where traffic noise may be an issue, if the two textures are shown to have equivalent safety performance.

The present effort focused on rural freeway crashes between 1991 and 1998. This guaranteed the presence of high speeds and absence of various factors unrelated to pavement texture that affect safety, such as traffic signals, cross-street traffic, etc.

The Wisconsin Department of Transportation uses TT portland cement concrete (PCC) surfaces. California provided the only substantial database for LT PCC. Climatic differences between Wisconsin and California were addressed by using hourly precipitation and travel data to calculate wet pavement crash rates for each of the two states.

No statistically significant difference in the risk for wet pavement accidents vs. dry pavement accidents was identified between the two pavement textures (2.25 times higher risk on wet pavements for TT vs. 2.39 times for LT pavements). The database comprised 72.6 hundred million vehicle miles of travel (HMVM) in Wisconsin and 510 HMVM in California.

Key words: concrete pavement—crash rate—pavement texture—traffic safety—wet pavement

PROBLEM STATEMENT

Longitudinally tined (LT) concrete pavement surfaces were shown to produce less traffic-induced noise than transversely tined (TT) ones, making them more desirable where traffic noise may be an issue. However, before LT surfaces can be recommended for statewide application, they should be shown to have equivalent or superior safety performance to their TT counterparts.

A Technical Working Group (TWG) representing state highway agencies, industry, academia, and the Federal Highway Administration (FHWA) that convened between 1993 and 1996 stated that the purpose of surface texture is to reduce the number and severity of wet weather accidents. The TWG recommended that pavement surface texture safety should be evaluated based on a consecutive three- to five-year period crash analysis.

The present effort was based on eight consecutive years of crash information (1991 to 1998) that was merged with vehicular travel and hourly precipitation information, in order to produce wet pavement crash rates for each of the two pavement surface textures. Wet pavement safety concerns are most prevalent where high operating speeds prevail. This led to a focus on rural freeways that typically operate under uncongested conditions and high operating speeds. The freeway environment guaranteed the absence of various factors unrelated to pavement texture that affect safety, such as traffic signals, cross-street traffic, curbside parking, unusual geometry, etc.

A major challenge for this Wisconsin Department of Transportation-originated study was that the only state that applied LT portland cement concrete (PCC) surface specifications statewide during the past three decades was California. The choice of rural freeways for the analysis provided a background of similar geometry; mountainous-terrain California freeways were excluded from consideration given the flat/rolling Wisconsin terrain. However, climatic differences between Wisconsin and California had to be addressed for a persuasive safety comparison.

RESEARCH OBJECTIVES

The present effort is a comparison of TT Wisconsin PCC pavements with LT PCC pavements. The focus of this comparison is differences in wet pavement crash risk on high-speed facilities. Motivation is provided by findings of lower highway noise levels generated by LT surfaces vis-à-vis concerns for the safety performance of these pavements when compared to the widely used TT surfaces. Departments of transportation would use a quieter pavement surface texture, especially in urban areas, if it is shown not to be detrimental to safety; inferior safety performance will immediately disqualify a surface texture from further consideration.

RESEARCH METHODOLOGY

If safety differences exist between LT and TT pavement surfaces, these differences are expected to be the greatest under wet conditions and especially where high operating speeds prevail. Rural freeways were chosen as the ideal facilities for the desired comparison for a number of reasons:

1. They are typically not congested, thus free-flow speeds are likely to prevail.
2. No intersections are present. Intersections introduce a large number of variables affecting safety performance (number of approach lanes, lane designation, traffic control parameters, cross-street volumes, etc.).
3. There is no friction with on-street parking, pedestrians, and bicyclists.

4. Good quality crash data and other highway information is available.
5. High geometric design standards eliminate to a large extent the influence of sharp horizontal and vertical curves on crashes.
6. Uniform geometric design standards eliminate the influence of differences in state-specific geometric design practices.
7. A large number of crashes are typically available for analysis.

Extensive data were available in California for LT pavement surfaces. Wisconsin provided information on TT pavement surfaces.

The following safety performance measures of effectiveness (MOE) were calculated for each year for each of the two states (definitions and interpretations of these MOE are presented in the following section):

- Crash rate
- Wet-to-dry ratio
- Liquid precipitation safety ratio (LSR)

MOE Definitions and Interpretations

This section presents the meaning and interpretation of statistics used in this paper. Multiple interpretations of the fundamental LSR statistics are provided for the benefit of the interested reader.

Crash Rates

Crash rates were calculated as total crashes per one hundred million vehicle miles of travel (HMVMT = 100 MVMT) and rounded to integer values. A higher crash rate indicates that a higher number of crashes occurred per vehicle-mile of travel and is an indication of poorer safety performance.

$$Crash\ Rate = \frac{Total\ crashes}{100\ MVMT} \tag{1}$$

Wet-to-Dry Ratio

The wet-to-dry ratio (wet-to-dry crashes) is the number of crashes that occurred on wet pavement, divided by the number of crashes that occurred on dry pavement.

$$Wet-to-Dry\ ratio = \frac{Tot\_Wet}{Tot\_Dry} \tag{2}$$

where

Tot_Wet is the number of crashes on wet pavement
Tot_Dry is the number of crashes on dry pavement

This ratio is affected by the amount of wet precipitation in a given area as discussed below. For example, a wet-to-dry ratio of 0.50 indicates that half as many crashes occurred on wet pavements as did on dry pavements.

Discussion

If the region where this ratio was observed had half as many rain days as it had dry days, then the risk of being involved in a crash on a wet pavement would be equal to the risk of being involved in a crash on a dry pavement. However, the same wet-to-dry ratio (0.50) would indicate that the risk of a wet pavement crash is twice as high as the risk of a crash on dry pavement, if pavements were wet only 25% of the time. Thus, the wet-to-dry ratio is mainly useful in comparisons between facilities that experience similar rainfall patterns. Under similar rainfall patterns, a high wet-to-dry ratio would indicate facilities that are more prone to wet pavement crashes.

Liquid Precipitation Safety Ratio

LSR was defined based on the following formula:

$$LSR = \frac{\left(\dfrac{Tot\_Wet}{\%\ time\ wet\ pavement}\right)}{\left(\dfrac{Tot\_Dry}{\%\ time\ dry\ pavement}\right)} \tag{3}$$

where

Tot_Wet is the number of crashes on wet pavement
Tot_Dry is the number of crashes on dry pavement
% time wet pavement is the percent of time a pavement is wet
% time dry pavement is the percent of time a pavement is dry

Discussion

The LSR can be thought of as the ratio of the wet pavement crash rate (number of crashes on wet pavement divided by 100 MVMT on wet pavement, see equation (1)) divided by the dry pavement crash rate (number of crashes on dry pavement divided by 100 MVMT on dry pavement).

$$LSR = \frac{\left(\dfrac{Tot\_Wet}{travel\ on\ wet\ pavement}\right)}{\left(\dfrac{Tot\_Dry}{travel\ on\ dry\ pavement}\right)} = \frac{\left(\dfrac{Tot\_Wet}{total\ travel \times percent\ time\ wet\ pavement}\right)}{\left(\dfrac{Tot\_Dry}{total\ travel \times percent\ time\ dry\ pavement}\right)} \tag{4}$$

Since travel on wet (dry) pavement is calculated by multiplying the total vehicular travel in a year by the percent time that a pavement is wet (dry), total travel is eliminated on the right-hand-side of equation (4), and the result is the right-hand-side of equation (3).

The LSR can also be expressed as follows:

$$LSR = \frac{\left(\dfrac{Tot\_Wet}{Tot\_Dry}\right)}{\left(\dfrac{\%\ time\ wet\ \ pavement}{\%\ time\ dry\ \ pavement}\right)} = \frac{(wet-to-dry\ ratio)}{\left(\dfrac{\%\ time\ wet\ \ pavement}{\%\ time\ dry\ \ pavement}\right)} \qquad (5)$$

Interpretation

That is, LSR is the wet-to-dry ratio divided by an adjustment factor that indicates how much more frequently pavements are wet than dry. If the wet-to-dry ratio is equal to the proportion of time pavements are wet to the time they are dry, then LSR = 1.00 and a motorist has an equal chance to be involved in a crash when a pavement is wet as when the pavement is dry. If the wet-to-dry ratio is greater than the denominator, then LSR > 1.00 and the chances of being involved in a crash are greater on wet pavements than dry pavements.

This way, LSR allows comparisons of wet pavement performance across areas with different rainfall patterns. In other words, it provides a measure of how many times more likely one is to be involved in a wet pavement crash, relative to being involved in a dry pavement crash if equal mileage is driven under each of these two pavement conditions. Calculation of LSR requires weather and precipitation information as well as information of how long pavements remain wet after precipitation accumulation on the pavement.

KEY FINDINGS

The focus of the present evaluation was a safety comparison of wet TT and LT high-speed pavements. Rural freeways were chosen as the ideal representatives of such pavements for reasons explained in the Problem Statement and Research Methodology parts of this paper. The majority of available mileage was on rural freeways with an average daily traffic (ADT) less than 60,000 vehicles per day (VPD).

Some urban California freeway statistics, of secondary importance to the present analysis (since lower speeds typically prevail on such pavements), are presented here since LT texture is desired in the urban environment because it generates a lower noise level. Very limited information was available for TT Wisconsin urban freeways and is not presented.

An extensive effort was dedicated to collect and analyze friction number (FN) information for relations to wet pavement crashes. Identified FN databases were not comprehensive enough to be representative of all analyzed freeway mileage; furthermore, FN data showed wild fluctuations from year to year and lane to lane, even for pavements with identical construction years and identical traffic volumes. Thus, no further effort was put into developing a relationship between FN and wet pavement crash rates.

Rural Freeways

Table 1 below presents statistics for TT Wisconsin and LT California rural freeway pavements with ADT less than 60,000 VPD. The two pavement surface types had identical crash rates when crashes over the entire 1991 to 1998 period were analyzed (42 crashes per hundred million vehicle miles of travel). During these years, crash rates were in the 35 to 50 crashes/100MVMT range for TT Wisconsin pavements; those for LT California pavements were in the 41 to 45 crashes/100MVMT range.

The database included approximately 1,460 directional miles of California freeways (730 centerline miles) and 230 directional miles of Wisconsin freeways, a ratio of approximately 6:1. Approximately seven times as much travel occurred on the analyzed California freeways as did on the analyzed Wisconsin freeways over the eight study years (510 vs. 72.6 100 MVMT, respectively). The same ratio held in terms of total analyzed crashes in the two states (21,645 vs. 3,048 crashes, respectively).

When the percent time that pavements were wet in each state is taken into account, TT surfaces outperform LT surfaces since the average LSR value was lower for TT pavements at 2.25 vs. 2.39 for LT pavements. However, this difference was not significant at the 0.99 level of confidence.

Table 1. Wisconsin (Trns PCC) and California (Long PCC) rural freeway statistics 1991–1998 (less than 60,000 VPD)

Year	Wear Surface	Crashes per 100 MVMT	Wet to dry crashes	Total crashes	Liquid safety ratio	Length miles	100 MVMT
1991	Trans. PCC	47	.19	267	2.21	119.7	5.6
	Long. PCC	42	.08	2668	1.62	733.5	63.4
1992	Trans. PCC	40	.28	233	3.18	121.4	5.8
	Long. PCC	41	.11	2608	2.42	733.5	63.4
1993	Trans. PCC	46	.20	391	2.16	166.8	8.5
	Long. PCC	42	.09	2663	1.71	733.5	63.4
1994	Trans. PCC	40	.08	345	1.21	166.8	8.6
	Long. PCC	42	.09	2646	2.60	731.6	63.6
1995	Trans. PCC	40	.12	382	1.51	185.6	9.6
	Long. PCC	44	.14	2720	2.27	730.1	62.5
1996	Trans. PCC	50	.23	522	4.07	196.8	10.4
	Long. PCC	45	.14	2922	2.93	730.8	64.5
1997	Trans. PCC	35	.13	411	2.39	219.7	11.6
	Long. PCC	42	.09	2726	2.66	719.8	64.6
1998	Trans. PCC	40	.14	497	2.16	233.7	12.5
	Long. PCC	42	.19	2692	3.06	715.4	64.4
Overall	Trans. PCC	42	.16	3048	2.25	233.7	72.6
	Long. PCC	42	.12	21645	2.39	728.5	509.7

California Urban Freeways

Aggregate eight year statistics for all California urban freeways with an ADT of less than 60,000 VPD are presented in Table 2 below.

Table 2. California urban freeway statistics 1991–1998 (less than 60,000 VPD)

Wear Surface	Crashes per 100 MVMT	Wet to dry crashes	Total crashes	Liquid safety ratio	Length miles	100 MVMT
Long. PCC	76	.17	23132	3.45	278.4	304.2

Eight year statistics for urban freeways with an ADT of more than 60,000 VPD are presented in Table 3. A very substantial database supported these findings, with approximately 490,000 crashes. Crash rate statistics were exceptionally stable through the analyzed time period.

Table 3. California urban freeway statistics 1991–1998 (more than 60,000 VPD)

Wear Surface	Crashes per 100 MVMT	Wet to dry crashes	Total crashes	Liquid safety ratio	Length miles	100 MVMT
Long. PCC	100	.16	486892	3.22	1114.6	4863.0

CONCLUSIONS

Essential Background

A TWG representing state highway agencies, industry, academia and the FHWA convened in the early 1990s to address tire/pavement noise generated by TT pavements. The TWG published a comprehensive report that stated that "the purpose of surface texture is to reduce the number and severity of wet weather accidents." Analyses over consecutive three- to five-year periods were recommended to determine the wet weather accident rates of different textures and pavement types and the change of accident rates over time for the different textures and pavement types. Reliance on FN as a traffic safety surrogate was discounted by the TWG which stated that "available information supports only a general correlation between friction numbers and wet weather crash rates."

The focus of the present effort was a comparison between LT and Wisconsin TT PCC pavement surface textures. It was desired to compare wet pavement safety performance of these two pavement textures based on extensive crash data spanning multiple years as recommended in the TWG final report.

Since Wisconsin did not have LT pavements, this effort would necessarily have to rely on an inter-state data comparison. Data from neighboring states were desirable, but after an extensive search, the only identified state with adequate LT pavement mileage, crash, and vehicular travel information was California. This information came from the well-documented FHWA-supported HSIS database. Hourly precipitation information was used to calculate the number of hours Wisconsin and California pavements were wet and vehicular miles of travel during these hours. This information was used to provide a fair comparison of TT and LT wet pavement performance across the two states, despite their rainfall pattern differences.

Eight years of data were analyzed in accordance with TWG recommendations in order to provide stable statistics based on the largest available database. Reduced friction under wet pavement conditions was the major TWG safety concern. This concern was addressed by focusing the analysis on rural Wisconsin and California freeways with ADT lower than 60,000 vehicles per day. Lower congestion levels and higher operating speeds are typically present at such facilities, conditions that result in lower friction numbers for any given pavement. If TT and LT pavements differed in safety performance under wet pavement conditions, their differences would be most clearly demonstrated where higher speeds were present. In addition, the freeway environment eliminated the safety influences of intersecting facilities, parked vehicles, pedestrians, intersection right of way control devices, and severe geometry.

Conclusions

1. Use of FN as a freeway pavement safety performance surrogate was shown to be impractical due to wide FN seasonal and spatial variations for similar age pavements experiencing similar levels of traffic.
2. Among rural freeways, Wisconsin TT freeway pavements were found to have similar safety performance to California LT pavements when pavements were wet. This finding was supported by a very substantial database spanning eight years and took into account vehicle miles of travel on wet pavements in each analyzed state. The comparison between high-speed facilities of high design standards provided evidence that the two pavement textures provided similar safety performance under the most adverse conditions: the combination of high operating speeds and wet pavement.

California data on urban freeways were analyzed in order to provide baseline statistics for LT PCC pavement surface applications, should LT pavements be applied in Wisconsin in the future.

3. Among California LT PCC pavements, urban freeways with ADT less than 60,000 VPD had statistically significantly higher crash rates than rural freeways. This finding is consistent with findings across the United States for urban freeways, regardless of pavement texture.
4. Urban LT California freeways with ADT higher than 60,000 VPD had statistically significantly higher crash rates than similar freeways with lower ADT. Traffic volume should be taken into account when analyzing crash rates for a given pavement surface texture.
5. The risk of being involved in a crash on wet pavements based on the LSR was higher on urban LT freeways than rural LT freeways; however, urban freeways with lower ADT had the highest chances of wet pavement accident involvement. Traffic volume should be taken into account when analyzing the risk of being involved in a wet pavement crash on a given pavement surface texture.

Primary conclusion summary

1. LT PCC pavements are expected to display similar wet pavement safety performance to TT PCC pavements on high-speed, high-design standard facilities (rural freeways), with an ADT less than 60,000 VPD.
2. The chances of being involved in a crash on wet LT pavements is higher for urban than rural freeways, a result consistent with crash experience across the United States.
   The chances of being involved in a crash on wet urban LT freeways are higher when the ADT is lower than 60,000 VPD.

RECOMMENDATIONS

1. Based on the findings of no significant wet pavement safety performance differences between LT and TT pavement textures on rural freeways, it is recommended that the comparison between the two types of pavements is extended to include safety performance under winter pavement conditions (when snow or ice are present on the pavement). If no differences are found between the two pavement textures under winter weather conditions, the construction of LT pavements would be recommended for rural Wisconsin freeways, given the benefit of lower levels of traffic-generated noise.
2. LT texture is used extensively by the California Department of Transportation (Caltrans) District 3, which is in charge of an extensive network of snow routes. Contacts with the District 3 Office of Maintenance Equipment and Emergency Operations are recommended in order to address any

winter maintenance concerns related to LT surfaces. Such contacts will identify the types of winter maintenance equipment, materials and policies in force by Caltrans.

3. It was indicated in the literature search that initial attempts at constructing LT textures in California were abandoned due to concerns about the quality of ride for motorcycles and light vehicles. It is recommended that extensive communications are exchanged with departments of transportation that are currently constructing LT textures (especially Caltrans) in order to avoid similar pavement surface construction pitfalls when/if they are first introduced in Wisconsin.
4. The main motivation for the introduction of LT freeway pavements in Wisconsin is their applicability where noise concerns exist, for example in the urban environment. The safety performance of California LT pavements has been addressed herein. Safety performance should be the paramount criterion in choosing a pavement surface texture. Therefore, a comprehensive comparison with the safety performance of pavement textures currently in use in urban Wisconsin freeways is recommended. If one surface texture is shown to be clearly superior in terms of safety, that surface texture should be chosen for application in the urban environment. If LT pavement surfaces are on par with their counterparts, construction of urban freeway LT pavements may be recommended based on noise, durability, material availability, or other pertinent considerations.

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Summary of Thirty Years of TxDOT-Funded Research on Coarse Aggregate Issues in Concrete Paving

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ABSTRACT

This paper summarizes a number of research projects funded by the Texas Department of Transportation (TxDOT) on aggregate-related issues in concrete paving within the past 30 years. The main focus of this paper is on coarse aggregates, which normally occupy more volume than other ingredients in concrete. Research on aggregates in Texas can be traced back to 1928, but in the past 30 years, aggregate research funded by TxDOT has focused mainly on the effects coarse aggregates have on the performance of continuously reinforced concrete pavement (CRCP). In the past 10 years, research on aggregate-related durability issues has also been a focal area. Visual condition survey data have been collected since 1974 on rigid pavements across Texas. A 1978 summary report on field surveys showed significant performance variations between pavements constructed with different aggregates, especially between crushed limestone and siliceous river gravel. From 1986 to 1995, an extensive research program was devoted to establishing a design tool and studying the effects of coarse aggregates on concrete paving. A total of 13 reports were produced. The cracking issue of concrete pavement appears to have been adequately addressed. However, the spalling of CRCP remains a challenge for TxDOT. Several research projects have recently been completed or are underway to address spalling distress in CRCP, including crushed gravel and optimized aggregate gradation. Major findings from these TxDOT-funded research projects and future challenges are summarized.

Key words: coarse aggregate—cracking—CRCP—spalling

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INTRODUCTION

Texas is a big state with a lot of concrete pavement. In the year of 2006, Texas had about 9,400 lane miles of continuously reinforced concrete pavement (CRCP) and about 4,100 lane miles of jointed concrete pavement (JCP). With the population booming in Texas, urban areas are expanding quickly. More and more Texas Department of Transportation (TxDOT) districts are specifying the building of concrete pavements, mostly CRCP, to encounter the public's demand for a safe, economic, and smooth transportation system in urban areas. CRCP is known for its long service life and relatively low maintenance. In Texas, CRC pavement is expected to serve the public for about 30 to 50 years with minimum maintenance interruptions. To meet this goal, it is important for TxDOT to construct good concrete pavement that will last the expected service life.

Many factors are known to influence the behavior of CRC pavement, including pavement thickness, concrete properties, construction practice, environmental variables, and traffic loads. Among these factors, pavement thickness is normally governed by established thickness design procedures. Environmental variables, e.g. humidity and temperature fluctuations, and traffic loads are not controllable. As such, it is perceived that better concrete pavement can be produced if we understand the effects of concrete properties and construction practice on the short-term and long-term behaviors of CRC pavement. Over the years, TxDOT has funded research projects to obtain knowledge of concrete materials and to put the knowledge into practice. It is necessary to review the research performed for TxDOT to determine the future research needs. This paper is the natural product of the need to periodically review and assess current conditions.

Aggregates, occupying 70% to 80% volume of concrete material, is the focus of this paper. The importance of aggregate to the properties of concrete has long been recognized by university researchers and the state highway agency in Texas. As early as 1921, the State Highway Department of Texas (now TxDOT) was involved in concrete aggregate research by furnishing a truck to researchers and paying the freight on samples shipped to the laboratory (Thomas and Parkinson 1928). At that time, concrete technology was still in its infancy. It is not surprising to learn that concrete was mixed by hand and the water-cement ratio was changed to make workable concrete. The researchers found that wear was not correlated to compressive strength of concrete. In addition, they learned that using a larger aggregate size resulted in less wear. Although these conclusions may be quite obvious to us today, the efforts of early research laid a solid foundation of the path, on which we are still traveling, to long-lasting concrete pavements.

RESEARCH EVOLUTION

Like any human effort to understand a natural phenomenon or myth, research supported by TxDOT struggles but yet pushes forward with unyielding endeavors to provide a better concrete pavement system to the public. Although mistakes are unavoidable, researchers are slowly but surely unveiling the ways that concrete properties and construction practices affect CRC pavement performance. During the past 30 years, TxDOT provided full support to concrete aggregate researchers at Texas public universities, as clearly shown in Figure 1. Please note that not all related research projects are listed. For instance, research projects addressing the durability issue of aggregates, e.g. alkali-silica reaction, are not included because this paper concentrates on aggregate specifically in paving concrete. The research embraced laboratory studies and field tests. The importance of test sections for concrete pavement was emphasized throughout all the research projects, and more than ten field test locations around Houston area were included. The total funding for research activities on aggregate in paving concrete is well over five million dollars.

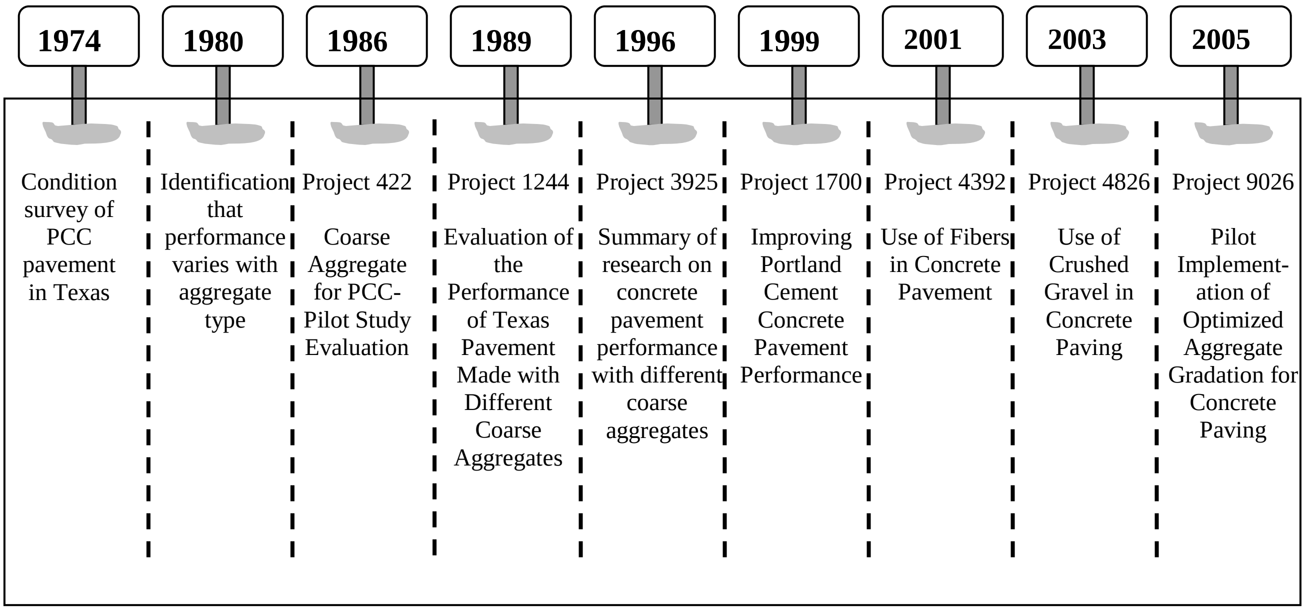


Figure 1. Historical development of research projects related to aggregate in paving concrete

CRC pavement was first placed on Texas highways in the 1950s, primarily in conjunction with the interstate highway program (Hankins, Suh, and McCullough 1991). The Center for Transportation Research (CTR) of the University of Texas at Austin (UT Austin) performed field surveys on ten districts in 1974 and four urban districts in 1976 for TxDOT. Again in 1978, a follow-up survey was performed. The survey was conducted by two persons in one vehicle, traveling on the shoulder at approximately five mph. The distresses recorded were transverse and localized cracks, spalling, pumping, punchouts, and repair patches. One important conclusion was that the use of 8 in. CRCP in Texas for 1 to 6 $\times 10^6$ equivalent 18 kip single-axle application was not adequate (de Velasco and McCullough 1981). Another important finding was that limestone concrete pavement outperformed gravel concrete pavement by showing larger crack spacing (i.e., fewer cracks per unit length) and less spalling. Texas highway engineers started comparing the performance of CRC pavement made with river gravel or crushed limestone aggregate in the 1970s. The surveys and subsequent analysis by CTR affirmed the conjecture that CRC pavement performs differently with various coarse aggregate type. Mainly, CRC pavement with limestone coarse aggregate performs better than those with siliceous river gravel with regard to crack spacings and failures.

To further pursue the coarse aggregate type issue, a pilot research project 422 was sponsored by the Texas State Department of Highways and Public Transportation, now TxDOT. This project mainly focused on the laboratory evaluation of concrete materials made with siliceous river gravel (SRG) and crushed limestone (LS). The measured properties were elastic modulus, thermal expansion coefficient, drying shrinkage, and tensile strength (Green et al. 1987). It was confirmed that mix containing LS aggregates exhibited higher tensile strength (indirect tension test), a higher modulus of elasticity (flexural test), higher flexural strengths, and lower shrinkage values than the mix containing SRG aggregates.

Recognition of the vast challenges to evaluate the effects of various coarse aggregate types on CRC pavement performance led to the joint study of two flagship universities, UT Austin and Texas A&M University for the Project 1244 in the year of 1989. The project spanned over seven years and a total of 13 reports were generated. With a total of eight testing projects, it was conclusively established that concrete material properties (mostly aggregate related), construction practices (e.g., placement temperature and time, curing method), and environmental conditions (humidity and temperature variations) can determine CRC pavement performance.

With the most important factors identified, TxDOT provided more research funds for improving CRC pavement performance. The five-year Project 1700 was purposed to better predict the behavior of concrete in the field and to better evaluate relevant concrete properties in the laboratory. Research included predicting concrete hydration temperature, in situ concrete strength, moisture-related cracking in concrete, and measuring of thermal expansion coefficient of aggregates for concrete. The research results from the study were further evaluated in the field, and it was expected that all the efforts would help TxDOT and the industry manufacture better CRC pavement for Texas highway travelers.

At the same time, TxDOT was willing to experiment with potential approaches that may improve the performance of CRC pavement made with siliceous river gravel aggregate. These examined methods included the use of synthetic and steel fibers in concrete (Folliard et al. 2006) and crushed river gravel (Research Project 4826). Recently, an implementation project was launched to evaluate the potential benefit of optimized aggregate gradation for paving concrete (Project 9026).

The historical development of research projects clearly demonstrates our sharpening understanding of the influences of concrete constituents on concrete materials and the effects of concrete properties on pavement performance. In general, interested properties were first studied in the laboratory and then their effects on CRC pavement performance were evaluated in the field by the use of test sections. Field studies led to more findings, and thus another round of lab testing. TxDOT personnel strove hard to make this process as smooth as possible. The CRC pavement testing locations around the state of Texas are evidence of the endeavor of TxDOT for a better transportation system for the public.

Because there are so many reported results over the span of 30 years, it is impossible to detail every aspect related to aggregate in paving concrete. Thus, the authors decided to present the major findings in four separate parts: laboratory tests, pavement performance, model development, and aggregate classification. The focus will be on the influence of coarse aggregate type, especially LS vs. SRG.

LABORATORY TESTS

Mechanical Properties

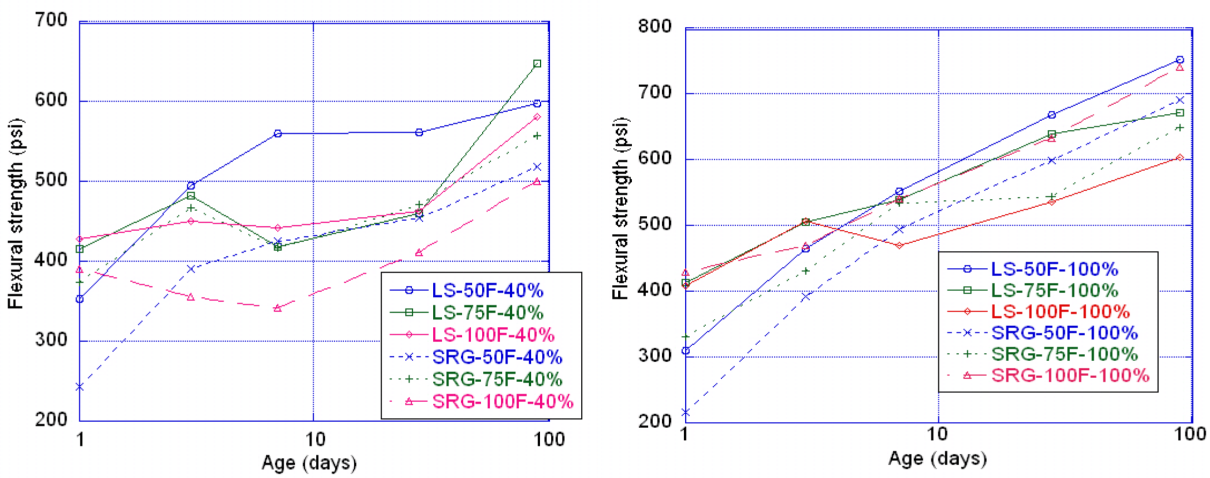
Strength

It is generally accepted that pavement cracks occur when the tensile stress developed in the course of time exceed the concrete strength. Although this premise oversimplifies the complicated cracking initiation phenomenon, it is still applied in current research for pavement.

In one laboratory study, at the curing condition of 75°F temperature and 40% relative humidity (simulating field condition), it was found that with a similar mix design, concrete with limestone coarse aggregate had slightly lower compressive strength up to seven days than that of siliceous river gravel (Dossey and McCullough 1992). At 28 days, the two concrete mixtures had comparable compressive strength.

Splitting tensile test (ASTM C 496) was used to compare concrete mixtures in several projects. This test method measures the indirect tensile strength of concrete. Laboratory tests indicated that the use of LS in concrete was comparable to the use of SRG in respect of tensile strength at early ages (Dossey and McCullough 1992; Dossey, McCullough, and Dumas 1994). For instance, an average splitting tensile strength of 180 psi for SRG mixture and 187 psi for LS mixture at the age of one day. As such, the splitting tensile strength cannot be used to explain the difference of the use of LS and SRG in CRC pavement. As one of the recommendations of Project 1244, this indirect tensile strength was proposed as the quality control parameter in the place of compressive strength (McCullough, Zollinger, and Dossey 1995).

Tensile flexural strength measured following ASTM C 78 is often viewed by researchers as similar to the loading condition of concrete pavement under loading. The evaluation of this property of concrete mixture is often preferred by engineers. A recent study focused on the flexural fatigue strength of concrete mixtures made with LS and SRG aggregate (Suh 2005). It was found that except for very high stress levels ($S > 0.9$), the LS mixtures generally had a higher fatigue resistance than SRG mixtures for a given stress level. However, the specimen size (6 in. × 6 in. × 20 in.) required in the test procedure prevents its popularity among inspectors. A study by Green et al. (1987) revealed the significant effects of curing humidity and temperature on the flexural strength development of concrete mixtures made with LS and SRG aggregates. The results are plotted in Figure 2 below. For 100% relative humidity, only at low temperature (50 °F) the flexural strength of concrete made with LS aggregate was higher than those made with SRG aggregate at all ages, i.e., 1, 3, 7, 28 and 90 days. For high temperature (100°F) the flexural strength of LS concrete was lower than that of SRG concrete after 7 days. However, for the 40% RH curing condition, which was used to simulate the field condition, LS concrete obviously outperformed SRG concrete, especially at low and high temperatures.



(a) 40% relative humidity **(b) 100% relative humidity**

Figure 2. Tensile flexural strength development with age (plotted with data in Green et al. 1987). Example of label: LS-50F-40% -concrete made with LS aggregate, cured at 50°F and 40% RH.

Aggregate type was also found to affect the correlation between compressive and tensile strengths (McCullough, Zollinger, and Dossey 2000). The tensile-flexural strength ratio of concrete was also influenced by the coarse aggregate used (de Velasco and McCullough 1981). Clearly, the prediction of one strength parameter from another should be cautioned.

However, the sole use of strength parameters to predict the performance of CRC pavement is questionable. This opinion is backed up by a 24-year performance review of concrete pavement made with SRG and lightweight coarse aggregate (Won, Hankins, and McCullough 1989). The 28-day concrete properties of the two concretes are summarized in Table 1. The lightweight coarse aggregate was prepared by heating shale at high temperatures and the maximum size was approximately ¾ inch. In comparison, the conventional SRG aggregate used had a maximum size of approximately 1½ inches. The test slabs consisted of a 6 in. thick concrete on a subbase of 6 in. of cement-stabilized oyster shell. Lightweight concrete performed better by having no failures, relatively large crack spacings, and a good appearance after 24 years even though the compressive, tensile, and flexural strengths of the lightweight concrete were all lower than the SRG counterpart. Clearly, a stronger concrete does not necessarily provide a better concrete pavement.

Table 1. 28 day mechanical properties of concrete made with different coarse aggregate (Won et al. 1989)

Mechanical property	Concrete with lightweight	Concrete with SRG aggregate
Compressive strength, psi	3828	4313
Tensile strength, psi	312	488
Modulus of elasticity, psi	3.05×10^6	7.8×10^6
Flexural strength, psi	607	643

Elastic Modulus

Elastic modulus is expected to affect early-age cracking pattern by influencing the stress due to drying shrinkage and thermal fluctuations (Suh, Hankins, and McCullough 1992). The relatively larger crack spacing reported for lightweight concrete pavement (Won, Hankins, and McCullough 1989) might be partially attributed to the lower elastic modulus of lightweight concrete (3.05×10^6 psi), in addition to the low thermal expansion coefficient.

However, the measurement of elastic modulus itself is an interesting evolution process. For Project 422, the concrete elastic modulus values were obtained by the use of a beam tested under third point loading, following ASTM C 78 (Green, Carrasquillo, and McCullough 1987). The slope of the cord connecting the two points on the deflection-stress curve at 20% and 50% of ultimate stress values was used to calculate the modulus. For Project 1244, the modulus was measured in compression testing (Dossey and McCullough 1992) according to ASTM C 469. Because different procedures were followed, the observations were contradicting. In flexural testing, the elastic modulus of LS concrete was found to be higher than SRG concrete at early ages, up to 7 days. On the contrary, the compressive elastic modulus of LS concrete was found lower than SRG concrete for all tested ages, up to 28 days.

It is felt that the measurement of elastic modulus of concrete under direct tensile would be more meaningful when concrete pavement cracking is concerned. However, unless some breakthrough occurs, the testing procedure of direct tension is too complicated for practical application.

Toughness

With the increase of pavement thickness (larger than 8 in.), tying pavement with its shoulder and the requirement of 1 in. asphalt bond breaker and better base (4 in. hot mix base or 6 in. cement-treated base) in Texas, the spalling of concrete became the major distress of CRC pavement, taking the place of punchout. Two-year monitoring of 32 CRCP test sections with four construction projects built in 1989 in Houston area raised the issue of spalling with concrete made with SRG aggregate (Otero Jimenez, McCullough, and Hankins 1992). Subsequently, TxDOT has been sponsoring research projects focusing on reducing spalling in CRC pavement. Implementation projects on spalling repair were also funded.

Severe spalling of CRC pavement is currently considered by researchers as being the development of delaminations in concrete pavement before the traffic load (Senadheera and Zollinger 1996). Delamination is often found passing through the interface between SRG aggregate and mortar in spalled pavement. Two remedy methods were proposed based on the above observation and two separate research projects were sponsored by TxDOT. One method is to increase bond strength between coarse aggregate (e.g. SRG) and mortar. It is hoped by doing so the delamination will be reduced. The method was thoroughly studied in Project 0-4826. The fracture toughness test was used in this project. The other method is to prevent the crack propagation by the use of steel fibers, recognizing cracking/delamination is not totally avoidable. This approach was evaluated in research project 0-4392 (Folliard et al. 2006). Flexural toughness of concrete mixtures was evaluated following ASTM C 1018.

Texas Transportation Institute (TTI) researchers used two approaches to evaluate the fracture toughness of concrete mixture made with different coarse aggregate. The RILEM procedure of size-effect fracture test on notched concrete beams was originally used (Senadheera and Zollinger 1996). The results indicated that concrete with SRG aggregate had higher fracture toughness (0.41 MPa m) than that with LS aggregate (0.37 MPa m) at 24 hours. However, the brittleness of siliceous gravel concrete was found to be more than twice that of limestone concrete, and the effective length of the fracture process zone for limestone concrete was more than twice that for siliceous gravel concrete. In Project 4826, TTI researchers turned to the test procedure proposed by Tang et al. (1999) on notched Brazilian splitting tensile specimens. However, for concrete with limestone aggregate at 24 hours, the fracture toughness was found to be about 0.45 MPa$\sqrt{m}$, while for SRG concrete the value was 0.23 MPa$\sqrt{m}$. Again, as for the case of elastic modulus, the use of different test procedures generated different values and trend. More research is needed to clarify the discrepancy.

CTR researchers found that the use of steel and synthetic fibers with different dosages increased the flexural toughness for concrete mixtures, especially for those with SRG aggregate (Folliard et al. 2006). However, in the study, concrete with SRG aggregate demonstrated higher flexural toughness indices than LS counterpart in all categories. It is also interesting to note that ASTM C 1018-97 is now withdrawn as an ASTM standard test.

Physical Properties

Coefficient of Thermal Expansion

The importance of the coefficient of thermal expansion (CoTE) of concrete to CRC pavement has long been recognized. The coarse aggregate is the major part of concrete volume and thus is the major factor of concrete CoTE. However, the lack of an accurate and repeatable test method on aggregate CoTE prevented its wide application in the construction industry. Currently, the evaluation of CoTE value on a concrete mixture can be regularly performed by TxDOT, using modified AASHTO TP 60 (Won 2005). In a recent study, TxDOT tested a total of 94 coarse aggregate using the same mix design (Du and Lukefahr 2007). All concrete mixtures with LS aggregate had a CoTE value less than 5.5 in/in/°F and only 20% of test SRG concrete mixtures had such low thermal expansion. Noticeably, about 50% of tested SRG concrete mixtures had a CoTE smaller than 6.0 in/in/°F. Lightweight aggregate concrete tested in this study showed a low CoTE value of 4.8 in/in/°F.

The prototype CoTE measurement was quite rudimentary about 20 years ago. Two strain gages were placed on each specimen, and deformations were recorded for every 30°F curing temperature change from 45°F to 135°F and backward to 45°F (Green et al. 1987). However, even with current technology, these tests may still generate contradicting results. For instance, it was found in Project 7-3925 that gap-graded concrete mixtures generally had lower CoTE values (McCullough, Zollinger, and Dossey 2000). A recent research publication found that optimized gradation reduced the CoTE value of concrete by 0.81 in/in/°F (Kim and Won 2007). TTI researchers attempted to directly measure CoTE of coarse aggregate using dilatometer in a recent project (Mukhopadhyay, Neekhra, and Zollinger 2007). However, TxDOT is still in the stage of evaluating the device and procedure.

Drying Shrinkage

The importance of drying shrinkage to CRC pavement has long been recognized by researchers. As early as 1981, the effect of aggregate type on concrete drying shrinkage was clearly pointed out (de Velasco and McCullough 1981). It was believed that the increase in crack width of CRC pavement is a function of the residual shrinkage (drying shrinkage after formation of the crack) and an early-age crack would have higher residual shrinkage than a later crack, a fact that results in the greater width (Suh et al. 1992). At low (50°F) or high (100°F) temperatures and a relative humidity of 40%, Green et al. (1987) found that LS concrete shrank, measured on 6 x 12 in. cylinders, significantly slower than SRG concrete, probably due to the higher bond strength between coarse aggregate and mortar in LS concrete. However, for the condition of 75°F, the differences were negligible. On the contrary, a later study found limestone concrete had a higher drying shrinkage at 256 days than SRG concrete (Dossey and McCullough 1992). It is not easy to explain these observations. This actually raised a very interesting concern. Is the standard curing temperature around 73°F a valid approach for evaluating drying shrinkage for paving concrete with different coarse aggregate? In addition, can coarse aggregate be adequately characterized by its type? Or do we have to evaluate important properties of a specific aggregate source even though the aggregate type is known?

Durability Properties

The aggregate-related durability issue was drawn to the attention of TxDOT more than 20 years ago (Carrasquillo and Farbiarz, 1989). In the 1990s, alkali-silica reaction (ASR) was identified with Texas concrete pavement and structures. A five-year research project, 0-4085, “Preventing ASR/DEF in New Concrete,” was initiated in 2000, and many important findings were generated and have been used in TxDOT specifications. The research evaluated several ASTM standard tests methods on assessing ASR. The accelerated mortar bar test (AMBT) ASTM C 1260, is deemed as a reasonable indicator of aggregate reactivity or a reasonable means of assessing various mitigation measures. However, its severity and sometimes false passing of reactive aggregates cast doubt on its effectiveness (Folliard et al. 2006). One-year testing of ASTM C 1293 is preferred, but in many cases we cannot afford the waiting time. As for delayed ettringite formation (DEF), researchers proposed a maximum internal concrete temperature of 158°F to effectively prevent the occurrence of DEF. Alkali-carbonate reaction was not emphasized because the potentially susceptible rocks are not common in Texas.

PAVEMENT PERFORMANCE

Introduction

In Texas, test sections have been used to validate laboratory research findings and new technologies to better understand pavement behaviors with proposed changes of mix design, steel placement, and construction practices. Cracking spacing, crack width, concrete and steel stress, punchout, and spalling are generally recognized as pavement performance indicators (McCullough, Zollinger, and Dossey 2000). Stresses in concrete and steel are important, but they are difficult to evaluate in situ. In the passing years, very little research had focused on this topic. Punchouts are no longer a major distress for CRC pavement in Texas, and this paper will not discuss it here. The remaining section emphasizes cracking pattern of CRC pavement and its spalling hazard.

Cracking Patterns

Crack Control

In order for CRC pavement to perform satisfactorily, it is generally agreed that crack spacings should fall between 3.5 and 8 ft. (Otero Jimenez, McCullough and Hankins 1992). Before the 1990s, the steel reinforcing ratio in CRCP was thought to be the main factor controlling cracking spacing because steel has a lower CoTE value than concrete. While SRG concrete has a higher CoTE, it was thought by researchers and engineers that a lower steel reinforcing ratio in SRG pavement may generate similar cracking spacings and thus similar crack widths as those in LS pavement. Based on this methodology and using software model CRCP-4, a design standard, TxDOT CRCP-89B, was originated but lasted only for one month (McCullough, Zollinger and Dossey 2000). This standard put unfair penalty to LS aggregate by requiring more reinforcing steel, and test sections did not support the premise of similar cracking spacing and crack width. Instead, SRG sections had shorter cracking spacing and wider crack width than LS sections did two years after construction (Otero Jimenez, McCullough and Hankins 1992). This highlights the importance of carefully examining the results predicted by computer models before putting them into practice.

Further research found that a variety of factors could affect the cracking pattern of CRC pavement including placement temperature (setting temperature), construction season, curing techniques, crack inducers, saw cutting, and aggregate blend (McCullough, Zollinger and Dossey 2000). For instance, substantially more failures occurred when the concrete placement temperature was in the range of 90°F to 99°F. Because there are many factors affecting cracking pattern of CRC pavement, the control of crack spacing and width is still an art to be mastered.

Crack Initiation and Stabilization

Before opening to traffic, cracks are normally observed in CRC pavement at early ages due to volume change. This emphasizes the importance of strength and stress developments of concrete during the first several days for CRC pavement. Incidentally, many factors are influencing the strength and stress of concrete at the same time. Loss of moisture and temperature fluctuations of fresh and hardened concrete are identified as the most important controllable parameters for early-age cracking. Tensile stresses are developed due to drying shrinkage and temperature drops. When the strength is exceeded by the stress (or certain crack initiation criterion is met), cracks form, subsequently propagate, and finally stabilize. They are determined by environmental loads (temperature and moisture changes), traffic loads, and physical restraints (external and internal).

Researchers found that almost without exception, cracks occurred when the slab temperature dropped significantly (Suh, Hankins, and McCullough 1992). The first concrete crack of the winter construction occurred much later than in summer construction, as a result of a smaller temperature drop. LS concrete sections had fewer and delayed cracks than corresponding SRG sections, possibly due to lower CoTE value, lower temperature rise, larger strain capacity, and lower elastic modulus of LS concrete. The differences were not as significant for winter construction as for summer construction. In addition, it was believed that the steel did not have as much influence on cracking during the early ages because the bond between concrete and steel may not have been fully developed. Many of the transverse cracks occurred over the transverse steel bars, a phenomenon that was more significant in the sections having double-layered steel (Suh, Hankins, and McCullough 1992).

Another interesting observation in one research project was that all of the surface cracks could be traced to cracks observed on the pavement edge, though many cracks observed on the pavement edge did not reach the pavement top surface (McCullough, Zollinger, and Allison 1993). This calls for the evaluation of the effect of applying curing compound on exposed edge concrete.

Suh et al. (1992) identified the location pattern of new cracks in a slab segment, defined by existing cracks. They found that for a slab segment longer than about eight ft., the occurrence of new cracks is quite random. However, some trend was observed if the segment length was less than eight ft. The new crack, if any, would be almost at the middle length if the segment and was about three ft. or shorter. For a segment with a length between about three and eight ft., the cracks would be closer to the ends as the length increased. Several research projects revealed that the crack pattern evolved quickly in the first month and 80% to 90% of the stable pattern developed between 28 and 90 days after construction. The final surface cracking patterns typically took three years to stabilize (McCullough, Zollinger, and Allison 1993). Note that the crack distribution of SRG test sections at five days for the summer placement is poorer than that at 2,600 days for the winter placement (McCullough, Zollinger, and Dossey 2000).

Crack Spacing

Crack spacing of CRC pavement is probably the most important parameter affecting its long-term performance. AASHTO has recommended a desirable spacing between three and a half (or three by some researchers) and eight ft. The lower limit is to avoid the high tensile stress of concrete at the transverse cracks, which may cause punchouts and spallings. The upper length is to prevent the high stress and yielding in longitudinal steel. Crack spacings are of bimodal distribution and average spacing is discussed here for convenience.

Various factors have been identified to affect crack spacings, including concrete material properties and environmental conditions. Concrete tends to contract due to drying shrinkage and temperature drops, but the internal (longitudinal and transverse steel) and external restraints (underlying base) through the interface resist this contraction. If the resulted complex stress conditions exceed certain criteria, cracks form. The following discussions are qualitative because of the nature of this paper.

Since stress is proportional to strain (due to drying and temperature drop from setting temperature) and elastic modulus, the crack spacing is expected to be larger if the concrete mixture has a lower CoTE, a lower modulus, a slower drying process, smaller temperature drops, and fewer contraction restraints. A high-strength concrete is not necessarily beneficial because the elastic modulus will also be higher, and the setting temperature may also be higher, which means a larger temperature drop at later ages. Researchers have found that in colder areas, crack spacings appear to be smaller for both LS and SRG aggregates (de Velasco and McCullough 1981).

The type of coarse aggregate in concrete is well known as the important factor affecting crack spacing. Researchers have found that in areas with similar temperatures, limestone pavements have larger crack spacing than gravel concrete pavement (de Velasco and McCullough 1981). LS pavements tend to stabilize at a crack spacing of around six ft and there is a much lower spacing of two to three ft. for SRG pavements (McCullough, Zollinger, and Dossey 2000). After 24 years, the lightweight aggregate test sections had an average crack spacing larger than seven feet (Won, Hankins, and McCullough 1989). The differences in elastic modulus, CoTE value, and brittleness of the concrete with different coarse aggregates are often resorted to for the explanation. However, this approach is challenged by the tests using blended aggregate of LS and SRG. Although it is conclusively demonstrated in the laboratory that the properties of blended aggregate concrete vary with blending ratio, the test sections could not confirm its benefit in crack spacing because one test project showed that there was little difference at 640 days between using SRG and a blend (50/50 by mass) of SRG and LS, as shown in Figure 3 (McCullough, Zollinger, and Dossey 2000).

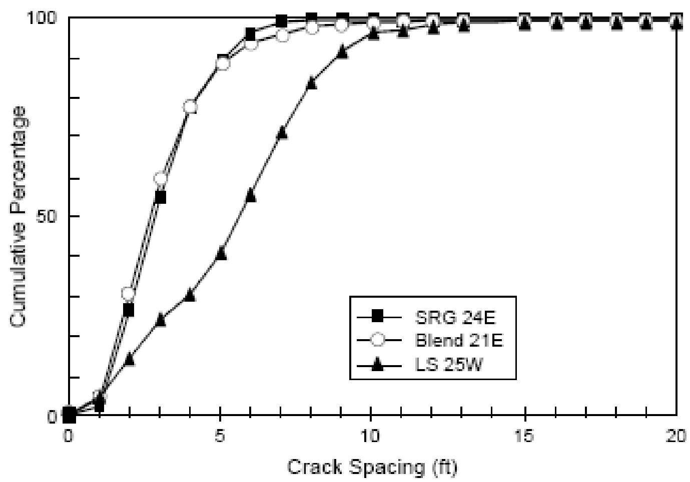


Figure 3. Comparison of crack spacing distributions for LS, SRG, and blended coarse aggregate at 640 days (McCullough, Zollinger, and Dossey 2000)

To understand the effect of temperature drops on crack spacing, we need to follow the concrete setting and hardening process. During this process, the heat of cement hydration is generated, and concrete will usually have a temperature rise above the ambient. The temperature peak normally occurs six to eight hours after concrete placement. In addition, there is little volume expansion of concrete from plastic to solid state, and thus concrete is under slight compression initially. As such, after passing the peak temperature, concrete will contract due to cooling and drying. At a certain temperature, concrete should be at a zero-stress condition, ideally. This temperature is called the zero-stress temperature, and it is used to calculate the stress due to temperature drops. Apparently, for the same low annual temperature, the temperature drop will be lower if the zero-stress temperature is decreased. This highlights the importance of controlling concrete placement temperature and paving time. Furthermore, there is a decrease of crack spacing when placed in the morning than in the afternoon for blended aggregate (Fig 3.9, McCullough, Zollinger, and Dossey 2000). For sections placed at the same time and cured the same way, LS concrete had temperatures about 10°F lower than SRG concrete. The probable explanation lies in the fact that the heat capacity of calcium carbonate is about 12% higher than that of silicon dioxide.

The use of skewed transverse steel or steel fibers only had limited effects on crack spacing.

Crack Width

When CRC pavement cracks, the monolithic concrete slab is divided into discontinuous segments, if the top surface is considered. The cracked surfaces, once formed, separate with time due to releasing of tension stress, drying shrinkage, loading deformation, creep, temperature drops, substrate concrete restraints, and other factors, with certain randomness. One researcher found that the coefficient of variation for crack widths in the same crack was about 30% (Otero Jimenez, McCullough, and Hankins 1992). It is critical that the crack width is small enough to provide full load transfer and prevent water flow through the cracks. AASHTO has a maximum limit of 0.04 in., and researchers pointed out a crack width less than 0.025 in. at 32°F is desirable (McCullough, Zollinger, and Dossey 2000). Fortunately, no test sections in Texas had cracks wider than the limit of 0.04 in.

Thermal contraction, residual drying shrinkage after cracking, creep and relaxation, and possibly longitudinal steel reinforcement are significant factors affecting the crack width of CRC pavement. Lightweight and LS aggregate concrete pavements were found having smaller crack widths than SRG pavements, even though the former two had longer crack spacing. The explanations included smaller deviations from zero-stress temperature, lower CoTE values, and delayed initial cracking for lightweight and LS concrete. The later the cracking happens in CRC pavement, the more creep compensates for thermal contraction and moisture drying. Crack width is not necessarily proportional to crack spacing because beyond some certain length, the contraction of the upper concrete should be totally restrained by the substrate, including longitudinal steel. The above observations also explain why summer construction produces wider cracks than winter construction. Winter construction is featured with smaller temperature rise, slowing drying, late cracking, and longer crack spacing.

One interesting observation in one project was that the thicker the CRC pavement, the smaller the crack width (McCullough, Zollinger, and Dossey 2000). This is contrary to the observation on reinforced beams where the thicker the cover, the wider the crack. It is possible that the thicker the pavement, the slower the drying of surface concrete due to moisture supplied from substrate, yielding more creeping of concrete into the equation. It is known that poor curing causes wider cracks. Another explanation lies in the fact that if we keep the reinforcing ratio the same, the increase of pavement thickness actually reduces the ratio of steel bond surface area to surrounding concrete volume.

Crack Meandering

Crack meandering in CRC pavement is associated with the crack propagation in concrete. Early-age (one or two days) crack patterns have a tendency to be meandering, with relatively wide crack widths. Cracks in lightweight and LS aggregate concrete pavements were found to be straight with little meandering. To explain the above phenomena, it is believed by the authors of this paper that the coarse aggregate has a determining effect. Lightweight and LS aggregate concrete mixtures have weaker aggregate and stronger bond strength between more aggregate particles, as clearly shown in splitting tensile test, than those of SRG concrete. As a result, existing cracks can propagate usually through lightweight and LS aggregate particles but seldom through SRG particles. To circumvent the SRG particles, cracks may deviate from their straightness and meandering results. When cracks happen at an early age (most often for SRG), the bond strength is weaker and, subsequently, the chances for the crack to go through aggregate particles are lower, which means more potential for meandering.

Spalling of CRCP

Spalling is currently the major distress of CRC pavement in Texas. Several research projects were launched to study spalling mechanisms and potential measures to prevent or minimize its happening. In addition, an implementation project on spall repair of CRC pavement was sponsored. Despite the above endeavors, our understanding on spalling is still limited, and the following discussions are preliminary. To have a brief taste of the complexity of the spalling issue, consider that limestone and lightweight coarse aggregate concrete were found to have much less spalling distress in field surveys. For instance, SRG concrete pavement could have 14 times more spalled cracks than LS concrete pavement for the same length. Also consider that spalling can happen as early as 12 months, or as late as 10 years, or never.

The research on spalling dates back to the 1960's. In the 1970's, the survey by CTR on Texas CRC pavements found that warmer districts had the lowest percentages of spalling in most of the cases (de Velasco and McCullough 1981). There was also a trend for more spalling with larger crack spacings. In addition, LS concrete had less spalling compared to SRG concrete for similar crack spacing. Another project found no spalling in lightweight aggregate test sections after 24 years (Won, Hankins, and McCullough 1989). Lower elastic modulus, stronger aggregate-mortar bond, and possibly higher strength of LS concrete were proposed to explain the differences.

Spalling is often rated as minor and severe. A minor spall is defined as edge cracking where the loss of material has formed a spall of one-half in. width or less, the cause of which could be local deformation or debris clogging. Before the 1990s, severe spalling, which are so wide that a smooth ride is affected, was attributed to poor construction operations, for instance, over vibration of concrete. The occurrence of severe spalling was found to be less variable with age, traffic, location (de Velasco and McCullough 1981). From the early 1990s, TxDOT has been funding a number of research projects related to spalling in CRC pavements. Researchers further classified spalling as "deflecting spalling" and "delamination spalling" (McCullough, Zollinger, and Dossey 2000). Deflection spalling occurs at the cracks due to repeated traffic load and temperature variations. With existing delaminations in depths of one to three in. from the surface, delamination spalling generally starts at the crack and progresses away from cracks for some distance. Delamination spalling will be the focus of the following discussions.

Placement season and aggregate type are found to be the two most important variables in spalling development. Delamination of concrete can occur in the early ages before opening the pavement to traffic. Shear and vertical stresses caused by moisture gradient due to drying and, to a lesser degree, temperature gradient in the vertical direction are used to explain the observation that CRC pavements placed in summer showed more spalling than those placed in winter. The importance of timely and adequate curing is highlighted. Because delaminations start from cracks between coarse aggregate particles and mortar, the integrity of interfacial transition zone is essential to minimize delamination development. Limestone aggregates are usually thought to have a better bonding with the mortar phase than SRG aggregate, especially at early ages. Stronger bond strength and lower CoTE are used to account for the outstanding performance of LS concrete. TxDOT Project 4826, "Use of Crushed Gravel in Concrete Paving," attempted to improve the spalling resistance of SRG concrete by generating rougher aggregate surface (crushing) and producing stronger interface (holding back mixing water). Unfortunately, field tests are still inconclusive at this time, and a longer evaluation will be necessary.

The use of fiber-reinforced concrete in CRC pavement is still under evaluation with test sections.

MODEL DEVELOPMENT

Several models for use in CRC pavement have been developed and are still under development through the course of time. Accompanying our more and deeper understanding of material properties and pavement behaviors, models are changing from purely empirical to empirical-mechanistic. More and more relevant factors, including aggregate type and temperature variations are put into the models. The development of the software CRCP series is the perfect example. The first version CRCP-1 was introduced in 1975 and is now evolved to CRCP-10. The modeling of pavement deformation under traffic load was expanded from one dimension to three dimensions, using the finite element method.

Researchers also attempted to predict the concrete properties, including CoTE, when different aggregates are used. The prediction was purely based on the oxide contents of coarse aggregate, using programs CHEM or CHEM2 (Dossey, McCullough, and Dumas 1994). However, this accuracy of this approach still requires validation.

AGGREGATE CLASSIFICATION

Fully recognizing the importance of coarse aggregate for paving concrete, TTI researchers proposed a classification system to TxDOT (Peapully, Zollinger, and McCullough 1994). Physical, chemical, mechanical, and thermal properties of coarse aggregates were discussed, and their importance for concrete pavement was evaluated. With the goal of being simple and easy to use, visual examinations in the field and laboratory evaluation were proposed. Visual examination is to provide preliminary information on coarse aggregate, including nominal size, origin, surface type (crushed or not), color, and presence of any obvious impurities. Laboratory evaluation requires detailed investigation of aggregate properties.

Unfortunately, a lot of aggregate information is needed to establish a usable classification system for coarse aggregates, and the proposed system is not applicable now.

SUMMARY AND SUGGESTIONS

The effects of coarse aggregate on the short-term and long-term performance of CRC pavements, especially limestone and siliceous river gravel, have long been known to engineers and researchers. However, how the coarse aggregate influences concrete properties and thus pavement performance is more complicated than it appears. Although in the past 30 years, intensive and comprehensive research was performed in Texas to understand pavement performance with respect to coarse aggregate and to further mitigate the adverse influence of SRG coarse aggregate, we are not yet in a comfortable position to specify coarse aggregate simply based on its physical, mechanical, and chemical properties.

There are several complex phenomena in CRC pavements that we are still trying to comprehend. For instance, the spalling issue was thought to be related to stresses due to moisture loss. However, the better performance of LS concrete could not be explained. In addition, TxDOT is currently sponsoring research on mid-depth cracking issue, which appears similar to punchout at the surface but the detachment of concrete blocks happen at the mid-depth of pavement. Another case is the evaluation of bond strength between coarse aggregate and mortar. Concrete should be treated as a composite mixture rather than a homogeneous material. It is felt that research should be focusing on this direction.

With the development of electronic devices, more and more fundamental properties of aggregate and concrete can be easily evaluated in the laboratory. One outstanding case is the thermal properties of aggregate. The heat capacity and thermal conductivity of the aggregate are important for CRC pavement because they influence the temperature rise and zero-stress temperature. It is anticipated that more fundamental properties of aggregate will be used to determine its acceptance for a specific construction project.

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Weather Information for Surface Transportation (WIST): Update on Weather Impacts and WIST Progress

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ABSTRACT

Just over five years ago, the National Oceanic and Atmospheric Administration (NOAA) released its first report on improving surface transportation safety and cost efficiency through improved weather and climate information products. The 2002 report, *Weather Information for Surface Transportation: A National Needs Assessment Report*, which sought to provide the roadmap for the nation's surface weather activities, helped launch a rapid expansion of interagency, intergovernmental, and public-private efforts to enhance safety and mitigate the economic impacts of what was called "surface transportation weather." The six sectors of the surface transportation community affected by this report are roadway, railway, transit, pipeline, marine transportation system, and airport ground operations.

This update of the 2002 report focuses on the status of surface transportation weather issues in the United States and the results achieved since the first report. This update delineates the positive impact of the increased focus and attention on the functional area of surface transportation weather, which has helped reduce fatalities and injuries, improve operations efficiency, and reduce property damage on the nation's transportation systems, thus enhancing the nation's economy. While challenges remain in terms of gathering the data needed to distinctly tally the safety and economic impacts of weather on every transportation sector, and in terms of demonstrating the difference that timely, targeted weather information can make in enhancing safety and economic benefits, the available data does indicate progress. This update also details efforts to develop a federal surface transportation weather research and development program plan.

Key words: research—safety—surface transportation—weather

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FEDERAL METEOROLOGICAL COMMUNITY ORGANIZATION

Office of the Federal Coordinator for Meteorological Services and Supporting Research Mission

The mission of the Office of the Federal Coordinator for Meteorological Services and Supporting Research (OFCM) is "to ensure the effective use of federal meteorological resources by leading the systematic coordination of operational weather requirements and services, and supporting research, among the federal agencies." The key point is the focus on systematic coordination among the federal agencies and their stakeholders.

Federal Meteorological Coordinating Infrastructure

The OFCM executes this mission through the Federal Meteorological Coordinating Infrastructure, as depicted in Figure 1. The overall policy guidance is provided by the Federal Committee for Meteorological Services and Supporting Research. Fifteen federal departments and agencies are currently engaged in meteorological activities and participate in the OFCM's coordination infrastructure. The OFCM carries out its tasks through an interagency staff working with representatives from the federal agencies, who lead and serve on program councils, committees, working groups, and joint action groups. This infrastructure supports all of the federal agencies that are engaged in meteorological activities or that have a need for meteorological services. The OFCM also assesses the adequacy of the total federal meteorology program and reviews current and proposed programs to identify opportunities for improved efficiency, reliability, and cost avoidance through coordinated actions and integrated programs. The OFCM also provides analyses, summaries, and evaluations that provide a factual basis for federal agencies and the executive and legislative branches to make appropriate decisions related to the allocation of funds. In this regard, the OFCM recently made significant contributions to the interagency meteorology community in areas such as natural disaster reduction (hurricanes and post-storm data acquisition), wildland fires, space weather, phased array radar, and weather information for surface transportation.

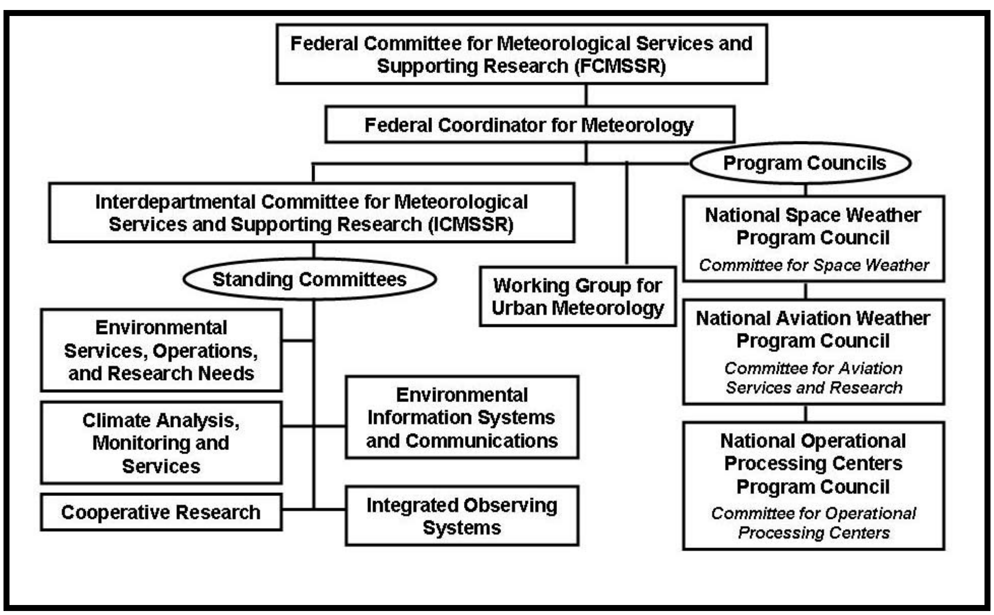

Figure 1. Federal Meteorological Coordination Infrastructure

Weather Information for Surface Transportation

Based on the OFCM-sponsored document, *Weather Information for Surface Transportation: A National Needs Assessment Report* (2002), it became apparent to the federal meteorological community's leadership that it needed to conduct an in-depth coordination and synchronization of all requirements, research and development needs, and services regarding weather information for surface transportation (WIST). Therefore, a Working Group for Weather Information for Surface Transportation (WG/WIST) was chartered and aligned under the Committee for Environmental Services, Operations, and Research Needs, within the Federal Meteorological Coordinating Infrastructure, to coordinate and synchronize WIST.

IMPACT OF WEATHER ON SURFACE TRANSPORTATION SYSTEMS

It is common knowledge that weather has a significant impact on the nation's surface transportation systems. For the roadway sector, weather-related accidents and weather events, as indicated by safety and cost data estimates, lead to approximately 7,300 fatalities per year, 713,537 injuries, and $42 billion in economic costs. Moreover, nearly 25% of nonrecurrent delays on freeways are weather-related. In the railway sector, annual average weather-related fatalities are much lower than for roadways. However, between 1995 and 2005, 865 weather-related accidents or incidents occurred on America's railways, causing 8 deaths, 1,242 injuries, and property damage of more than $189 million. Most of these weather-related deaths (62.5%) and injuries (91.1%) between 1995 and 2005 were associated with accidents or incidents associated with extreme temperature variations. In the marine transportation system sector, between 1996 and 2000 weather-related events accounted for 11% of marine transportation accidents and 3.6% of all recreational boating accidents. In the pipeline systems sector, between 2002 and 2005 the pipeline systems sector experienced 4 weather-related fatalities and 14 weather-related injuries, all occurring in 2005. In 2005, the number of pipeline incidents caused by natural forces (defined as heavy rains/floods, high winds, lightning, temperature, earth movement, and various other causes) increased dramatically due to hurricane damage. Three of the fatalities were attributed to incidents caused by temperature, and one was due to high winds; all four incidents were in natural gas distribution activities.

Finally, weather's impact on transportation congestion is large. During remarks to the National Retail Federation in May 2006, Secretary of Transportation Norman Mineta called congestion one of the single largest threats to the economy and announced a new national initiative to tackle highway, freight, and aviation congestion. In May 2006, the Department of Transportation released the National Strategy to Reduce Congestion on America's Transportation Network, which attributed 15% of all transportation system congestion to the adverse weather conditions of snow, ice, and fog.

PROGRESS BEING MADE THROUGH WIST

There is preliminary evidence that, since the first WIST report was released in 2002, some improvement has been achieved in terms of lessening the impact of weather on the surface transportation system. Although data collection and incident monitoring is still limited, the available data do show that potentially some progress is being made, specifically in the following areas:

- On the nation's roadways, weather-related crash injuries declined by 3.5% (21,023 fewer injuries) in the first two years following the release of the WIST report in 2002. During the same period, vehicle-miles driven increased by 3.7%. The 21,023 fewer injuries equate to about $0.5 billion saved in direct and indirect economic costs.

- From 2002 to 2004, weather-related recreational boating accidents decreased from 228 (with 66 fatalities) to 178 (with 43 fatalities). In 2002, weather dropped out of the coast guard’s top ten contributing factors for recreational accidents. While weather returned to the list in 2005, there were fewer accidents than in past years.
- Surveys of users of state 511 road information services show that users want and use information about weather conditions that affect their route of travel. Most of these WIST users have altered their routes or their travel plans to avoid weather-related hazards or delays. Local television and radio stations now routinely carry combined traffic and weather updates for their broadcast area.
- Roadway freight lines are equipping their trucks to receive National Oceanic and Atmospheric Administration (NOAA) weather radio anywhere on the nation’s highways. Automobile manufacturers are offering new cars equipped with radios that can receive NOAA weather radio.

The successes achieved are due to the combined efforts of many federal agencies, state and local authorities and transportation departments, the university research and development community, professional organizations, the news media, and partners in industry who provide or use WIST services and products. When statewide transportation incident reporting systems are implemented, we will be better able to monitor, assess, and manage transportation weather risks, as well as evaluate the benefits of WIST-informed transportation decisions. Research and development programs are currently improving warnings and decision support systems, implementing weather-responsive traffic management in communities, and providing the observational support necessary for location-specific WIST.

CHALLENGES FACING WIST

Focusing Resources

In today’s constrained federal budget environment, we must focus limited resources on the top-priority needs in research and development and application development regarding surface transportation weather. Some of these key needs include the following:

- Lessening weather’s impact in terms of causing congestion
- Meeting travelers’ need for timely, local weather information
- Improving access to WIST before and during travel
- Enhancing surface transportation weather observation collection
- Incorporating WIST into warning and decision support processes
- Increasing user understanding of the use of WIST products

Partnering and Leveraging

With limited budget resources, moreover, we will be challenged to take advantage of all opportunities to partner and leverage other research and development and application development activities, even those outside the surface transportation weather community, to meet WIST needs. Some opportunities for such partnering and leveraging may be found in the following research areas:

- Urban meteorology
- Aviation meteorology
- Tropical cyclone research and development
- Multifunction Phased Array Radar (MPAR) Risk Reduction Program
- University transportation centers

- Commercial weather vendors
- Automobile manufactures
- State and local departments of transportation and road maintenance activities
- Social science

FUTURE DEVELOPMENTS FOR WIST

The OFCM and the Federal Highway Administration are sponsoring the Third National Surface Transportation Weather Symposium, July 25–27, 2007, at the Sheraton Premiere at Tysons Corner in Vienna, Virginia. The theme for the symposium is "improving commerce and reducing deaths and injuries through innovative weather-related research and development and applications for the surface transportation system."

The overarching objective for the symposium is to provide a forum for the surface transportation weather and transportation research and user communities to work together to enhance collaboration and partnerships, ultimately helping to improve surface transportation weather products and services for those who use, operate, and manage surface transportation infrastructure. The sub-objectives for the symposium include the following:

- Enhance understanding of the social and economic benefits derived from the increased use of improved surface transportation weather and climate information.
- Review, validate, and prioritize surface transportation weather research and development needs.
- Define and prioritize the products and services needed to support the surface transportation community.
- Provide recommendations for the weather and surface transportation communities regarding the way ahead to meet needs, using attendee input and feedback.
- Provide information on surface transportation weather and climate activities to enhance decision making processes.

CONCLUSIONS

Based on projections of U.S. population growth and the limited expansion of our highway system, the need for improved surface transportation weather data, forecasts, integration, dissemination, and education is real and growing. Much is already being done to meet the surface transportation user community's needs for weather information, as outlined in the 2002 WIST report, and there is some preliminary data showing that progress is being made in reducing deaths, injuries, and property damage. However, to continue to move forward effectively, a coordinated and prioritized approach is needed to improve our surface transportation weather products and services that incorporates the ideas and capabilities of all the stakeholders and service providers. Through the WG/WIST, work is underway to develop such an approach, and it will take the input and support of the surface transportation and meteorology communities to achieve success.

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Taking the Bang out of Transverse Cracks: Fly Ash Slurry Injection Method

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ABSTRACT

In this study, the nature and extent of transverse cracking in asphalt pavements on I-70 in Kansas was determined. A pavement investigation was conducted to determine the effectiveness of the fly ash slurry injection (FASI) method (a crack stabilization procedure) to eliminate or minimize the depression (bump) caused by the depressed transverse cracks. The intent of FASI is to fill the subsurface voids at severely distressed transverse cracks to delay depression and reflective cracking. The initial objective of the study was to find a low-cost "maintenance" approach to improve ride by filling the transverse cracks and their associated depression. A variety of products and application procedures were attempted, with variable results. Most attempts had recracked within a year, and depressions soon followed. Ten sections of test pavement were constructed using FASI to fill the voids under the existing pavement adjacent to the transverse cracks, followed by cold milling, cold in-place recycling of the next 4 in., a hot recycle action, and a hot mix asphalt overlay. Pavement roughness values before and after the rehabilitation action were compared. Roughness values (right wheel path IRI [in./mi], westbound and eastbound) were plotted for each year since 1988. The results indicated that a more extensive procedure, which involves using FASI followed by milling and overlaying with hot mix asphalt, has provided several years of excellent service on these ten test sections of I-70 in western Kansas.

Key words: asphalt pavement—fly ash slurry injection—transverse cracking

INTRODUCTION

Pavement profiles and detailed recordings of surface elevations are frequently used to characterize smoothness. Smoothness is an important indicator of pavement riding comfort and safety. From an auto driver's point of view, rough roads mean discomfort, decreased speed, potential vehicle damage, and increased operating cost. Highway users demand a good pavement condition (Bukowski 1990).

One of the primary causes of increasing roughness in asphalt pavements in Kansas has been the naturally occurring transverse cracks. Asphalt pavement shrinks as it ages, and this causes tension forces great enough to result in transverse cracks. Once a crack is created, the moisture in the crack causes the asphalt to further deteriorate, allowing a depression to develop at each crack (Snethen and Ahmed 1991).

The depressed area usually associated with the crack causes a rough ride and vehicle wear, and the impact load from heavy vehicles causes accelerated deterioration of the pavement structure. Needless to say, it is not a pleasant ride. The Kansas Department of Transportation (KDOT) has been continually faced with the problem of providing a satisfactory ride on the 10,000 mile primary highway system. Since about 75% of the system is asphalt pavement and the transverse cracks occur at intervals of about 50 ft., attempting to reduce or eliminate the effect of cracking and depression is a sizable endeavor.

Transverse cracking of asphalt pavements is a problem across the state of Kansas, with the severity of the problem varying from district to district based on such factors as pavement age, pavement cross section, traffic, asphalt properties, and maintenance procedures. The development of an effective method of filling the cracks and depressions would greatly improve the ride and safety for the motorist using the Kansas system of highways. It also would extend the life of the pavement by reducing the deterioration from water intrusion and from impact loading caused by vehicles bouncing as a result of the "bump," as well as reducing future maintenance costs.

HISTORY OF TRANSVERSE CRACKS ON I-70 WEST OF SALINA

A review of the history of transverse cracking on I-70 in the western half of Kansas reveals that very wide cracks (top down) developed in cold weather, and no suitable treatments were available from the 1960s through the 1980s. Maintenance forces continually tried to seal the cracks; however, the cracks continued to grow wider and the depression deeper. It was common to have cracks about 60 ft. apart 4 to 5 in. wide, depressed 2 to 3 in., and extended across all lanes.

Transverse cracking of asphalt pavements is a costly pavement distress occurring in Kansas, which experiences cold/freezing temperatures during the winter months. The cracks are caused by low-temperature-induced tensile stresses that exceed the tensile strength of the pavement material. The majority of these cracks occur in the transverse direction, relative to the pavement and with regular frequency along the roadway.

Once an open crack in asphalt has formed, the space tends to open further in cold periods and to close in warms periods (lateral movement primarily in response to thermal changes). Based on research, most crack movement occurs in a six- to eight-month Period, with a peak opening about the end of February or early March. Crack motion is generally consistent with temperature changes. Crack movements are a consequence of the changes in the thermal regime of the total pavement structure. These changes are in proportion to the average daily temperature and are not necessarily a direct reflection of spot surface or air temperatures taken usually during the warmer part of the day. The magnitude of lateral crack movement is a function not only of temperature changes, but also other factors. These include environmental effects, aging, and type of mix. However, the magnitude of the crack opening is not a function of the distance between adjacent cracks (Bukowski 1990).

A review of the literature shows that water is needed for bacteria to grow and deteriorate the asphalt. A soft, loosely bound asphaltic cement is often present between the rubble and sound materials. This seems to indicate a cause and effect relationship between bacteria and asphalt deterioration, since the rubble and loosely bound material (rubble-like deteriorated material) contained bacteria. The soil is the main habitat for bacteria. The bottom of the pavement tends to stay moist longer than would the exposed soil without the asphalt road above it. The soil-asphalt interface provides a great environment for bacterial existence. A crack in the asphalt allows air and water to contact the crack interface and provides more surface area of asphalt available to the bacteria. Therefore, full-depth hot mix recycling would be more effective than partial depth recycling in retarding bacterial decay at cracks and would likely destroy bacteria that are already in the pavement (Ramamurti and Jayaprakash 1987).

RESEARCH STUDY 91-3

A Type B research study, 91-3 (NCP #4C2C2152, KsDOT 91-3, RE-0706), “Taking the Bang out of Transverse Cracks: Fly-ash Slurry Injection Method,” began in FY 1991 as part of the annual Highway Planning and Research Work Program. The initial objective of the study was to evaluate a low-cost “maintenance” approach that had been used previously to fill transverse cracks and the associated depression. A variety of products and application procedures were used to fill transverse cracks and the associated depressed area in highways that did not need structural rehabilitation, except for the cracks that had been previously treated with variable results but had not been documented. Most attempts had recracked within a year, and further deterioration soon followed.

In the present study, the major objectives of the research included the following: (1) determine the nature and extent of transverse cracking asphalt pavements on I-70 in Kansas and (2) determine the effects of the fly ash slurry injection (FASI) maintenance action (a crack stabilization procedure) to eliminate or at least reduce the bump caused by the transverse crack and the depression on asphalt pavements. The intent of the FASI action was to fill the voids at severely distressed transverse cracks and thus delay reflective cracking.

FLY ASH SLURRY INJECTION METHOD

There are many miles of existing pavement that are in fine condition in terms of surface friction, ruts, shoving, map cracking, and other forms of distress, but which have continually enlarging transverse cracks. Filling the crack and depression would greatly improve ride and safety. It would reduce wear on vehicle suspension and preserve existing pavement by reducing the water intrusion into the crack and reducing the impact loads created by vehicles bouncing through the bump (Bukowski 1990).

Asphaltic concrete overlays and other maintenance actions temporarily correct the riding surface, but as reflection cracks appear the degradation continues and the depression is again formed. A need therefore exists to identify a procedure which would retard or eliminate the cracking and degradation of the new overlay.

By 1985, most of the pavement on I-70 west of Salina, Kansas, to Colorado had been overlaid two or more times during its 15- to 25-year life and ranged from about 10 to 24 in. thick. Each overlay provided a new riding surface and filled most of the depressions and cracks. However, on some overlays within weeks, the crack would reappear and the depression would start to grow. Therefore, a review of existing

KDOT policies and procedures for crack sealing and crack filling was conducted. A summary printout of the data in the Pavement Management Information System provided an indication of the severity of transverse cracking on the state primary highway system. Data was collected on the cost and performance of current repair efforts.

The FASI method injects a fly ash slurry into the area below each crack/depression through holes drilled into the pavement on both sides of the crack. The depressions are removed by cold milling the pavement after the slurry cures, and a hot mix bituminous overlay is placed. The technology used is similar to that used for undersealing or mud jacking portland cement concrete pavements. This method would have the added benefit of introducing a material that would retard bacterial growth, potentially stabilize the loose material at the bottom of the crack, and possibly create a “bulb” of material below the crack to help support it.

As noted in Table 1 and Figures 1 through 10, ten sections were constructed, first using FASI to fill the voids under the existing pavement adjacent to the transverse cracks, followed by cold milling, cold in-place recycling for the next 4 in., a hot recycle action, and a hot mix asphalt overlay. For extensively cracked and depressed pavements, milling the surface helps with the leveling of the pavement prior to overlaying.

Table 1. Projects selected for FASI study (I-70 in Western Kansas)

No.	Project no.	County	Completion date	Begin M.P.	End M.P.
1	K-5982-01	Ellsworth	2000	0	16.945
2	K-5983-01	Ellsworth	2000	16.945	23.248
3	K-5978-01	Gove	1998	0	19.254
4	K-5981-01	Gove	1999	19.254	37.508
5	K-5980-01	Lincoln	1998	0	7.247
6	K-2610-02	Saline	1997	8	14.9
7	K-5572-01	Thomas	1996	0	4.393
8	K-5908-01	Thomas	1998	4.393	10.342
9	M-1775-01	Thomas	1996	19.070	28.031
10	K-5979-01	Thomas	1998	28.062	39.554

As shown in Figures 1 through 10 and Table 2, pavement roughness values before and after the rehabilitation action were compared. Roughness values (right wheel path IRI [in./mi], westbound and eastbound) were plotted for each year since 1988. Figures 1 through 10 demonstrate that a more extensive procedure, which involves using FASI followed by milling and overlaying with hot mix asphalt, has provided several years of excellent service on ten sections of test pavement of I-70 in western Kansas. As a result, FASI was identified as an effective crack repair procedure that can extend pavement service life and reduce future maintenance costs.

Examination of transverse cracks at the ten field sites confirm that, in most cases, transverse cracks originate at the pavement surface and extend down into and/or through the pavement. The primary cause of transverse cracking is thermal-induced stress that causes contraction of the asphalt concrete surface layer. An average low temperature has been found to have a significant effect on transverse cracking, and average spacing between the cracks decreases with a decreasing average low temperature (Traxler 1966).

Transverse crack depressions are caused by an ingression of water through the cracks in the pavement to the subgrade soils. This causes softening of the subgrade soil and subsequent depression adjacent to the crack. Additionally, pumping of fines from the base and subgrade through the cracks contributes to the loss of support. The plastic limit of the fine-grained soils has been correlated with crack indexes such as number of cracks per 1,000 ft. This indicates that pavements overlying fine-grained soils have a greater risk of transverse cracking (Hicks, Kimberly, and Moulthrop 1997).

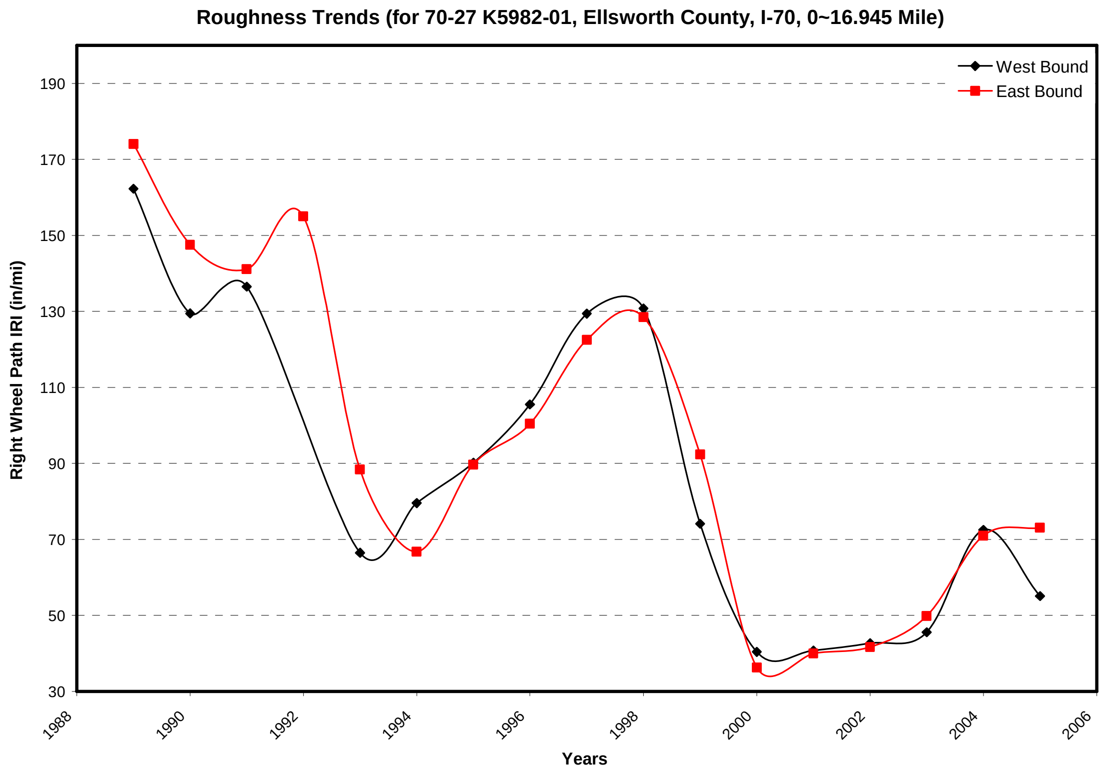

Figure 1. K-5982-01, completion date 2000 with a 70% improvement

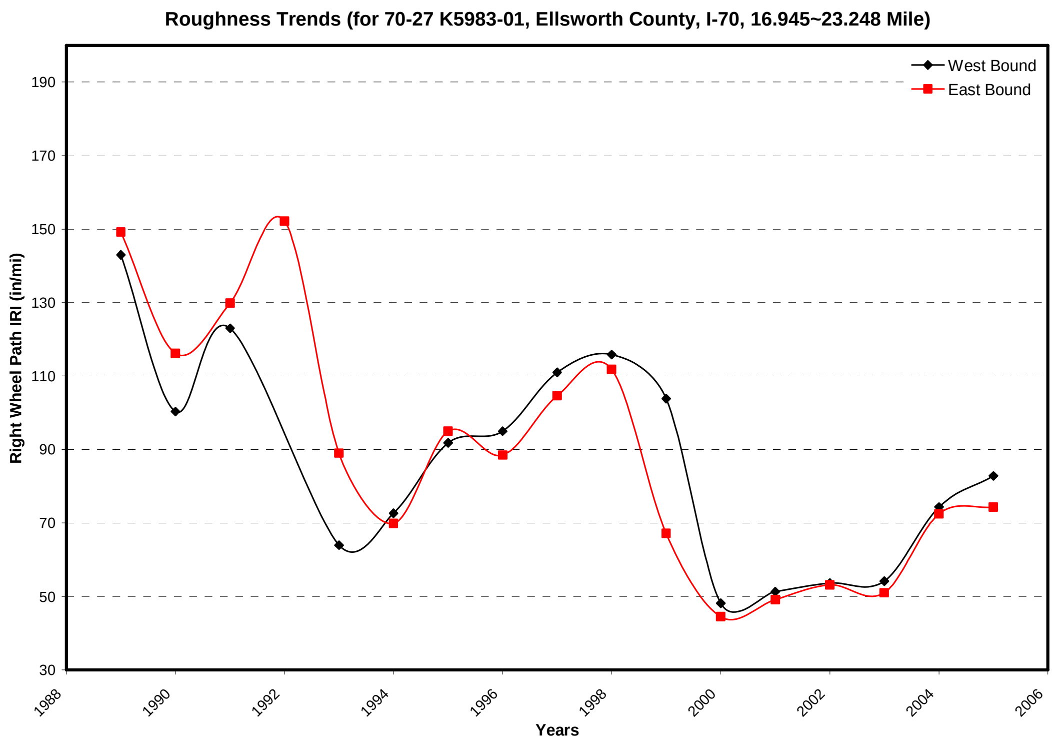

Figure 2. K-5983-01, completion date 2000 with a 61% improvement

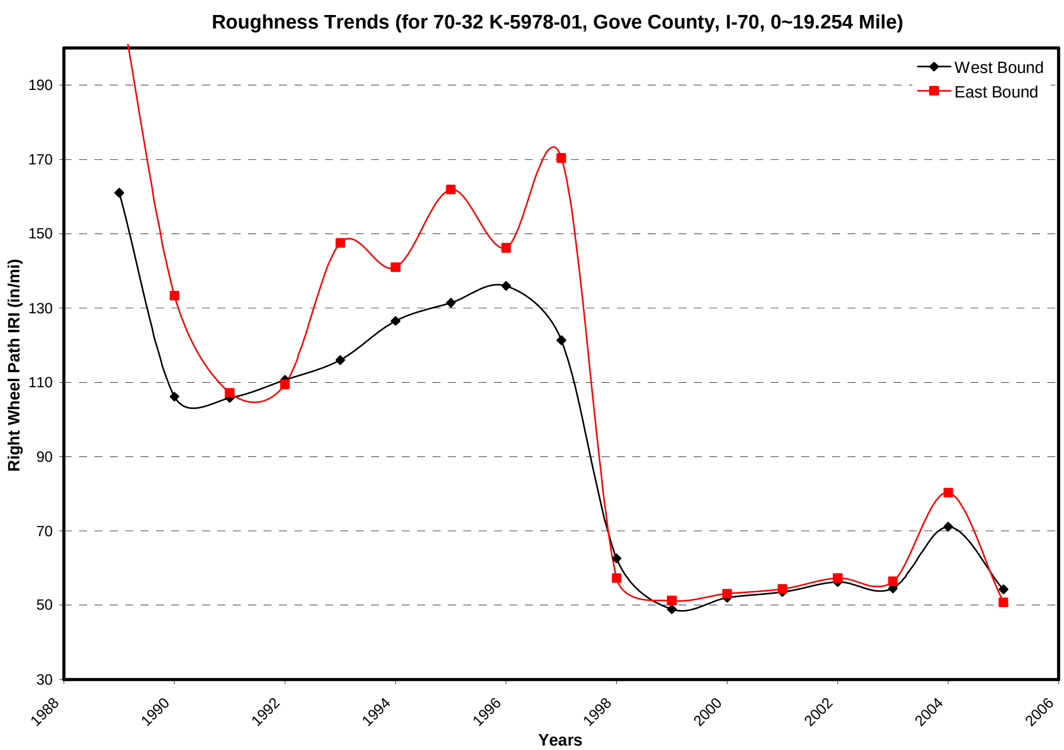

Figure 3. K-5978-01, completion date 1998 with a 68% improvement

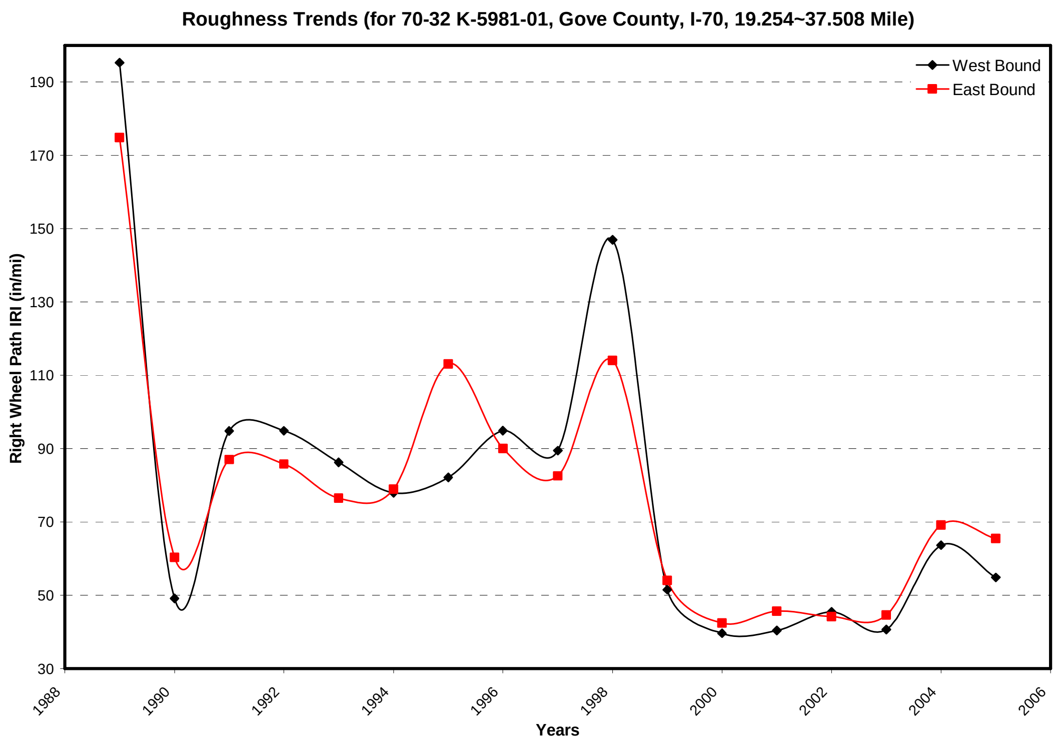

Figure 4. K-5981-01, completion date 1999 with a 70% improvement

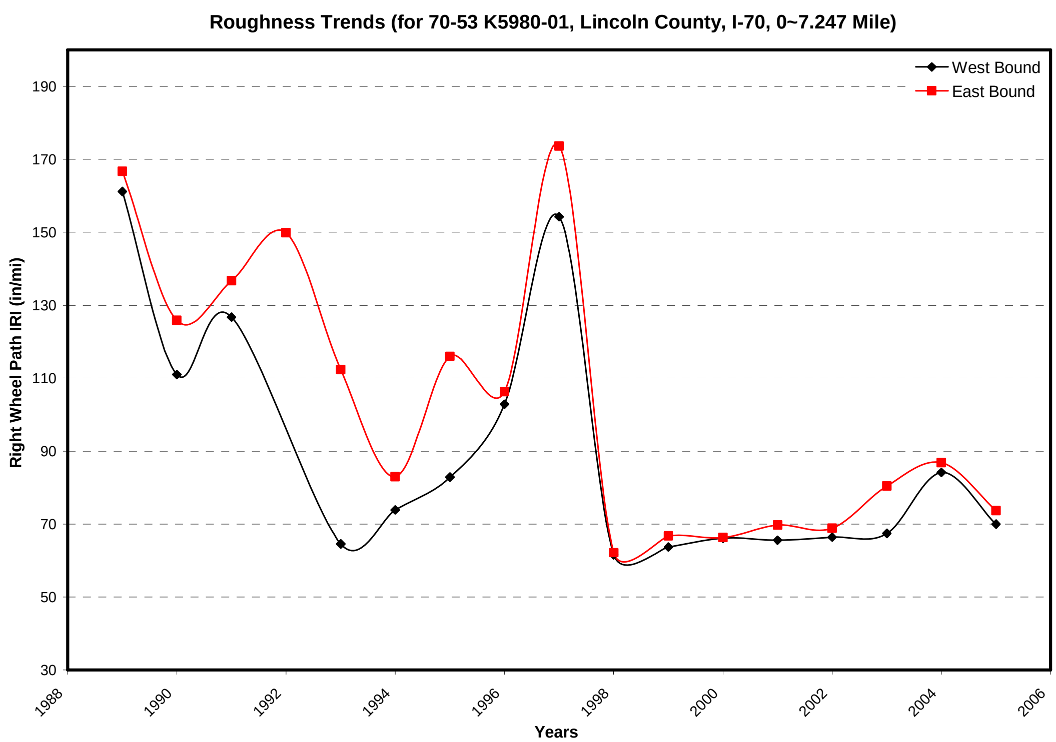

Figure 5. K-5980-01, completion date 1998 with a 63% improvement

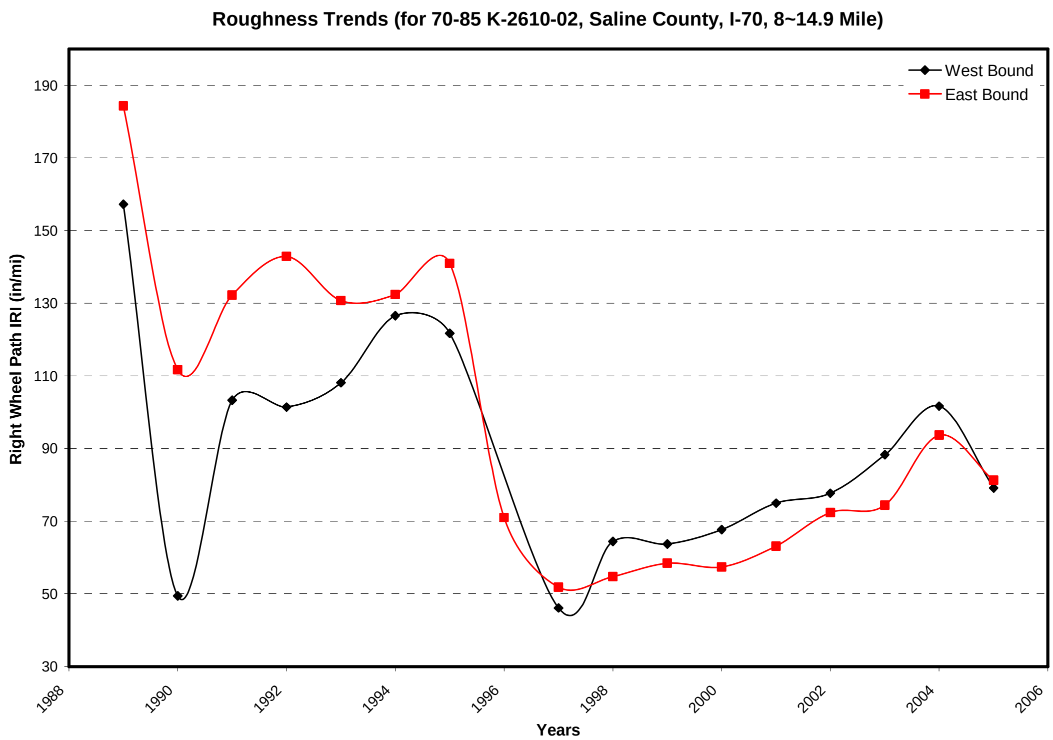

Figure 6. K-2610-02, completion date 1997 with a 64% improvement

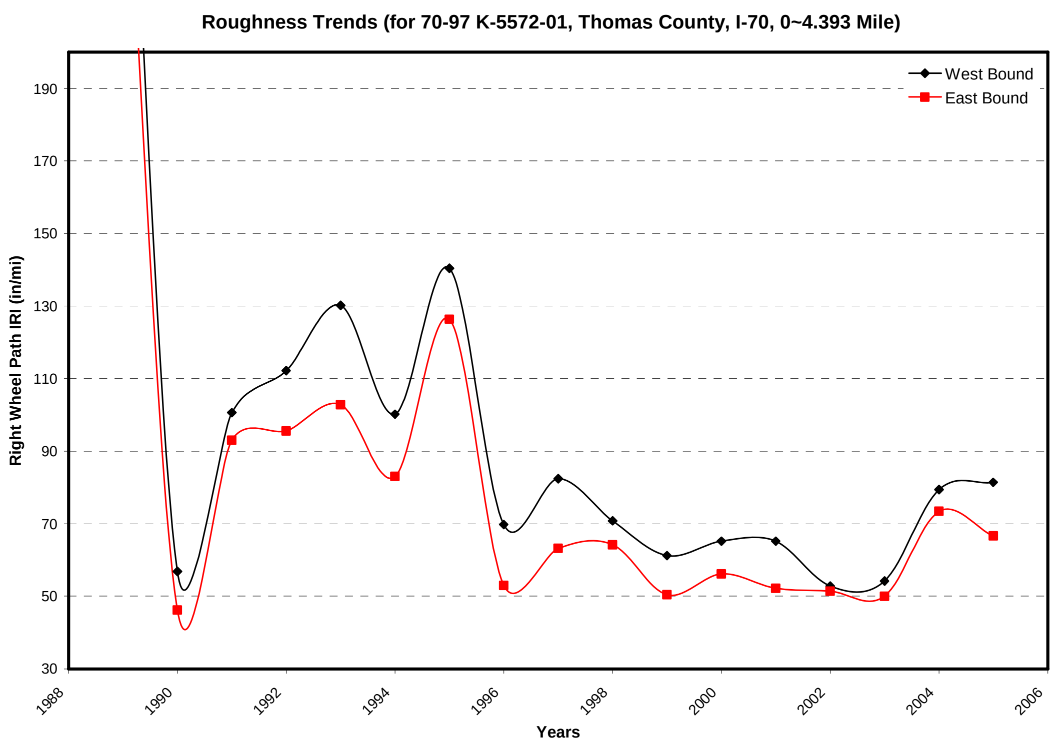

Figure 7. K-5572-01, completion date 1996 with a 56% improvement

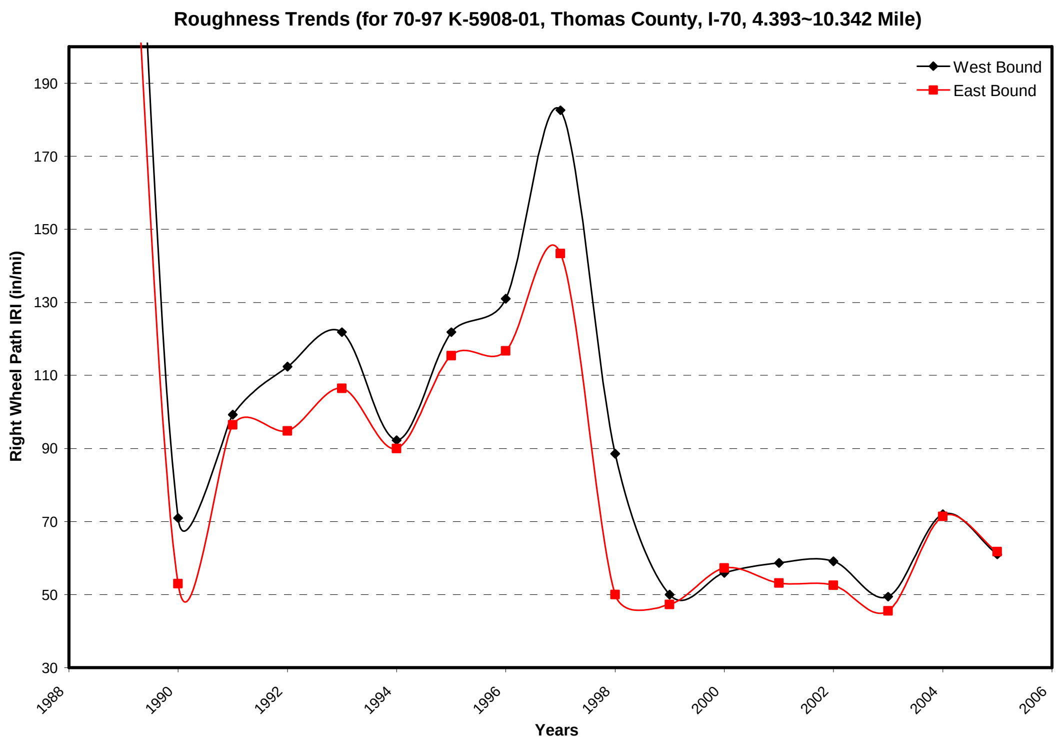

Figure 8. K-5908-01, completion date 1998 with a 71% improvement

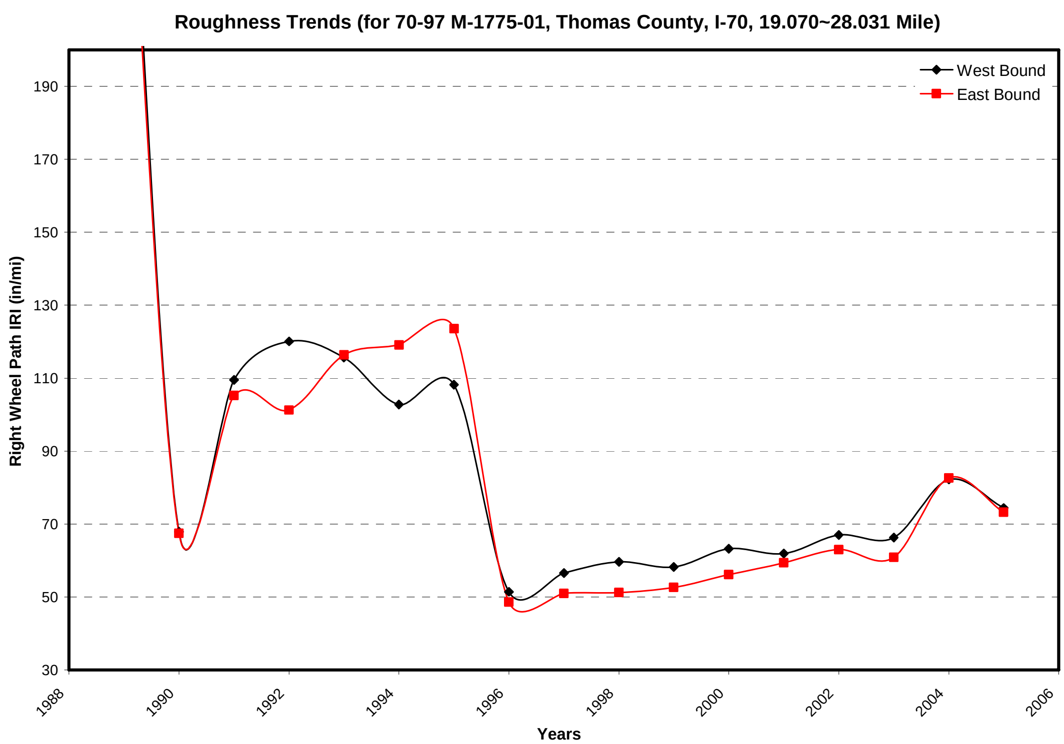

Figure 9. M-1775, completion date 1996 with a 60% improvement

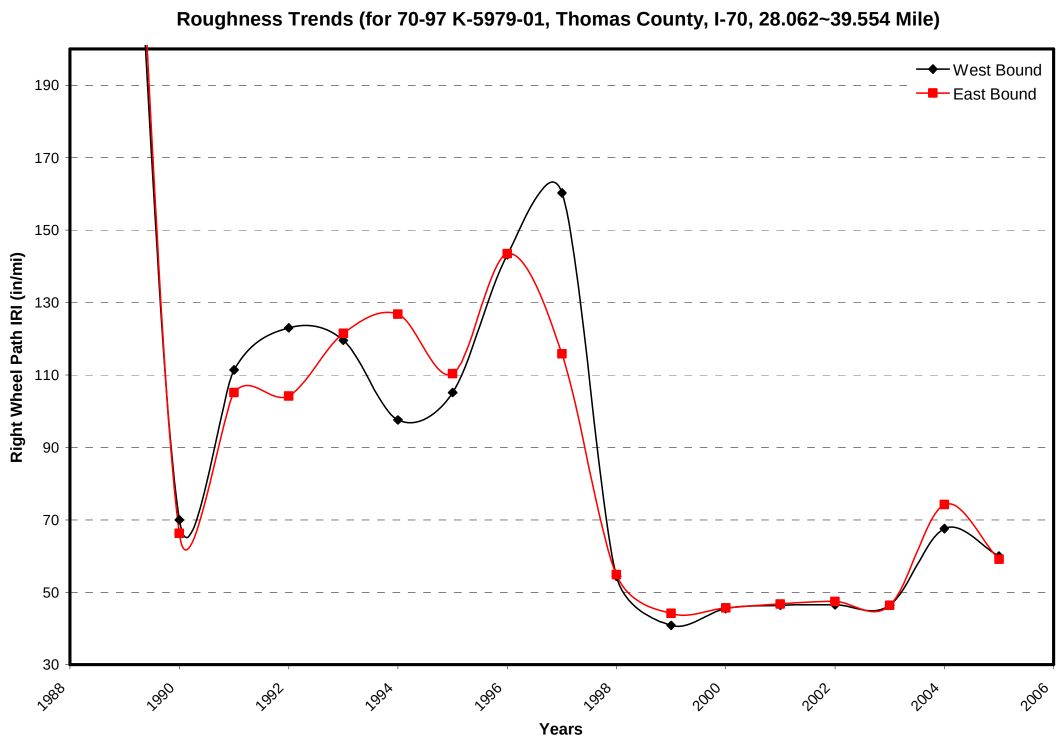

Figure 10. K-5979-01, completion date 1998 with a 75% improvement

Table 2. Pavement roughness values before and after the rehabilitation action

No.	Project no.	Completion date	Before rehabilitation	After rehabilitation	Improvement (%)
1	K-5982-01	2000	132	40	70
2	K-5983-01	2000	115	45	61
3	K-5978-01	1998	155	50	68
4	K-5981-01	1999	135	40	70
5	K-5980-01	1998	160	60	63
6	K-2610-02	1997	135	48	64
7	K-5572-01	1996	135	60	56
8	K-5908-01	1998	165	48	71
9	M-1775-01	1996	120	48	60
10	K-5979-01	1998	160	40	75

The FASI technique seems to have been the most successful. The injected fly ash slurry appears to have stabilized the deteriorated material in the vicinity of the crack and has resulted in the elimination of the depression for up to 12 years. This technique has been used in many locations across the state, resulting in improved highway smoothness. The ride from Goodland to Salina has been greatly improved, and the bang from the transverse cracking is gone.

CONCLUSIONS

The FASI technique seems to have been the most successful. The injected fly ash slurry has resulted in the elimination of the transverse crack depressions for up to 12 years. This technique has been used in many locations across the state, resulting in improved highway smoothness. Therefore, the ride has been greatly improved. The major conclusions that can be drawn from the study may be summarized as follows:

1. The results indicated that using FASI to fill the voids under the existing pavement, followed by cold milling and overlaying with hot mix asphalt, has provided the benefit of retarding reflective cracking and providing several years of excellent service on ten sections of I-70 in western Kansas. This method can extend pavement service life and reduce future maintenance costs.
2. FASI was identified as an effective crack filling procedure. Filling the cracks and depression would greatly improve ride and safety. It would reduce the wear on vehicle suspension and preserve existing pavement by reducing the water intrusion in the crack and reducing the impact loads created by vehicles bouncing through the bump.

RECOMMENDATIONS

The following recommendations are considered pertinent to the results of this research investigation:

1. Further comprehensive testing of cracked and uncracked pavement sections should be undertaken to develop a better understanding of actual thermal effects on stress, strain, and stiffness values.
2. Asphalt chemistry and aggregate properties should also be considered in the evaluation.
3. A comprehensive evaluation of preventive and remedial maintenance procedures should be conducted to establish the conditions under which the various procedures perform best and should be considered useful in extending the life of a pavement if applied at the right time.
4. Cracks in asphalt pavements should be repaired as soon as possible after detection. Well-defined cracks with no secondary cracking should be sealed to prevent the incursion of water and noncompressibles.
5. A field and laboratory investigation of pavement materials and highway features should be conducted in order to determine and evaluate the various factors that influence transverse cracking.
6. Cracks and depressions should be studied and inventoried each year. This would provide a means to historically watch and study the crack problems in order to help bring the problems to the attention of management.
7. The effects of various crack sealing procedures on different thicknesses of bituminous overlays should be determined.
8. The FASI injection method must be revamped. One idea is to drill a vertical hole through the crack and then pump the grout. Another idea is to drill an angled hole intercepting the crack in the bottom 1/3 of the pavement. Both of these methods would place more grout in the crack; however, their success would need to be monitored. It is recommended that KDOT collaborate with industry to determine the best method of fly ash slurry injection.

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Electronic Speed Feedback Signs as a Rural Community Traffic Calming Measure

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ABSTRACT

Electronic feedback signs are a universal form of traffic calming, giving a visual cue to the driver that he or she is speeding in an area that could potentially be dangerous to the driver or to bystanders. Speed feedback signs have been used extensively in large urban areas. This paper presents research that evaluated speed feedback signs in two small communities in Iowa. The signs were used as part of research that evaluated different traffic calming treatments along the major road through a small community. One feedback sign was installed in Slater, Iowa, which has a population of just over 1,300 citizens. The speed sign in Slater is capable of digitally recording and reporting speeds to the driver and can be programmed to flash a variety of messages at various speed limits. The second location was Union, Iowa, which has a population of just over 500 citizens. Two speed feedback signs were installed in different locations. The speed feedback signs in Union display the driver speed up to 5 mph over the speed limit, at which time the message "SLOW DOWN" is displayed. The effectiveness of the feedback signs was evaluated using a before–and-after study. Data were collected before installation of the signs and one month after installation. Data were also collected at three- and six-month intervals so that the effect over time could be observed.

Key words: driver reaction and adaptation—rural Iowa main streets—speed feedback signs—traffic calming

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Effectiveness of Iowa's Automated Red Light Running Enforcement Programs

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ABSTRACT

One of the most controversial topics facing traffic engineers, city councils, and public awareness groups is the implementation of automated red light running enforcement camera systems at urban signalized intersections. Red light running is a significant safety problem, as drivers become more aggressive on city roads and become impatient waiting for a traffic signal to change. Red light running camera systems are automated enforcement systems that detect vehicles running a red light and then issue the vehicles a citation. They are becoming widely used in the United States to reduce the number and severity of red light running crashes. The effectiveness of automated red light running enforcement cameras is constantly debated among government officials and citizens who see cameras as either intrusive or constitutionally illegal to an extent. In some cases, it has been argued that automated red light running enforcement increase the percentage of rear-end collisions.

In 2004, the state of Iowa reported over 2,900 crashes (approx. 4.9% of all reported crashes) involving "failure to yield right of way making right turn on red signal" and "ran traffic signal," both which constitute a driver being involved in a red light running collision. This paper presents the results of a research project that evaluated the effectiveness of Iowa's currently deployed automated red light running camera systems in Council Bluffs, Davenport, and Clive. Violation data were collected from each community, and system effectiveness was measured through a before-and-after study and a comparison of enforced intersections with similar intersections where the automated enforcement system is not expected to have any spillover effect. The before–and-after data study investigated the reduction in total accidents, red light running-related crashes (e.g., broadside, right-angle, and rear-end), and crash severity. The findings were compared to other communities around the nation to see if Iowa's automated red light running enforcement system was effective on a national level.

Key words: automated enforcement—public acceptance—red light running camera effectiveness—spillover effects—urban intersection crashes

Passing Activity on a Busy Recreational Road in Spain: Level and Climbing Section

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ABSTRACT

Passing activity was measured with low to medium flows in a road leading to a mountainous recreational area near Madrid, Spain. Passing was measured both in a level-to-rolling section and a climbing section. Passing activity was compared with theoretical passing demand and other values from the literature.

Key words: mountainous area—passing activity—Spain

Performance Prediction and Maintenance of Flexible Pavement

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ABSTRACT

This paper discusses the performance and maintenance of flexible pavement. The main objectives of this study are to predict the performance of flexible pavement using two distress models in the KENLAYER computer program and eight deterioration models in Highway Development and Management **(**HDM-4) and provide appropriate maintenance at the appropriate time based on performance using HDM-4. KENLAYER computer program has been used for determining the damage ratio using distress models. HDM-4 computer software has been used for predicting the performance using pavement deterioration models and also for pavement maintenance. Prediction of performance and maintenance has been carried out for the test section located in Mumbai Metropolitan Region, India. This region has a humid, warm, and wet climate prevalent in the west coast of India. The test section has seven layers, and it is a six-lane divided highway. Design life in years has been determined using distress models in the KENLAYER computer program. Asphalt Institute (AI) and Shell design methods have been considered using equivalent standard axle load (ESAL) and spectrum of axle methods of incorporating traffic for the design period. Comparison of design life has been made, and design life using AI design method due to vertical compressive strain on the top of the subgrade has been found to be governing while considering traffic using the spectrum of axle method. Eight deterioration models in HDM-4 have also been used to determine pavement performance. To determine the governing deterioration model, the output of the eight deterioration models has been compared based on the allowable limits for Indian conditions, and pavement performance using the cracking model has been found to be governing. Comparison has been made between KENLAYER and HDM-4 output. The analysis of the test section indicates that the life of pavement predicted by HDM-4 is less than that predicted by KENLAYER. Only cracking and roughness have been found out to be critical, and as a result, condition-responsive maintenance has been carried out using HDM-4. HDM-4 is a user friendly software and useful to predict the performance of the pavement and then provide appropriate maintenance at the appropriate time.

Key words: design life—flexible pavement—prediction—performance

INTRODUCTION

Highway engineers design flexible pavements after carrying out all required investigations. The mechanistic method of flexible pavement design is an emerging technology for design, which contains a number of distress models: mainly fatigue cracking and rutting. These models are used to determine the design life of the pavements. Pavements are constructed as per the standards and specifications after design. However, pavements usually do not serve for the design period efficiently, safely, comfortably, and economically due to early deterioration. Pavement deterioration is broadly a function of the original design, material types, construction quality, traffic volume, axle load characteristics, road geometry, environmental conditions, age of pavement, and the maintenance policy pursued.

The focus of roadway activity in the early to mid 20th century was on the construction of new pavements. In the latter part of the 20th century continuing into the 21st century, this focus has shifted to the maintenance and rehabilitation (M & R) of pavement infrastructure. Maintenance includes actions that retard or correct the deterioration of infrastructure facilities (Guignier and Madanat 1999). Pavements must be selected for maintenance when they are still effective. In most cases, the proper time to apply maintenance is before the need is apparent to the casual observer. This is because once pavements start deteriorating, they deteriorate rapidly beyond the point where maintenance is effective. With the increasing use and awareness of pavement management systems and the growing emphasis on asset management of pavement infrastructure, it is important to strengthen the maintenance components of these (Hein and Croteau 2004).

OBJECTIVES OF THE STUDY

The main objectives of this study are as follows:

- Review distress and deterioration models in KENLAYER and HDM-4, respectively.
- Compare flexible pavement performance using distress and deterioration models in KENLAYER and HDM-4, respectively, and then to recommend a deterioration model that gives comparable result with distress models.
- Review maintenance strategies in HDM-4, maintenance trends in India, and finally, to see the effect of maintenance on pavement deterioration using HDM-4.

LITERATURE REVIEW

Distress Models in KENLAYER

Distress models in KENLAYER are cracking and rutting. Strains due to cracking and rutting have been considered most critical for the design of asphalt pavements. One is the horizontal tensile strain (ε_t) at the bottom of the asphalt layer, which causes fatigue cracking, and the other is vertical compressive strain (ε_t) on the surface of the subgrade, which causes permanent deformation or rutting (AI 1981). Distress models can be used to predict the life of new pavement assuming pavement configuration. If the reliability for a certain distress is less than the minimum level required, the assumed pavement configuration should be changed (Huang 2004).

Fatigue Cracking Models

Miner's (1945) cumulative damage concept has been widely used to predict fatigue cracking. It is generally agreed that the allowable number of load repetitions is related to the tensile strain at the bottom of the asphalt layer. The amount of damage is expressed as a damage ratio, which is the ratio between predicted and allowable number of load repetitions. Damage occurs when the sum of damage ratio reaches one.

The major difference in the various design methods is the transfer functions that relate the hot mix asphalt (HMA) tensile strains to the allowable number of load repetitions. The allowable number of load repetitions (N_f) can be computed using Equation 1:

$$N_f = f_1(\varepsilon_t)^{-f_2}(E)^{-f3} \qquad (1)$$

Where ε_t is tensile strain at the bottom of HMA, E is modulus of elasticity of HMA and f_1, f_2, and f_3 are constants obtained by calibration.

Rutting Models

Rutting models are used to limit the vertical compressive strain on the top of the subgrade and are widely used. The allowable number of load repetitions (N_d) to limit rutting is related to the vertical compressive strain (ε_c) on top of the subgrade by Equation 2:

$$N_d = f_4(\varepsilon_c)^{-f_5} \qquad (2)$$

where f_4 and f_5 are calibrated values using predicted performance and field observation.

Different institutions have provided different distress models. The coefficients for rutting and cracking used by some of the institutions are given in Table 1.

Table 1. Coefficient in rutting and cracking distress models

Sr. No	Distress Models	*f1*	*f2*	*f3*	*f4*	*f5*	Sources
1	AI Model	0.0796	3.291	0.854	$1.365*10^{-9}$	4.477	AI (1981)
2	Shell Model	0.0685	5.671	2.363	$1.05*10^{-7}$	4.0	Shell (1978)
3	Belgian RRC	$4.92*10^{-14}$	4.76	0	$3.05*10^{-9}$	4.35	Verstraeten et al. (1984)
4	Indian Model	$2.2*10^{-4}$	3.89	0.854	.4166E-05	4.534	IRC-37:2001 (2001)

Deterioration Models in HDM-4

Pavement deterioration models relate the functions, which are the measure of distress due to the magnitude of loads, number of load repetitions, pavement composition and thickness, and subgrade moisture (Sood and Sharma 1996). They should be able to predict the change in pavement condition over a given period of time under a set of conditions. They are exponential in nature, and the rate varies depending upon its condition with the passage of time. Road deterioration is computed as the incremental change in pavement condition over a period of time due to the effects of pavement characteristics, traffic, environment, and maintenance inputs. A model represented in incremental form can take care of pavements in any initial stage of condition and at any age and is the most preferred form for economic evaluation of road pavements and maintenance strategies.

There are eight deterioration models in HDM-4 under three categories. Most of them are characterized by initiation and progression. The major deterioration models in HDM-4 are discussed below.

Cracking Model

Cracking is one of the most important measures of deterioration in bituminous pavements. Fatigue and ageing have been identified as the principal factors which contribute to cracking of a bituminous pavement layer. The propagation of cracking is accelerated through the embrittlement resulting from ageing and the ingress of water, which can significantly weaken the underlying pavement layers. There are two types of cracking considered in HDM-4: structural and transverse thermal cracking. The first one is effectively load- and age/environment-associated cracking. It is modeled based on the relationships derived by Paterson (1987). Initiation of all structural cracking is said to occur when 0.5% of the carriageway surface area is cracked. The second one is generally caused by large diurnal temperature changes or in freeze/thaw conditions, and, therefore, usually occurs only in certain climates. For each type of cracking, separate relationships are given for predicting the time to initiation and then the rate of progression.

Ravelling Model

Ravelling is the progressive loss of surface material through weathering and/or traffic abrasion. The occurrence of ravelling varies considerably among different regions and countries according to construction methods, specifications, available materials, and local practice. It is a common deterioration in poorly constructed, thin bituminous layers such as surface treatment, but it is rarely seen in high quality hot mix asphalt. The construction defects indicator for bituminous surfacing (CDS) is used as a variable in the ravelling models. The initiation model is basically as proposed by Paterson (1987), with CDS replacing the original construction quality variable. It is said to occur on a given road section when 0.5% of the carriageway surface area is classified as ravelled. The progression model is also based on that proposed by Paterson (1987) but with a traffic variable introduced as proposed by Riley (1999).

Potholing Model

Potholing usually develops in a surface that is either cracked, ravelled, or both. The presence of water accelerates pothole formation both through a general weakening of the pavement structure and lowering the resistance of the surface and base materials to disintegration. Potholing models use the construction defects indicator for the base as a variable. Initiation of potholes arises once the total area of wide structural cracking exceeds 20%. Ravelling-initiated potholes arise when the ravelled area exceeds 30%. Progression of potholes arises from potholes due to cracking, raveling, and the enlargement of existing potholes. It is affected by the time lapse between the occurrence and patching of potholes.

Rut Depth Model

Rut depth is defined as the permanent traffic-associated deformation within pavement layers which, if channelised into wheel paths, accumulates over time and becomes manifested as a rut (Paterson 1987). Rut depth modeling is performed after the values of all the surface deterioration of cracking, raveling, potholing, and edge-break at the end of the year have been calculated. The rut depth model is based on four components of rutting:

- Initial densification
- Structural deformation
- Plastic deformation
- Wear from studded tires

Roughness Model

Roughness consists of several components of roughness such as cracking, structural, rutting, potholing, and environment. The total incremental roughness is the sum of these components. The surface deterioration values used in predicting roughness are those that have been adjusted so that the total damaged surface area plus the undamaged area equals 100%.

The remaining three models are edge-break, texture depth, and skid resistance. They are only characterized by progression models. These models are not common compared to the other deterioration models.

Pavement Maintenance in HDM-4

In HDM-4, maintenance standards are used to represent the targets or levels of condition and response that are aimed to be achieved. Maintenance standards define the maintenance work required to maintain the road network at the target level. Each maintenance standard consists of a set of one or more work items. Each work item is defined in terms of the road surface class to which it applies, an intervention level, an operation type, and the resultant effect on the pavement (Sood and Sharma 1996*)*. Routine and periodic maintenance are the two kinds of maintenance treated in HDM-4. All maintenance can be carried out based on scheduled and condition-responsive except inlays, in which it is always defined in terms of condition-responsive work.

Pavement Maintenance Scenario in India

Due to the poor condition of roads, it is estimated that an annual loss of approximately over Rs. 6000 crores ($1.33 billion) is resulted in vehicle operating costs (VOC) alone. Timely maintenance is missing due to many reasons, which otherwise could have minimized the losses to the exchequer. A rough estimate suggests that more than 50% of the primary road network is in bad shape and needing immediate attention. It should be borne in mind that for achieving the desired economic growth, the foremost requirement is to ensure a good and effective road network (MoRTH 2004).

Types of Maintenance

The maintenance activities have been divided into ordinary repairs (OR), periodical renewal (PR), special repairs, and emergency repairs for the purpose of organization of maintenance budgeting. It is pertinent to note that organizational structure of maintenance activities related to OR has worked well in the past, and it may be recommended that the same may be continued after updating to include all the new activities required to keep pace with time and development until a scientific maintenance management system (MMS) is placed in position. In general, there are two types of maintenance; namely, routine maintenance (to cover OR) and major maintenance (to cover PR and rehabilitation)

Optimization and Prioritization of Maintenance Strategies

When planning investments in pavement sectors, it is necessary to evaluate all the costs associated with the proposed project. These include construction costs, M & R costs, road user costs, and all other external or exogenous costs or benefits that can directly attribute to the pavement project. These three costs constitute what is commonly referred to as the total transport cost or the life cycle cost. A flow chart for prioritization of maintenance is given in Figure 1.

Intervention Criteria

There are two types of maintenance inputs in practice; time bound (scheduled) maintenance and pavement condition-responsive maintenance. For Indian conditions, it is suggested that condition-responsive maintenance intervention criteria may be adopted. To formulate condition-responsive maintenance criteria, some basic minimum desired serviceability level needs to be fixed. The suggested criteria are based on the widely accepted performance indicators such as roughness, cracks, rutting, skid, potholes, etc. Based on these performance indicators, the suggested intervention criteria for primary, secondary, and urban roads are given by the Ministry of Road Transport and Highways (2004). The intervention criteria for primary roads are shown in Table 2.

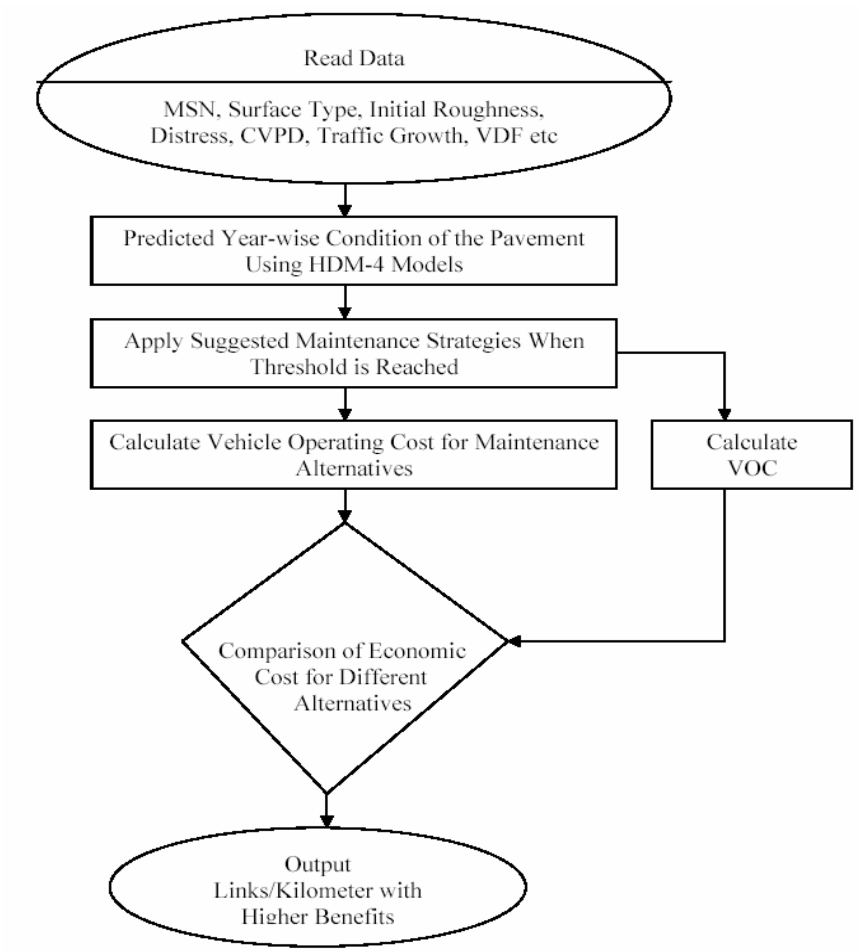


Figure 1. Flow chart for prioritization of maintenance (MoRTH 2004)

Table 2. Intervention criteria for primary roads

Sr. No.	Serviceability Indicator	Level 1 (Good)	Level 2 (Average)	Level 3 (Acceptable)
1	Roughness by Bump Integrator (max. permissible in mm/km	2000	3000	4000
2	Potholes per km (max. numbers)	Nil	2-3	4-8
3	Cracking and patching area (max. permissible in percent)	5	10	10-15
4	Rutting (mm)	5	5-10	10-20
5	Skid resistance (skid number min. desirable)	50	40	35

METHODOLOGY

In the present study, a test section in the Mumbai Metropolitan Region (MMR) has been used for carrying out the analysis of pavement performance using the KENLAYER computer program and the Highway Development and Management Model (HDM-4). Fatigue cracking and rutting are two distress models considered in KENLAYER at the bottom of the asphalt layer and on top of the subgrade, respectively. The program is used to predict the performance of the new pavement. HDM-4 software is used for determining the annual condition of the pavement once constructed and open for traffic (Wightman, Stannard, and Dakin 2002). There are eight deterioration models, mainly structured empirical models of flexible pavement, incorporated in HDM-4, which are used to indicate the annual condition of flexible pavement. Pavement performance has been predicted using two distress models in the latest version of the KENLAYER computer program and eight deterioration models in HDM-4 software. Comparison of the outputs has been carried out to determine the governing distress and deterioration models. Finally, HDM-4 has been used for maintenance purposes.

Application of KENLAYER Computer Program

The KENLAYER computer program applies only to flexible pavements with no joints. The backbone of KENLAYER is the solution for an elastic multilayer system under a circular loaded area. The solutions are superimposed for multiple wheels, applied iteratively for non-linear layers, and collocated at various times for viscoelastic layers. As a result, KENLAYER can be applied to layered systems under single, dual, dual-tandem, or dual-tridem wheels, with each layer behaving differently: linear elastic, nonlinear elastic, or viscoelastic. Damage analysis can be made by dividing each year into a maximum of 12 periods, each with a different set of material properties. Each period can have a maximum of 12 load groups, either single or multiple. The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life (Huang 2004).

Input Parameters in KENLAYER Computer Program

There are so many input parameters in KENLAYER. The parameters can be inputted both in SI and U.S. customary units. Some of the input parameters for linear elastic analysis are traffic load, material properties, thickness of each layer, number of periods, number of load groups, etc.

Output Parameters of KENLAYER

For single and multiple load groups, a maximum of nine and ten responses can be obtained, respectively. Only the vertical compressive strain on the surface of the subgrade and the radial (tangential) tensile strain at the bottom of asphalt layer are used for damage analysis.

Highway Development and Management (HDM-4)

The International Study of Highway Development and Management (ISOHDM) has been carried out to extend the scope of the HDM-III model and to provide a harmonized systems approach to road management, with adaptable and user friendly software tools. This has produced HDM-4. The scope of HDM-4 has been broadened considerably beyond traditional project appraisals to provide a powerful system for the analysis of road management and investment alternatives.

Applications of HDM-4

HDM-4 is a powerful system for the analysis of road management alternatives. With different application tools, HDM-4 can be applied in project analysis, program analysis, strategy analysis, research, and policy studies. Project analysis tools have been used for predicting pavement performance in this study, which include eight deterioration models.

Input Parameters in HDM-4

The HDM-4 application has been designed to work with a wide range of data type and quality. HDM-4 supplies default data that are user definable. Users can choose the prevailing values in the environment under study. The flexibility in data requirement not only reduces the data entry work but also permits all potential users with a variety of data to integrate HDM-4 into road management systems. Some of the main input data required are road network data, vehicle fleet data, traffic data, and road works standards.

Output Parameters of HDM-4

HDM-4 supports flexible options for data and analysis results. Users can make printed or electronic reports. They can also export data and results to standard database for other users. The file formats are not limited to text: Microsoft Word document, MS Excel, and lotus 1-2-3 spread sheet are also available. In addition, users have direct access to the result database files (DBF). HDM-4 can produce the following three types of output, which can help road managers to make informed decisions:

- Strategic road maintenance and development plans, produced from long-term predictions of road network performance
- Economic efficiency indicators, produced from analysis of individual road projects
- Multiyear work programs, produced from prioritization of several road projects

CASE STUDY

The case study is a test section in MMR, which has been built by the City and Industrial Development Corporation (CIDCO) of Maharashtra Ltd. in 1990. The test section is located at the southern tip of MMR and is planned over a total area of 2592Ha. comprising of 64 sectors. The test section has been basically planned to cater the port-based services. The area has a humid, warm, and wet climate prevalent in the west coast of India. The area is covered by two major deposits of marine and fluvial deposits and residual deposits connecting steep hill slopes. The underlying soil consists of a thick layer of soft marine clay deposits, which is very soft and highly compressible. CIDCO has implemented the ground improvement scheme before going ahead with developmental activities in the area (CIDCO 2000).

Terrain classification of the area is made by the general scope of the area across the road alignment. The road network of the test section is in the filled up areas, and, hence, it is plain terrain. However, the percent of cross slope for plain and rolling terrain is given in Table 3.

Table 3. Terrain classification of test section

Sr.No.	Terrain Classification	Percent Cross Slope of the Area
1	Plain	0-10
2	Rolling	10-15

The test section for this study has a six-lane carriageway. The cross slope of the carriageway is 2.5%. The test section has seven layers, and all layers are assumed to be linear elastic for the analysis. The cross section of the test section used for analysis is shown in Figure 2.

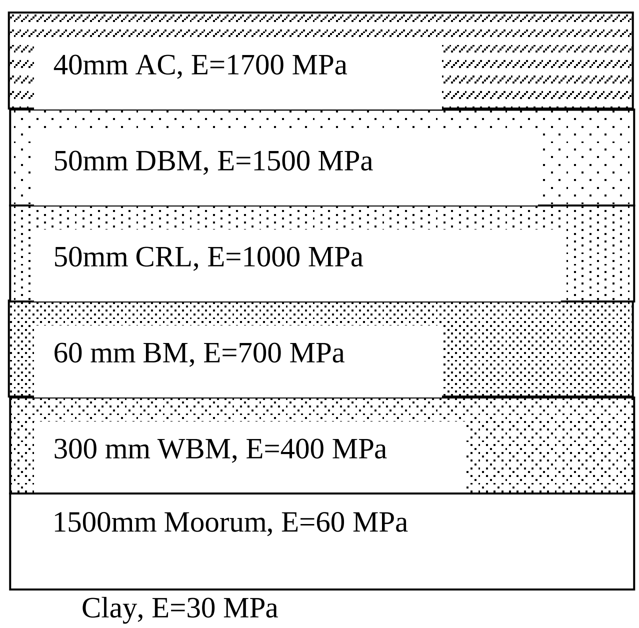


Figure 2. Cross section of test section for the analysis

RESULTS AND DISCUSSION

The output using the two software programs KENLAYER and HDM-4 are presented separately, and a comparison has been made in order to determine the governing distress and deterioration models. Then maintenance using HDM-4 and its effect on deterioration of pavement is presented. Traffic data collected by CIDCO has been used for the analysis of the test section using both software programs.

Pavement Performance Using KENLAYER

Traffic loads have been considered using Equivalent Standard Axle Load (ESAL) and spectrum of axle approaches. AI and Shell design methods have been used for predicting pavement performance.

In the ESAL approach, all axle loads have been converted into equivalent standard axle load for the design period. Since it is strengthening the already existing road, the design period has been taken as 10 years. ESAL at the end of the 10 year period, using an annual growth rate of 5% as per the recommendation by the consultant, is 51.79 million standard axles (msa), and the same has been used for predicting pavement performance. Horizontal tensile strain at the bottom of the asphalt concrete layer and vertical compressive strain on the top of subgrade are treated using AI and Shell design methods. Vertical compressive strain is governing in both cases. The sum of damage ratio is 0.0558, and design life in years is 18 using the AI method, while the sum of damage ratio on the top of the subgrade and design life in years are 0.0576 and 17, respectively while using the Shell method.

In the case of the spectrum of axle method, loads are considered axle-wise. Single axle with single wheel, single axle with dual wheels, tandem, and tridem axles have been considered. The damage ratio due to the axle loads is computed separately and summed up. The summation of damage ratio at the bottom of the asphalt concrete and on the top of the subgrade is compared, and the smaller of the two is taken as the governing one. Vertical compressive strain on the top of the subgrade is governing in both AI and Shell methods. Sum of damage ratio and governing design life in years using AI method are 0.06138 and 16, respectively. Sum of damage ratio and governing design life in years using Shell method are 0.05328 and 19, respectively. Table 3 indicates the summary and governing design life using distress models in KENLAYER.

Table 3. Summary of design life using distress models in KENLAYER

Method of Treating Traffic	Design Method	Design Life in Years	Governing Design Life
ESAL	Asphalt Institute (AI)	18	**16**
	Shell	17	
Spectrum of Axle	Asphalt Institute (AI)	16	
	Shell	19	

Pavement Performance Using HDM-4

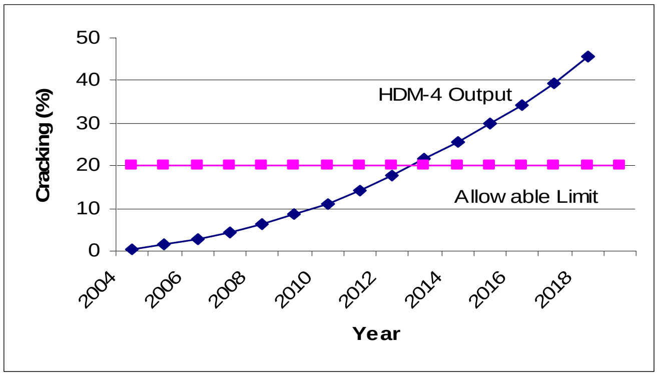
HDM-4 has been used for predicting pavement performance for the analysis period of 15 years using the parameters adjusted for Indian conditions by Jaya (2004) for the four deterioration models. These deterioration models are cracking, raveling, potholing, and roughness. It has been found that these deterioration models are governing for predicting pavement performance out of the eight deterioration models in HDM-4. Allowable limits for pavement distress as per HDM-4 and Indian conditions are given in Table 4. The maximum limit for Indian conditions has been used to determine the maximum performance of the pavement for the available ones. The results using HDM-4 are shown in Table 5 and Figure 3.

Table 4. Allowable limits for pavement deterioration as per HDM-4 and Indian conditions

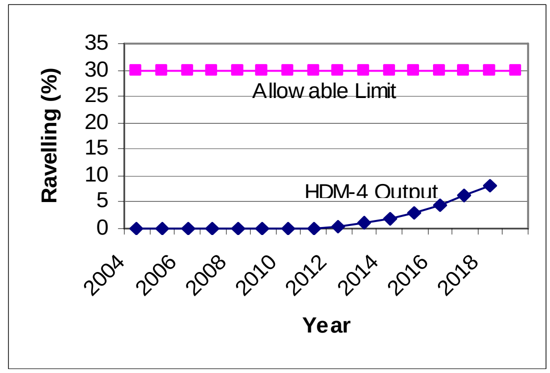
Sr. No.	Deterioration Model	Maximum Limit	
		HDM-4	Indian Condition
1	Cracking (%)	15	20
2	Raveling (%)	20	30
3	Potholing (No/km)	5	8
4	Edge-break (m^2/km)	100	-
5	Rutting (mm)	15	20
6	Roughness IRI (m/km)	6	-
7	Texture Depth (mm)	0.3	-
8	Skid Resistance SCRIM at 50km/hr	0.3	35SN

Table 5. Summary of pavement performance using deterioration models in HDM-4

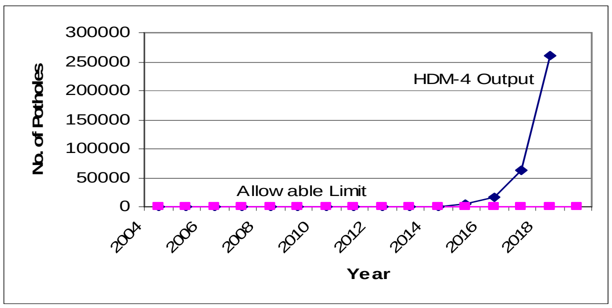
Sr. No.	Deterioration Model	Predicted Pavement Performance in Years	Governing Life in Years
1	Cracking	9	**9**
2	Raveling (%)	>15	
3	Potholing (No/km)	10	
4	Edge-break (m^2/km)	>15	
5	Rutting (mm)	>15	
6	Roughness IRI (m/km)	10	
7	Texture Depth (mm)	>15	
8	Skid Resistance SCRIM at 50km/hr	>15	



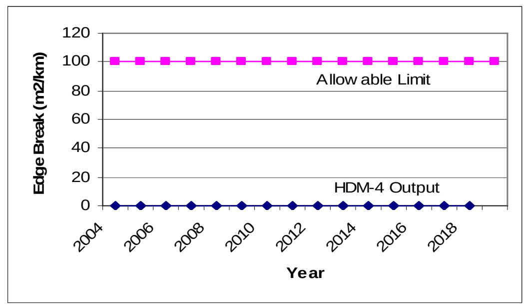
(a) Using cracking



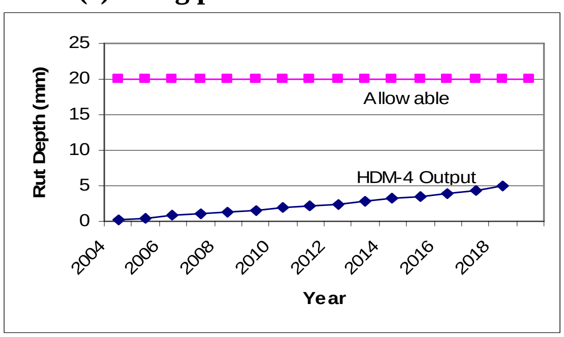
(b) Using ravelling



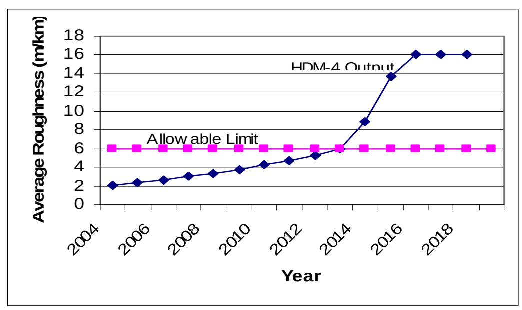
(c) Using pothole



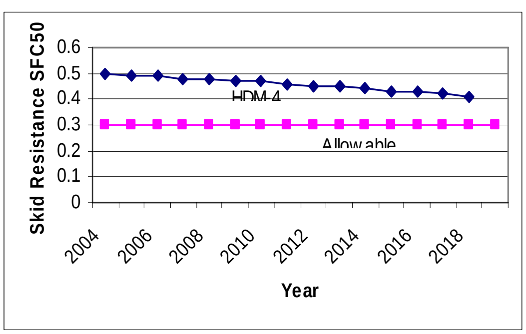
(d) Using edge break



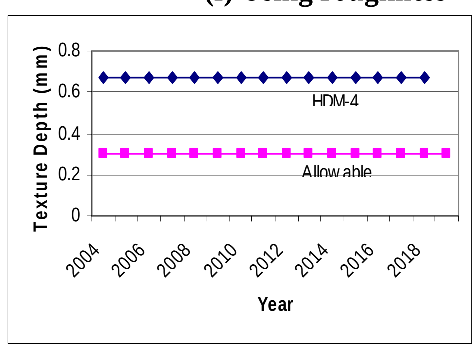
(e) Using rutting



(f) Using roughness



(g) Using skid resistance



(h) Using texture depth

Figure 3. Performance of pavement using deterioration models

Comparison of Pavement Performance Using KENLAYER and HDM-4

The governing design life using distress models in KENLAYER is 16, and it is governed by vertical compressive strain on the top of the subgrade using the AI design method while considering traffic using the spectrum of axle approach. The maximum number of years the pavement performs using deterioration models in HDM-4 is governed by cracking, and it is nine years. However, rutting and cracking distress and deterioration models do not give comparable results. Comparison is made in Table 6.

Table 6. Comparison of pavement performance using distress and deterioration models

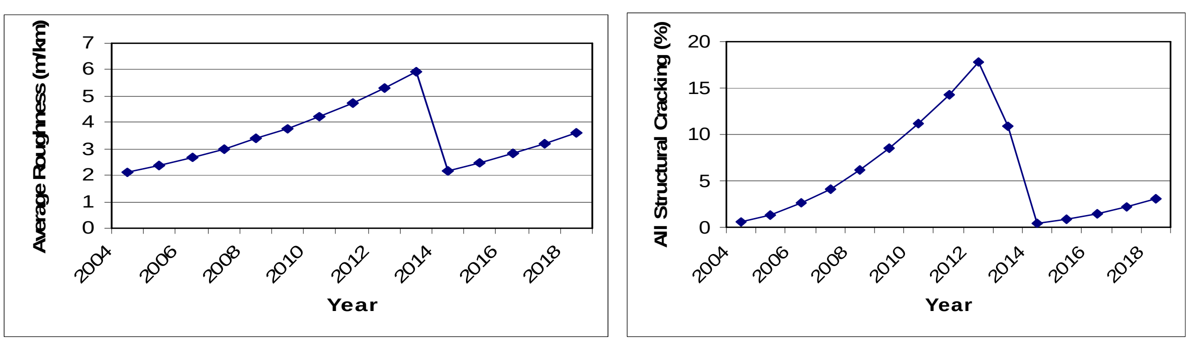
Type of Model		Performance Period
Rutting	Distress Model	16
	Deterioration Model	>15 (analysis period)
Cracking	Distress Model	Large
	Deterioration Model	9

Pavement Maintenance using HDM-4

Eight deterioration models in HDM-4 with their respective maximum limits as per HDM-4 and for Indian conditions are indicated in Table 4. Maximum limit for Indian conditions have been used for maintenance intervention, but limits as per HMD-4 have been used for which there are no standards in India**.** Pavement condition-responsive maintenance has been carried out using HDM-4 for analysis period of 15 years. Only cracking and roughness have been found to be critical and have needed maintenance. The effect of maintenance on both is discussed below.

Effect of Maintenance

Condition-responsive maintenance has been done at the end of 10 years for roughness and 9 years for cracking. Due to the maintenance intervention, roughness and cracking of the pavement have become equivalent to the original pavement as shown in Figure 4.



(a) On roughness **(b) On cracking**

Figure 4. Effect of maintenance

CONCLUSIONS

The following conclusions have been made based on this study:

- KENLAYER can be used to predict the performance of flexible pavement more easily and efficiently because it is user friendly.
- HDM-4 is a powerful system for the analysis of road management alternatives. It can be applied in project analysis, program analysis, strategy analysis, research, and policy studies**.**
- KENLAYER gives comparable results using AI and Shell design methods considering traffic based on ESAL and spectrum of axles approaches.
- Pavement performs for 16 and 9 years using KENLAYER and HDM-4, respectively. Performance is less using deterioration models, which indicate the early failure of the pavement due to various reasons.
- Rutting and cracking distress models in KENLAYER, and rutting and cracking deterioration models in HDM-4 do not give comparable results.
- Project analysis should be carried out using HDM-4 before deciding the maintenance of the pavement because the output shows the annual condition of the pavement.
- The attention of highway agencies has been changed from construction of new pavements to maintenance and rehabilitation of already existing ones.
- Out of eight deterioration models in HDM-4, only cracking and roughness have been found to be critical during the analysis period of 15 years.
- The condition of the pavement has become equivalent to new pavement after condition-responsive maintenance using HDM-4.
- HDM-4 is user friendly software, and it can be used for maintenance of pavements.

RECOMMENDATIONS

- The use of AI design method for determining design life in years by the KENLAYER computer program using spectrum of axle approach is recommended.
- Design life using vertical compressive strain on the top of the subgrade is governing using KENLAYER, and cracking is governing in the case of HDM-4. It is recommended to use cracking deterioration model based on the output of the analysis.

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An Emerging Development Pressure Index for Corridor Planning

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ABSTRACT

Recognizing the importance of consistency in the state's corridor planning activities, the Wisconsin Department of Transportation uses a statewide, systematic process for identifying priority management corridors. As part of this prioritization process, an index is needed to reflect the likelihood of future residential and commercial development. This paper describes a methodology for creating such an index, referred to as the Development Pressure Index (DPI). A high DPI value suggests a higher likelihood of future growth around the highway segment and indicates the need for increased level of service or capacity on the highway segment. A wide range of data, from population and economic projections to land development plans and forest protection programs, are incorporated into the computation of the proposed DPI. A geographic information system is used to perform various spatial join and aggregation methods to derive a set of growth indicators. A scoring and weighing process is then applied to collapse the multiple indicators into one index. The paper concludes by discussing the validity of the proposed methodology and possible directions for further enhancement.

Key words: corridor planning—corridor prioritization—emerging development

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1. INTRODUCTION

State highways represent billions of dollars of transportation capital investment. As with any other transportation asset, a highway system needs continual investment to maintain, update, and expand. Due to the limited availability of public funds and resources, it is important to make these investment decisions in an objective and equitable manner. In Wisconsin, these decisions are made based on the corridor management philosophy and approach, with considerations of the highway facility in the context of surrounding land uses, access management, condition of adjacent facilities, etc. The corridor management process begins by identifying priority corridors that warrant specific attention due to existing mobility problems, safety concerns, or emerging development pressure. Once these priority corridors are identified, a management vision is then developed and implemented for each corridor through the coordinated application of various planning activities, strategies, and tools.

Recognizing the importance of consistency in the state's corridor planning activities, the Wisconsin Department of Transportation (WisDOT) employs a systematic, statewide process for identifying priority management corridors. The process includes two analysis stages: a quantitative analysis conducted at the state level and a qualitative analysis conducted at the district level (WisDOT 2004). In the first stage, WisDOT staff evaluate the state trunk highway (STH) segments (approximately 15,000 segments in total) along three dimensions: mobility, safety, and development pressure. The evaluation along each dimension involves first assigning to each STH segment a set of scores corresponding to the constituting factors. The scores are then weighted and summed across all factors to give an index value. The weighted sum of the mobility index, safety index, and development pressure index forms the final priority score. The priority scores obtained from the quantitative analysis stage are then provided to the district planning agencies for the second stage of the priority corridor identification process. In this stage, the district staff review the priority scores in conjunction with local knowledge and qualitative considerations to determine the high-priority corridors for their respective districts.

This paper describes a framework developed to enhance WisDOT's prior approach for evaluating the development pressure along STH segments during the first stage of their corridor prioritization process. For the purpose of corridor prioritization, development pressure is defined as the likely intensity of future residential and commercial development in the vicinity of a STH segment. High development pressure indicates the need for an increased level of service or capacity on the highway segment. Currently, WisDOT determines development pressure based on three factors: population growth, employment growth, and land conversion rate. The population growth is measured by the GEH statistic, computed based on the population size of the base year (2000) and the projected population for year 2020. The employment growth is also measured by the GEH statistic, computed in a similar fashion. Land conversion is measured by the number of conversions from agricultural/vacant land to residential, commercial, or manufacturing uses. The values for all three factors are first computed for each city, town, and village (CVT) and subsequently mapped to the individual STH segments.

There are at least two issues with WisDOT's current approach for estimating the development pressure along STH segments. First, since CVTs are large administrative areas that may contain up to hundreds of STH segments, the mapping of population growth, employment growth, and land conversion rates from CVTs to STH segments inevitably leads to high homogeneity in the factor values, and the corresponding scores, among the STH segments. This lack of spatial variation and accuracy prevents planning agencies from being able to pinpoint the location of problematic corridors. Therefore, a computational approach that provides higher spatial variation and accuracy is much needed. The second issue with the current approach lies in the fact that only three factors are currently used to measure development pressure. Since richer and more detailed geographical data are becoming available both within and outside of WisDOT, it is highly desirable to identify and include additional, and perhaps better, indicators of development pressure in the computation of corridor priority scores.

In view of the abovementioned issues, in this study we have developed a new Development Pressure Index (DPI) that will be used to replace WisDOT's current set of three development pressure-related scores. The remainder of this paper describes the development and application of the DPI. Section 2 discusses potential indicators of development pressure. Section 3 describes the various data sources considered for DPI computation. Section 4 provides an overview of the new DPI computation framework. Section 5 presents the results obtained from applying the proposed framework to Wisconsin. Finally, Section 6 summarizes the paper and discusses the general applicability of the proposed framework.

2. INDICATORS OF DEVELOPMENT PRESSURE

Our scan of existing literature revealed that most states do not explicitly consider development pressure during their corridor evaluation and prioritization process, though some do consider factors such as population growth projections as indicators of travel demand (for example, Zemotel and Montebello 2002). Therefore, as the first step to developing a new DPI, a panel of transportation planners and engineers from WisDOT was invited to participate in a discussion of what factors would support or impede future development near highway segments. This panel discussion led to the identification of two categories of indicators: inclusion indicators and exclusion indicators.

2.1. Inclusion Indicators

Inclusion indicators refer to the factors thought to be positively related to the level of development pressure. For instance, higher (vs. lower) population growth near a STH segment is an indication of greater (vs. weaker) emergent development pressure. This category includes the three factors already used by WisDOT: population growth, employment growth, and land conversion rate. It also includes factors such as real estate development, utility systems extensions, transportation infrastructure improvements, access management plans, and school and business expansions.

2.2. Exclusion Indicators

In contrast to the inclusion indicators, the exclusion indicators are those thought to be negatively related to the level of development pressure. Examples of these types of indicators include steep terrain, flood plains, and protected forestry. The presence of the exclusion factors near a STH segment is expected to slow down or prohibit future development near the segment.

3. DATA SOURCES

While it is relatively straightforward to identify good candidates for inclusion and exclusion indicators, collecting the necessary data for measuring each of these potential indicators is a challenging process. This is because some data either are not readily available in the desired electronic format or do not cover the entire study area (state of Wisconsin). Some data are proprietary data that either are not accessible or need to be purchased by WisDOT. Other data are simply unavailable from any of the agencies that the research team has contacted.

The final dataset used for this study includes 16 complete data items collected from various organizations; 11 of them represent inclusion factors and 5 are exclusion factors. The sources and contents of these data items are summarized in Table 1. All data items are compiled into ArcGIS-compatible format.

Table 1. Summary of data items used in computing the Development Pressure Index

Name	Source	Content
Inclusion Factors		
Annual Growth Rate (AGR) of traffic	Wisconsin Department of Transportation	Projected traffic AGR from 2004 to 2024
Employment Growth	Wisconsin Department of Transportation	Projected growth in employment from 2000 to 2025 by traffic analysis zones (TAZ)
Land Conversion	Wisconsin Department of Revenue	Observed growth in number of land parcels converted into residential, manufacturing, or commercial use from 1990 to 2006 by CVT
Population Growth	Wisconsin Department of Administration	Projected growth in population from 2000 to 2025 by CVT
Agriculture Land Cover	Wisconsin Department of Natural Resources	Areas covered by agricultural land use
DPI Student Enrollment Growth	Wisconsin Department of Public Instruction (DPI)	Observed growth in number of enrollments from 2000 to 2006 by school district
Land Value	Wisconsin Department of Revenue (Research and Policy Division)	Observed growth in total equalized value of residential, manufacturing and commercial land and improvement from 2000 to 2006
Planned Development	Wisconsin Department of Transportation	Areas where a traffic impact analysis (TIA) has been conducted recently or where known real estate development has been planned
Railway Stations	Wisconsin Department of Transportation	Point locations of existing and planned rail stations
Tax Incremental District (TID)	Wisconsin Department of Revenue	Number of active TIDs by CVT. TIDs are created by cities and villages to financially support new development or redevelopment of blighted areas, or areas in need of rehabilitation
Transition Lanes	Wisconsin Department of Transportation	Location of 'tapers' – where the number of lanes reduces and traffic bottleneck is likely to occur – on the STH network
Exclusion Factors		
Elevation	Wisconsin Department of Natural Resources	Digital Elevation Model of Wisconsin that is used to determine terrain slopes of 100m by 100m grid cells
Poverty	Applied Population Laboratory, University of Wisconsin	Proportion of population below poverty line by census tract
Managed Forest Law	Wisconsin Department of Natural Resources	Polygons representing land parcels currently enrolled in the Managed Forest Law program, which prevents the land from being used for non-agricultural development
Managed Lands	Wisconsin Department of Natural Resources	Polygons representing land parcels that the WDNR has acquired in fee, easement, or lease
Wetland and Open Water Land Cover	Wisconsin Department of Natural Resources	Areas covered by wetland or open water

4. DPI COMPUTATION FRAMEWORK

The computation process by which the 16 input data items are combined to produce the final DPI is depicted in Figure 1. The process consists of four main steps, as discussed below.

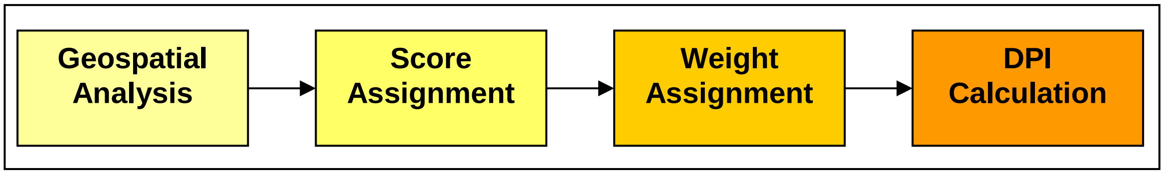

Figure 1. Main steps in the process of computing the Development Pressure Index

4.1. Geospatial Analysis

The first step in the DPI computation process is to produce an array of 16 attribute values for each STH segment based on the input data items. Since the 16 input data layers refer to different types of spatial objects, including lines (highway segments), points (site locations), and polygons (CVT, traffic analysis zones, school districts, census tracts, and land parcels), different spatial join operations have been devised to map the attribute values from each of these data (source layers) onto the STH segments (target layer). This geospatial analysis step is key to the integration of various spatial data sources. A total of four types of geospatial operations are used. These are illustrated in Figure 2 and are discussed in turn below.

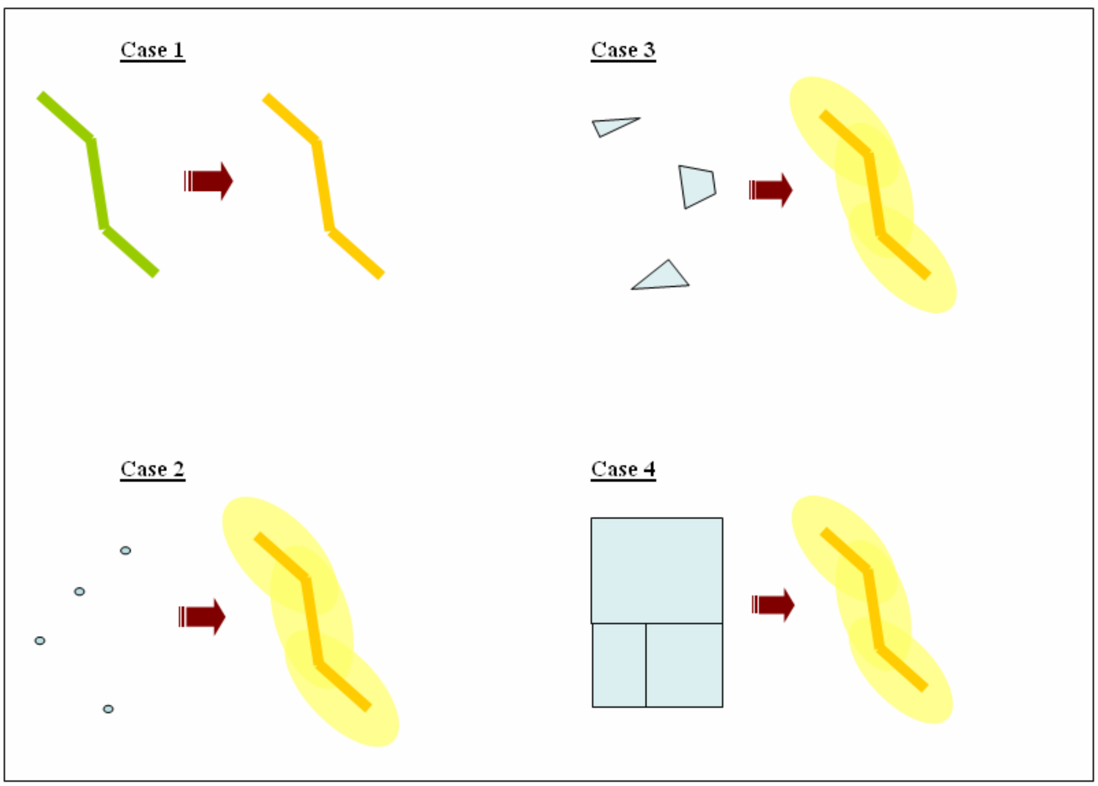

Figure 2. Operations involved in computing the Development Pressure Index

Case 1. Mapping from STH Segments to STH Segments

In the simple case where the source layer describes the same polyline objects as those in the target layer (for example, AGR Traffic and Transition Lanes), the source layer can be directly mapped to the target layer through a simple one-to-one join based on object ID.

Case 2. Mapping from Points to STH Segments

This case includes the Railway Stations data. The mapping from points to polylines requires that a buffer first be created around the highway segments. The number of rail stations that fall within each buffer area is then obtained using a location-based spatial join. A buffer size of 0.25 miles was used.

Case 3. Mapping from Polygons to STH Segments

Several data items describe polygons in space, including Agriculture Land Cover, Traffic Planned Development, Managed Forest Law, Managed Lands, and Wetland and Open Water Land Cover. The influence of each of these factors on a highway segment is measured by the amount of the corresponding polygons that fall within a buffer area defined around the segment. A buffer radius of one mile is applied to all of these polygon data.

Case 4. Mapping from Zones to STH Segments

A common type of source data is zone data. These are data that describe the quality of administratively or analyst-defined spatial units that are nonoverlapping and that collectively cover the entire study area. The different zoning systems involved in this study include CVT, traffic analysis zones, school districts, census tracts, and grid cells. Mapping attribute values from these zoning systems to STH segments requires a buffer to be first defined around the segments, then followed by spatial joins based on location. When multiple zones are overlaid onto a single segment buffer, appropriate aggregation methods (such as sum, average, or max) are applied to collapse multiple zonal values into one. The choice of aggregation methods depends on the nature of the factor.

4.2. Score Assignment

Once all 16 factor values are obtained for each STH segment, the next step is to assign a score for each factor. We accomplish this by first ranking all segments based on each of the 16 factors. Each segment is then assigned with 16 separate score values, ranging from 0 to 10, based on its percentage ranking with respect to the 16 factors. For any given factor, a score value of 0 indicates a relatively low likelihood of emerging development due to that factor; whereas a value of 10 indicates a high likelihood of development. For example, based on the number of Tax Increment Districts (TID) computed for each segment, the segments are rank ordered and are assigned a score of 0 if the segment is among the bottom 40% in the ranking, 3 if between 40% and 60% of the ranking, 6 if between 60% and 80% of the ranking, and 10 if above 80% of the ranking. For exclusion factors, the higher a segment is ranked, the lower its score value will be. For example, after STH segments are ranked based on the amount of Managed Land within a one-mile radius of the segments, a score value of 0 is given to those segments ranked 90% or above, 2 to those ranked between 80% and 90%, 5 to those ranked between 60 % and 80%, and 10 to those ranked in the bottom 60%.

4.3. Weight Assignment

Depending on the spatial accuracy, timeliness, and other qualities of the input data, the 16 factors represent varying degrees of strength in explaining emerging development pressure. In order to determine the relative importance of these factors, a peer review process involving five panelists from the Bureau of Planning and Economic Development at WisDOT was conducted. Each panelist was asked to rate the 16 factors using a five-point scale, with 1 being very unimportant and 5 being very important. For each factor, the average of the scores provided by the five panelists was computed and used as the weight of the factor. The five factors found to have the highest weights were traffic annual growth rate (4.6), employment growth projection (4.4), population growth projection (4.2), land conversion rate (4.2), and planned development (4.0). The ones with the lowest weights are railway stations (2.0), poverty rate (2.0), elevation (2.0), managed forest law (2.2), and managed land (2.4).

4.4. DPI Calculation

Once the score values are assigned and the weights are defined for each factor, we can compute a raw DPI value for each STH segment as the weighted sum of score values:

$$Raw\_DPI_l = \sum_k w_k \cdot S_{lk} \tag{1}$$

where

Raw_DPI_l is the raw DPI value of the l^{th} STH segment

w_k is the weight given to the k^{th} factor

S_{lk} is the score assigned to the l^{th} STH segment for the k^{th} factor

Based on the above equation, the highest raw DPI value that a highway segment can possibly attain is 160. In practice, however, the highest value found across all STH segments is likely to be lower. We denote this highest observed value as Max_Raw_DPI. This value is then used to normalize all DPI values as follows so that the final DPI takes a value between 0 and 10:

$$DPI_l = \frac{Raw\ DPI_l}{Max\_Raw\_DPI} \cdot 10 \tag{2}$$

5. RESULTS

The results obtained from applying the DPI computational process to the STH network of Wisconsin are described in this section. These results have been verified using statistical analysis and thematic mapping.

5.1. Statistical Analysis

Since the DPI values are computed as a function of many scores, the level of correlation among the constituting scores is a concern. The simultaneous inclusion of highly correlated indicators should be avoided, as it would lead to biased results. Our analysis of correlation coefficients between all pairs of 16 scores (as shown in Table 2) reveals that the highest correlation is between the scores for TID and Land Value, with a correlation coefficient of 0.53. The correlation between most of the remaining pairs of scores is generally low, suggesting that little “double counting” effect has been introduced into the DPI values.

The normalized DPI values obtained for the 14,912 segments on the Wisconsin STH are found to have an average value of 4.85 and a standard deviation of 1.60. The frequency and cumulative frequency distributions of the values are shown in Figure 3. Overall, the frequency distribution resembles a bell shape, which typically suggests a normal distribution.

Table 2. Correlation coefficients between all pairs of scores constituting the DPI

Correlation Coefficient	AGR Traffic	Employment Growth	Land Conversion	Population Growth	Agricultural Land Cover	DPI Student Enrollment	Land Value	Planned Development	Railway Stations	TID	Transition Lanes	Elevation	Poverty	Managed Forest Law	Managed Lands	Wetland and Open Water Land Cover
AGR Traffic	1.00															
Employment Growth	0.06	1.00														
Land Conversion	0.26	0.03	1.00													
Population Growth	0.13	0.36	0.21	1.00												
Agricultural Land Cover	0.20	0.01	0.43	0.19	1.00											
DPI Student Enrollment	-0.03	0.01	0.02	0.04	0.02	1.00										
Land Value	0.22	0.26	0.50	0.27	0.27	0.05	1.00									
Planned Development	0.26	0.10	0.29	0.22	0.25	0.03	0.41	1.00								
Railway Stations	0.06	0.02	0.12	0.04	0.01	0.01	0.13	0.05	1.00							
TID	0.11	0.22	0.18	0.10	0.16	0.01	0.53	0.21	0.09	1.00						
Transition Lanes	0.07	0.24	0.06	0.12	0.01	0.04	0.51	0.22	0.06	0.46	1.00					
Elevation	0.00	0.03	0.02	0.07	0.07	0.02	0.09	0.07	0.01	0.08	0.18	1.00				
Poverty	0.02	0.15	0.09	0.10	0.13	-0.01	0.15	0.04	0.02	0.09	0.12	0.05	1.00			
Managed Forest Law	0.05	0.14	0.07	0.13	0.08	-0.04	0.18	0.06	0.04	0.13	0.12	0.01	0.42	1.00		
Managed Lands	0.13	0.00	0.30	0.18	0.36	0.06	0.12	0.12	-0.04	-0.21	-0.05	0.10	0.08	0.03	1.00	
Wetland and Open Water Land Cover	0.02	0.23	0.03	0.20	0.05	0.06	0.13	0.07	0.00	0.12	0.15	0.01	0.11	0.12	0.07	1.00

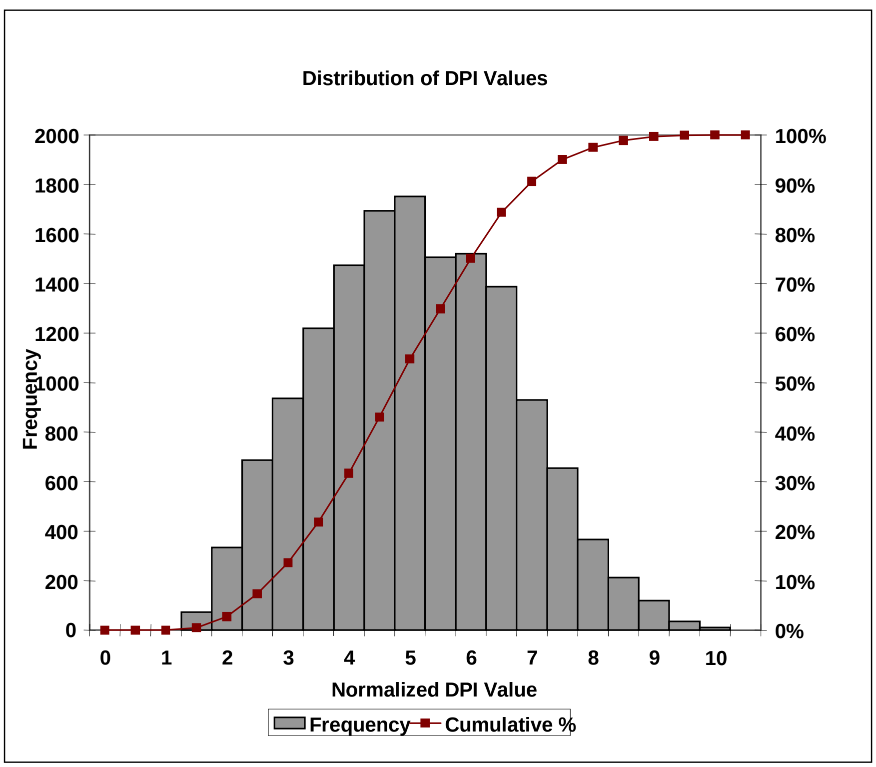

Figure 3. Distribution of DPI values computed for Wisconsin STH segments

5.2. Thematic Map

The spatial distribution and variation of the DPI values can be best visualized in a thematic map, shown in Figure 4. The darker the line color, the higher the development pressure is. The thematic map suggests that there is significant variation in DPI values across space. The map also shows spatial clusters of segments with similar DPI values.

The areas with a concentration of high DPI values are highlighted in Figure 5. These areas include Appleton, outskirts of Milwaukee, south of Madison, and Eau Claire. Detailed maps of these areas were reviewed by selected WisDOT staff who have strong knowledge of the state's development pattern. The reviewers verified that these areas are indeed among those undergoing significant growth and expecting increased traffic volumes in the near-term future. It is also noted that, although currently urbanized areas tend to receive relatively high DPI values, the highest DPI values are found in the urban fringes, where spillover growth and development is imminent.

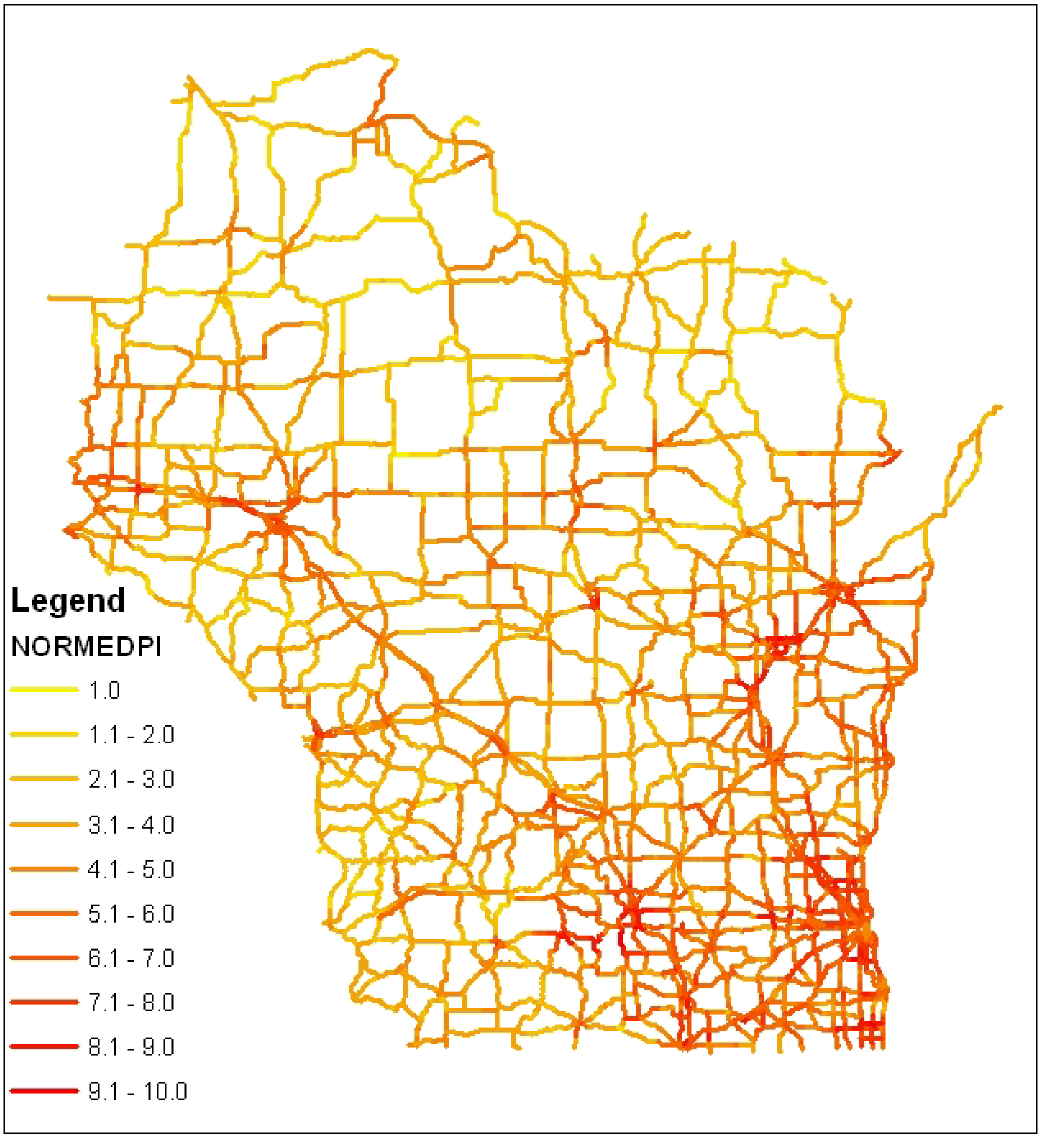

Figure 4. Thematic map based on normalized DPI values

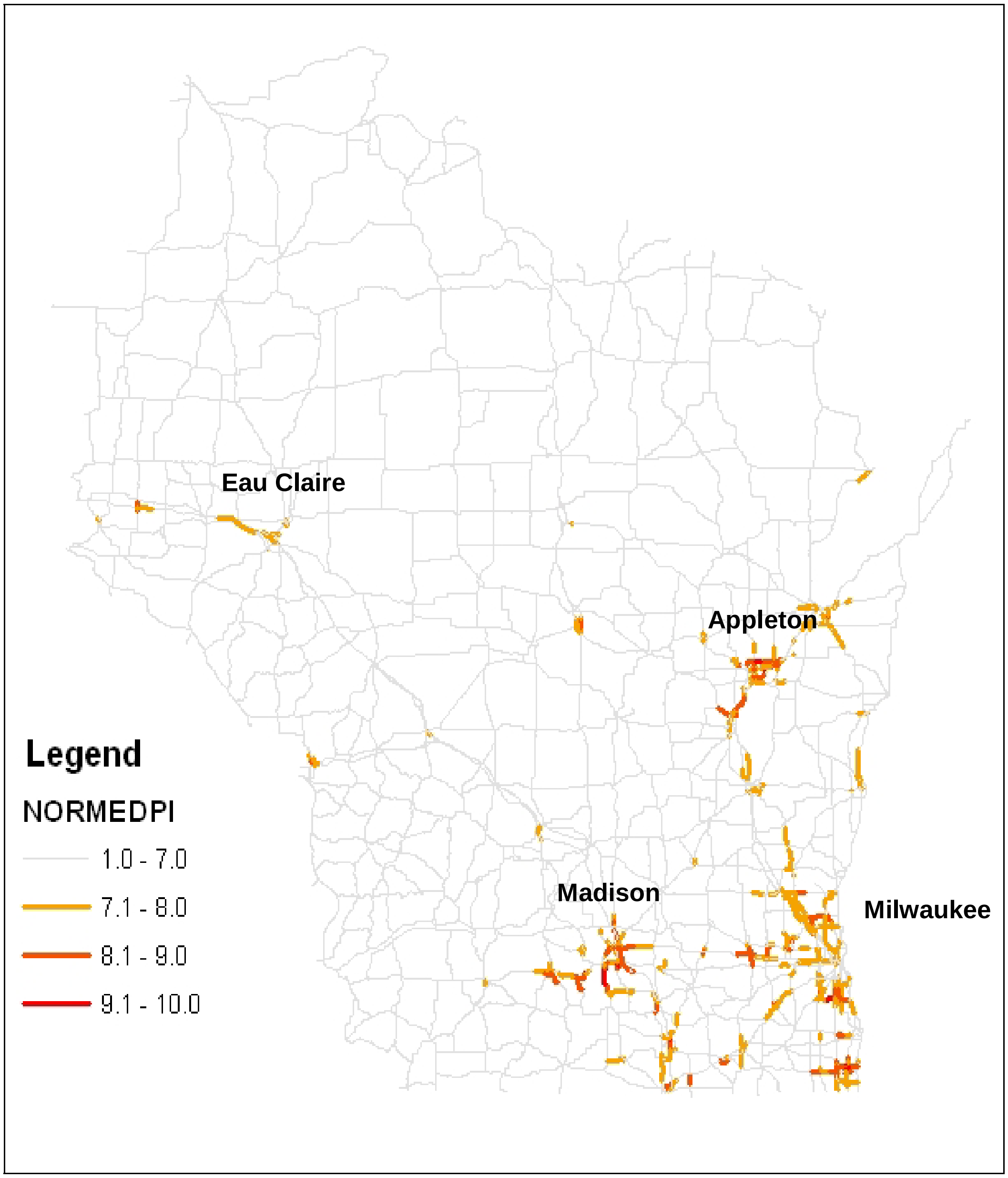

Figure 5. Areas with highest levels of development pressure, as indicated by the proposed DPI

6. CONCLUSIONS

This paper has described the systematic development and application of a DPI that can be used as a predictor of emerging development pressure along a STH segment. The DPI represents an integration of a rich set of geographical data that are considered as preliminary indicators of development pressure. The DPI is to be used in conjunction with additional indices of mobility and safety to help WisDOT planners identify priority corridors for further planning activities. Our empirical results and the subsequent assessment of the results reveal that the proposed methodology provides an effective and objective way to account for multiple influencing factors of development pressure.

It should be noted that the general methodology presented in this study is applicable to the planning-level evaluation and prioritization of corridors in future planning horizons. It is also potentially applicable to communities other than the state of Wisconsin. However, the methodology must be tailored to the planning goals and the data available for the planning activities. In particular, the design of the scoring and weighting schemes depends greatly on the quality of the data available for the indicators of consideration. The scoring and weighting schemes also need to be based on the relative emphasis that the public agency and other stakeholders place on the various indicators.

Many challenges related to data availability and data quality have been encountered during the course of this study. These challenges arise from the fact that the analysis of development pressure is a data intensive exercise. The analysis calls for data produced by many different agencies that often differ in format and nature, making the task of acquiring and integrating these various datasets laborious and difficult. Moreover, as the data are updated by the different agencies over time, tracking down the latest data and keeping the DPI values up to date presents yet another challenge. The lessons learned from this study point to the need for improving the data interoperability and data accessibility within state departments of transportation and across various state departments in order to support a comprehensive corridor planning process.

It is envisaged that, as geographic data of higher quality becomes available in the future, the DPI computational methodology proposed in this study can be further refined in at least a couple of ways. First, the DPI is currently computed for highway segments with a priori defined start and end nodes. The definitions of these segments are often too coarse and too inflexible to support the accurate identification of localities with emerging development. A computational method that is not bound by the STH segment definitions and that allows the DPI to be computed at a higher spatial resolution is desired. However, such a refined methodology would be effective only if data of high spatial resolution were available. Second, the DPI is currently computed based on variables and weights that have been selected in an ad hoc fashion. If historic data were available about the traffic volumes (endogenous variable) and the various development pressure indicators (exogenous variables) being considered, one could develop a multivariate time-series model to determine the statistical significance and the relative explanatory power of the exogenous variables. The model estimation results could then be used to inform the selection of input variables and the associated weights.

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Deploying Hybrid Electric School Buses in Iowa

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ABSTRACT

There are 450,000 school buses in the United States that transport 25 million children on approximately 10 billion student trips each year. These buses consume 1.1 billion gallons of diesel fuel and emit thousands of tons of pollutants per year. School buses represent a major segment of our country's transportation sector in terms of trips delivered, fuel consumed, and pollutants emitted.

In addition to policy changes, there are technological options for reducing bus emissions. These options include using different fuels, such as biodiesel or natural gas, and add-on emission control devices, such as particulate filters and oxidation catalysts. Hybrid electric technology is another option. Hybrids are available in the passenger vehicle market as well as the transit bus market. Currently, there are no commercially available hybrid school buses.

Hybrid electric school buses have the potential to reduce emissions and reduce the overall life-cycle cost when compared to conventional diesel buses. The technology has been demonstrated in passenger vehicles and transit buses. This project is to demonstrate that hybrid school buses can provide an economically viable alternative for school districts seeking to reduce emissions from their fleets. However, to penetrate the school bus market, there must be a demonstration of the technology. As part of a national coordinated effort, two school districts in Iowa have stepped forward to join a national consortium to encourage the demand for hybrid electric school buses. The buses will be deployed in the school districts by the spring of 2007. The Center for Transportation Research and Education will monitor and evaluate the buses' performance for this important project.

Key words: biodiesel—buses—hybrid

Sign Inventory: Legacy vs. New Technology

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ABSTRACT

The Iowa Department of Transportation (Iowa DOT), through its Sign Management Task Force, is embarking on a comprehensive effort to develop a sign management system that would allow the Iowa DOT to improve sign quality, as well as manage all aspects of signage, from request, ordering, fabricating, installing, maintaining, and ultimately to removing, and provide the ability to budget for these key assets on a statewide basis. This paper reports on the task force's efforts to address issues related to referencing sign locations as part of the inventory building process. This effort provides a contrast between legacy referencing systems (route and milepost) and GPS-based techniques (latitude and longitude). A summary comparison of field accuracies using a variety of consumer grade devices is also provided.

Key words: GIS—GPS--inventory—quality—signs

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Evaluation of Iowa Climate Data for the Mechanistic-Empirical Pavement Design Guide

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ABSTRACT

The new Mechanistic-Empirical Pavement Design Guide requires detailed climate data as part of the analysis input files. The national effort to develop the new pavement design guide applied historic climate data records as the future predicted climate. However, early findings by an Iowa State University climate study regarding the implementation of the new pavement design guide for the Iowa Department of Transportation show that the use of historic climate data is not adequate for predicting future pavement performance. A virtual climate database is needed to properly project future climates and predict future pavement performance.

Key words: climate—mechanistic-empirical pavement design guide

INTRODUCTION

In the late 1990s, the highway community embarked on a journey to elevate the national methodology used to design the structure of highway pavements. Most states' current practice uses the empirical results of a single research effort from the late 1950s known as the AASHO Road Test. It is commonly accepted that the pavement design process developed from the AASHO Road Test is no longer applicable to today's increases in traffic and advances in paving materials and testing. The new process, known as the Mechanistic-Empirical Pavement Design Guide (MEPDG), takes a vast array of knowledge and research on traffic, climate, materials, and pavement mechanics to predict the performance of pavements.

Each input component of the MEPDG is formatted to account for variations that naturally occur. The traffic pattern changes through the course of the day. The climate changes hourly and seasonally. The pavement materials respond differently as the traffic and climate change. In an attempt to capture these changes, the MEPDG requires large databases to reflect the project traffic, predict climate, and encompass the range of material responses. The ability to make these estimates over a typical pavement life of 20 to 30 years seems somewhat impractical; however, through a careful and conscience effort, this pavement design process will improve the ability to predict future pavement performance. This paper focuses on the requirements of the climate database.

CLIMATE MODEL

The NCHRP project to develop the national MEPDG model elected to compile climate data using the historic records of weather stations across the country. The required climate data includes hourly air temperature, hourly wind speed, hourly percent sunshine, hourly precipitation, daily maximum solar radiation, and monthly humidity. Only a limited number of weather stations could provide the level of climate data required for the MEPDG. For Iowa, only 15 weather sites are included in the MEPDG national database. While the early versions of the MEPDG software only contained about 5 years of data, the current version has more than 10 years (starting with the 1990s). The software uses the available climate data and repeats the climate data up to the number of pavement performance years desired. For example, a 30-year pavement design would apply a 10-year climate database three times.

This process raises some important questions. How much historic climate data do you need to properly represent the future climate? Should we simply repeat the data to accomplish the pavement analysis period? Are the critical climate features appropriately applied in the MEPDG model? The discussion of these questions leads to the decision to build a "virtual" climate database using the broader historic understanding of weather patterns.

ANALYSIS

The MEPDG research team concluded that 15 to 20 years of climate data was sufficient to represent the future climate in a target location. However, a simple analysis by Iowa State University climatologists demonstrated that temperature trends in Iowa over 10-year periods were dramatically different. See Table 1. Which 10-year period best represents the next 30 years of Iowa's climate? The 1970s may best represent the average temperature. The 1980s represent an above-normal temperature period, and the 1990s represent below-normal high temperatures and above-normal low temperatures. If we have to select a single 10-year period of climate data (and repeat it three times for a 30-year analysis), what 10-year period should we choose?

Table 1. Iowa climate averages and extremes

Decade	Average Daily High Temperature (°F)	Average Daily Low Temperature (°F)	100°F + Events
1970s	58.9	37.4	593
1980s	59.6	38.1	2073
1990s	58.4	38.3	235

Even if we could acquire all 30 years of data, what is the proper order of the 10-year periods? For asphalt pavements, a true test of rutting resistance occurs in the first 5 years during the heat of the summer. The test of resistance to low-temperature transverse cracking occurs after 10 years of binder aging. A conservative approach to the pavement performance analysis would deliberately use harsh climate patterns to measure critical pavement performance criteria. While this approach seems overly aggressive, the opposite approach (mild climate input) tells the pavement designer very little about the capability of the proposed pavement.

Long-term climate prediction is understandably a soft science. An approach that builds a “virtual” climate database using all of the available historical statistics should be better than selecting any 10-year period of climate history. See Figure 1. The broader knowledge of climate patterns and norms includes not only the daily averages, highs, and lows, but also the distribution of these parameters. With standard statistical tools, the climatologist can build a virtual 20-, 30-, or 40-year MEPDG climate input database. This database could represent the normal distribution of climate and the cycles of extremes. See Figure 2. Further, the virtual database could be manipulated to force some extreme climate events into specific pavement performance time windows to test the limits of the pavement design.

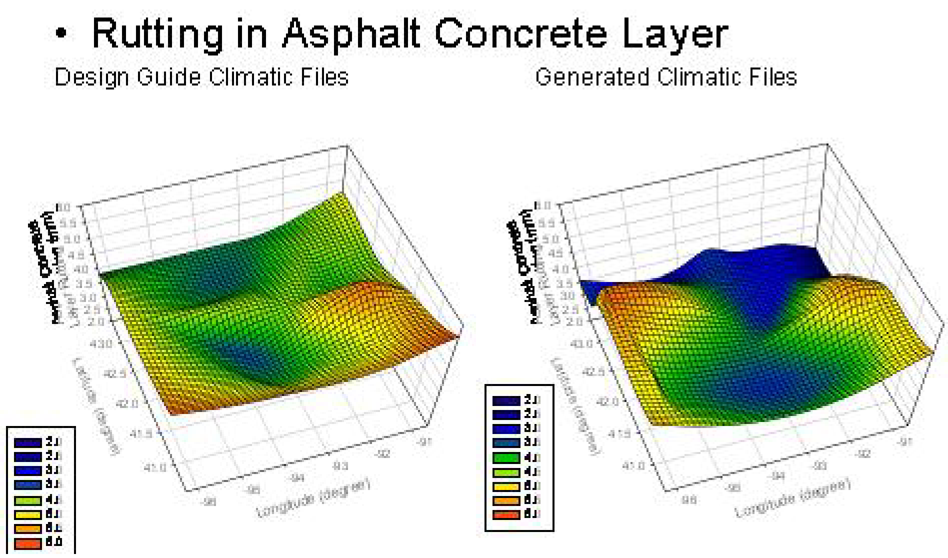

Figure 1. Comparison of repeated 10-yr data and 20-year data, medium-volume traffic

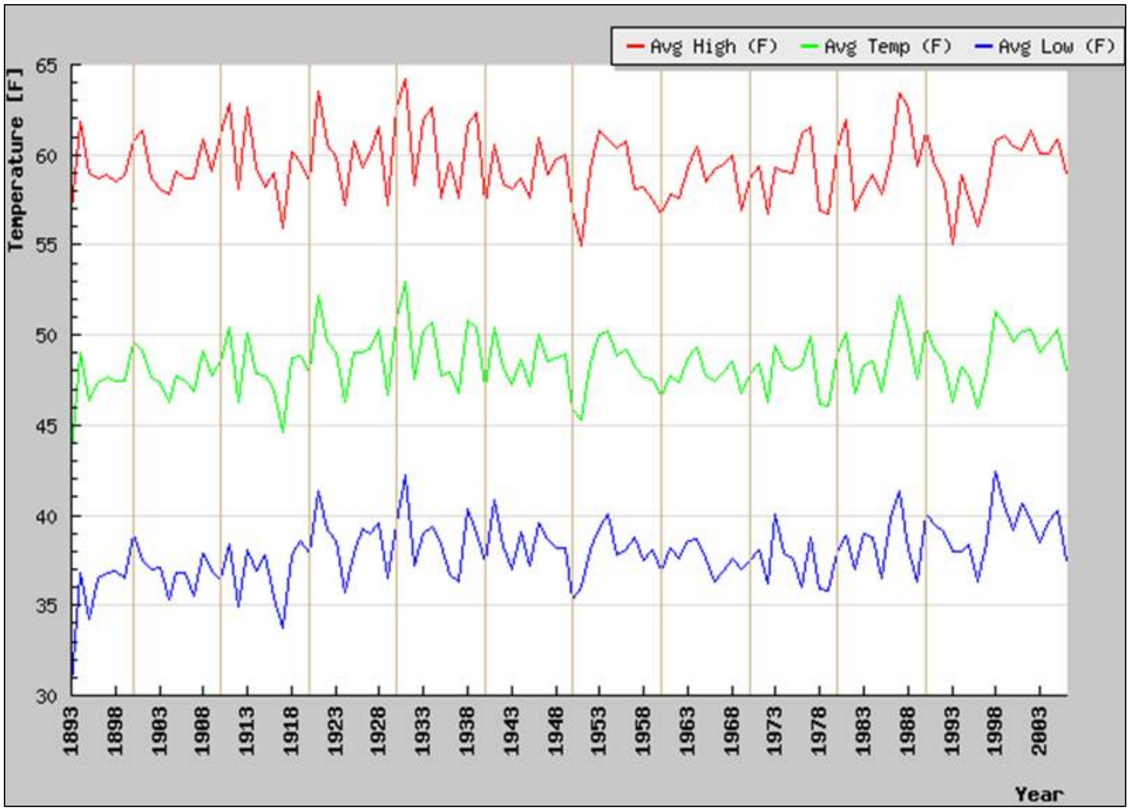

Figure 2. Historic record of climate cycles, Ames, IA

CONCLUSION

What is the future of climate databases for the MEPDG? In reality, the pavement designer needs both the historic climate and a virtual climate. The historic climate is critical for the validation and calibration of the pavement performance curves. Our ability to predict pavement performance is founded on our ability to quantify past pavement performance. The historic climate data should be matched (as close as possible) to the measured historic pavement performance to accurately model pavement performance. Historic climate database files are also needed to perform forensic studies on existing pavements with premature distress or failures.

A virtual climate database, using broad historical trends, will best predict the pavement performance into the future. The virtual model can better project historical cycles and trends in climate than any fixed 10- to 20-year historical climate record. With a careful adjustment of the virtual database, a pavement design performance prediction can reflect the ability of the pavement to perform under extreme climate limits.

ACKNOWLEDGEMENTS

The author recognizes the efforts of Iowa State University's Department of Civil Engineering MEPDG Implementation Team and the Department of Geological and Atmospheric Sciences. Particular recognition goes to Tamer Breakah and Daryl Herzmann for their efforts to analyze data and prepare the graphs used in this paper.

New Film Thickness Models for Iowa Hot Mix Asphalt

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ABSTRACT

Beyond rutting and fatigue, mixture durability is a key hot mix asphalt (HMA) performance characteristic. Mix design criteria in Iowa uses film thickness to ensure adequate mix durability. The current film thickness model is based on 1940s technology and analysis approaches. The use of today's computing power and mathematical models gives us the opportunity to better understand film thickness as a tool to measure HMA durability. A new model based on random spatial distribution of particles is applied to the film thickness concept and compared to Iowa DOT HMA mixtures. The results indicate that this new model may better measure mixture durability because it is sensitive to more variables that impact durability.

Key words: film thickness—hot mix asphalt

INTRODUCTION

The current standard model for calculating film thickness is not sufficiently detailed to adequately reflect differences in hot mix asphalt (HMA) mixtures; and therefore, has limited value as a tool to evaluate research or mix designs. Modifications to the model (or replacement of the model) would give practitioners a better tool to assess the durability potential of an HMA mixture. This study provides the asphalt community with new models with improved approaches to calculating film thickness that better reflect the unique properties of each HMA mixture. The study examines the historical development and application of the standard film thickness model. The proposed film thickness models account for the individual aggregate source gradations, specific gravities, and particle shape that comprise the HMA blend. The study provides a practical approach to the significant contribution of the mineral filler as both an aggregate and asphalt binder extender. These parameters were not adequately accounted for prior to this study. Based on the analysis in this study, future studies of HMA durability will have a more accurate perception of film thickness to compare differences in HMA durability.

Film thickness is a computed, not measured, value that defines the thickness of the effective asphalt binder coating on each particle in the mixture and is used to ensure that the HMA has adequate asphalt binder to achieve a desired level of mix durability. The procedure for computing the surface area was derived from the 1940s Hveem mix design process used to determine a *target* asphalt binder content and only requires the weighted proportion of the combined aggregate on each sieve. Any differences in aggregate particle specific gravity, shape, and texture are not taken into account. The film thickness value "assumed that all the asphalt exists in the form of uniform films as long as appreciable air voids exist" (Campen et al. 1959). The authors recognized that this assumption was not totally correct, but it was adequate for the purpose of their study. This research was undertaken to replace the standard model using the knowledge and tools available today.

Most of the current HMA research efforts focus on rutting and fatigue. Durability is the third key mixture performance criteria. The contractor is focused on keeping the asphalt binder content to a minimum, but the agency/owner's interest should ensure that the finished pavement is durable (adequate asphalt binder content). Film thickness is one computed parameter to define sufficient asphalt binder for durability. Both film thickness and voids in the mineral aggregate (VMA) are products of durability research in the mid-1950s, but later studies report that neither approach to define mixture durability was founded on extensive research. Research in the 1960s and 1970s conclude that durability is a function of film thickness, air voids, and permeability. Recent studies, starting in the mid-1990s, developed mixed results regarding the correlation between film thickness and mixture durability. Although some studies questioned the film thickness equation, they all used the 1942 Hveem table to determine aggregate surface area and applied Campen's 1959 approach for determining film thickness.

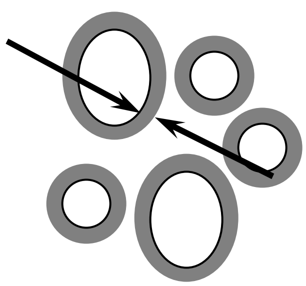
The current procedure for determining surface area of the total aggregate blend only requires the gradation expressed as the total percent (by weight) passing on each sieve. Each percent passing value represents all particles smaller than that sieve. Therefore, the surface area values are not a direct expression of total surface area for aggregate particles on a specific sieve and do not account for differences in aggregate particle specific gravity. Studies that examine differences in film thickness values in an attempt to identify trends in HMA mixture performance are not comparing equivalent film thickness values when the mixtures have aggregates with dissimilar specific gravities. The procedure for computing the film thickness value should account for known characteristics of the aggregate and may lead to a better understanding of the impact of film thickness on mixture performance.

NEW FILM THICKNESS MODELS

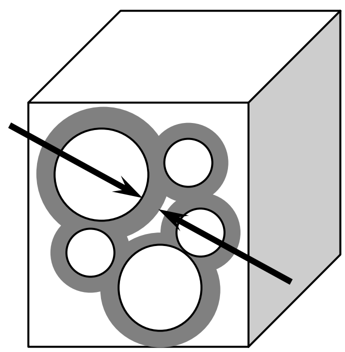
The current surface area factors are a product of a time period when engineers developed charts, tables, and nomographs to simplify the calculations. Any new approach to measuring film thickness should account for today's common practice that the aggregate gradation is a blend of multiple aggregates from different sources and that the as-constructed density is different from the mix design density. The proposed INDEX Model uses the fundamental principles of weight, volume, specific gravity, and particle geometry to calculate a theoretical surface area of each aggregate particle. The resulting aggregate surface area is a better approximation of the true surface area. The VIRTUAL Model approach is a further improvement using theoretical techniques to place the particles in a virtual three-dimensional matrix, fills the void space with effective asphalt, and measures the thickness of the asphalt from the particle surface to the air void space. The VIRTUAL Model requires knowledge of the HMA mixture volumetrics.

The proposed procedures generate two different film thickness values as shown in Figure 1. The first is an extension of the past practice of a uniform coating index, and the second is a new value based on a virtual three-dimensional model. Both procedures account for multiple aggregate sources, including differences in gradation and specific gravity. The INDEX Model can also account for particle shape but cannot reflect the impact of as-constructed air voids. The VIRTUAL Model does account for as-constructed air voids, but does not adjust for different particle shapes.

A large portion of the aggregate surface area is attributed to the mineral filler. For this study, particles less than 10 micron are not included in the determination of particle surface area, but are considered asphalt binder extenders. The volume of these particles is added to the asphalt binder in the VIRTUAL model.



Standard Model
INDEX Model,
Equal & uniform coating



VIRTUAL Model,
Aggregate surface to air void

Figure 1. Differences in proposed film thickness models

FILM THICKNESS RELATED TO IOWA MIX DESIGNS

Phase I of this study developed the INDEX Model and VIRTUAL Model to replace the standard film thickness equation. This phase further demonstrated how the new equations responded to changes in mixture characteristics. Phase II of the study applied the new equations to two HMA durability studies. The first durability study used one mixture and the second used six mixtures. The results of Phase II showed that the multiple variable studies (the second study) have difficulty clearly identifying mix durability. Durability is not a single parameter performance characteristic, and the common compacted mixture test methods give different responses for different mixtures. If we examine a large population of mixtures for durability, we would expect to have the same difficulty distinguishing durability from other mixture performance characteristics.

The impact of the new film thickness equations on field mix designs cannot examine the desired limits of mixture durability but can look at how the new film thickness values compare to the current film thickness values using *real* mix designs. How will the new film thickness equations change the preparation of mix designs? If the INDEX Model or VIRTUAL Model is used for mix design, will the range of film thickness values change from the current 8 to 15 microns?

Actual HMA mix designs represent a broad spectrum of mixtures. Mix designs for low-volume pavements are different from mix designs for Interstate pavements. Typically low-volume mixes have higher amounts of sand. High-traffic mixes require more compaction energy and tend to have lower amounts of asphalt binder. Numerous mixture characteristics will influence the final computed film thickness value.

Phase III of the study started with 348 approved mix designs for projects on Iowa highways and roads. The list included mixes for all traffic levels in all six districts of the state from the 2002 and 2003 construction seasons. The software used in Iowa for all mix designs is known as SHADES. One component of this software measures the accuracy and precision of several components of the laboratory mix design process. Each mix design is given a rating of excellent, good, fair, or poor. To get a better understanding of the impact of the new film thickness equations, only 280 of the mixtures with a good or excellent rating are used. This reduces the variability of the results due to laboratory error. The group of 280 mixtures was further reduced to 268 by eliminating the nine 100K mixtures and three incomplete mix designs. Table 1 summarizes the breakdown of the 348 mix designs.

For the purposes of this study, the 268 mix designs represent the total population of mixtures used in Iowa. The Iowa Department of Transportation (Iowa DOT) database does not include the individual gradations of each aggregate source used in the mix. To accomplish the Phase III objective, the database was supplemented with complete mix design reports for 40 of the 268 mix designs. Ten mix design reports were randomly selected from each of the four primary traffic equivalent single axial load (ESAL) levels to produce a stratified random sample. Only mix designs with the SHADES excellent rating for laboratory mix design process were considered for the 40 mix sample to reduce the affect of laboratory variation.

Table 1. Summary of Iowa mix designs

			Number of mix ratings excellent or good	
Mix design rating	**No. of mixes**	**Mix design level**	**Total**	**Used**
Excellent	106	100K ESALs	9	0
Good	174	300K ESALs	58	58
Fair	52	1M ESALs	85	84
Poor	16	3M ESALs	81	79
		10/30M ESALs	47	47
Total	348	total	280	268

This Phase III effort compares the INDEX Model and VIRTUAL Model film thickness values of the selected 40-mixture sample to the normal range of film thickness values of the 268-mixture population. The steps taken in this analysis included the following:

1. Determine the descriptive statistics of the population.
2. Determine the descriptive statistics of the four traffic subsets of the population.
3. Compute the INDEX and VIRTUAL film thickness values for the 40 mix designs.
4. Determine the descriptive statistics of the 40-mix sample.
5. Determine the descriptive statistics of the four traffic subsets of the sample.
6. Compare the population and sample statistics.
7. Draw conclusions regarding the impact of the INDEX and VIRTUAL models.

The film thickness values of the population set are representative of the range of film thickness commonly achieved in Iowa. The database of 268 mixtures includes the film thickness value prescribed by the Iowa DOT specifications. This film thickness value FT(DOT) applies a simplified version of the standard equation. To expand the analysis, the standard film thickness values FT(std) for the population were added. As expected, the FT(DOT) values are always slightly lower than the FT(std) values.

A summary of the statistics of the population is given in Table 2. The mean (10.27 and 10.58 microns) and median (10.10 and 10.43 microns) values are well above the Iowa specification minimum limit of 8.0 microns. Because the median values are slightly lower than the mean, the population has a slight positive skew. The skew is also reflected in the range, 7.7 to 14.8 and 7.9 to 15.3, which is 2.5 microns lower and 4.5 microns higher than the mean. The histogram in Figure 2 visually graphs the distribution of FT(DOT) population. A majority of the values range between 8.5 and 11.5 microns.

Table 2. Film thickness population statistics for Iowa mix designs

	Total population		**300K ESAL sub-group**		**1M ESAL sub-group**		**3M ESAL sub-group**		**10/30M ESAL sub-group**	
Film thickness model >>	**DOT**	**std**	**DOT**	**std**	**DOT**	**std**	**DOT**	**std**	**DOT**	**std**
Mean	10.27	10.58	10.25	10.61	10.12	10.43	10.03	10.33	10.94	11.23
Median	10.10	10.43	10.06	10.37	9.95	10.27	9.98	10.35	10.65	10.87
Standard Dev.	1.27	1.33	1.17	1.27	1.10	1.15	1.17	1.21	1.62	1.69
Skew	0.66	0.63	0.42	0.44	0.28	0.27	0.39	0.35	0.60	0.60
Minimum	7.73	7.91	8.39	8.64	8.24	8.45	7.73	7.91	8.28	8.53
Maximum	14.78	15.31	13.44	14.06	12.67	13.08	12.95	13.36	14.78	15.31
Count	268	268	58	58	84	84	79	79	47	47

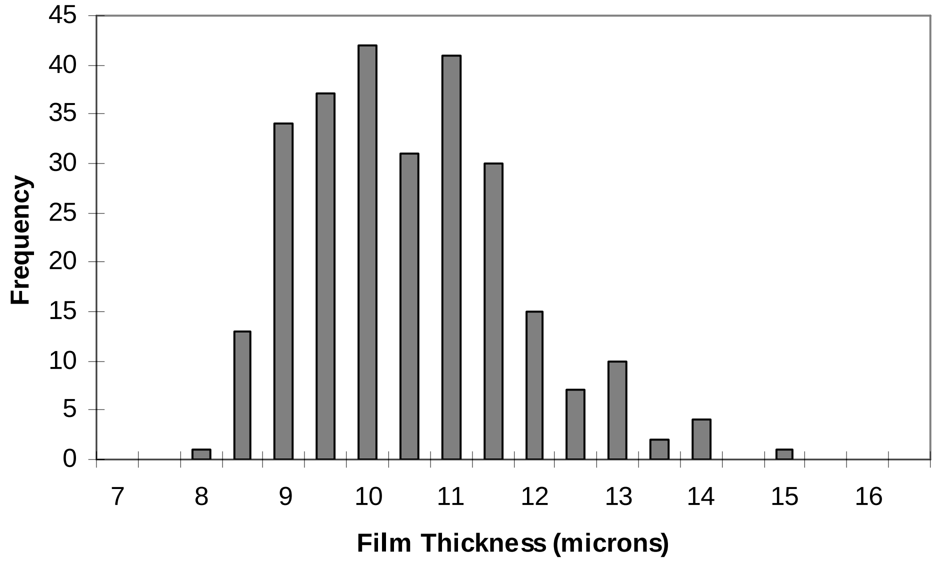


Figure 2. Film thickness population

The population is further divided into the four traffic (ESAL) categories. These categories have different mix design criteria, so it is appropriate to examine the film thickness values of each category. With the same component materials, the proportion of each component will change to satisfy the desired level of mix design criteria. Therefore, the general range of film thickness values should change with each traffic category. Table 2 provides a summary of the statistics for each population sub-group. In general, the shape of the data distribution (slight positive skew) for each sub-group is similar to the entire population. The standard deviation decreases as a result of the tighter range of values, even though the smaller data set could cause the standard deviation to increase. As shown in Figure 3, the trend in the mean film thickness values for the 300K, 1M, and 3M groups reflects the reduction in asphalt binder content associated with higher amounts of compaction effort in the design process. The dramatic increase in the 10/30M group is associated with two mix parameters: cleaner gradations (less fine aggregate) and the need to lubricate the mix during compaction.

The study population uses the Iowa DOT's mix design database, which only provided details of the combined gradation. To generate the INDEX Model and VIRTUAL Model film thickness values, the complete mix design reports were recovered to obtain the details of the gradation for each aggregate source used in the mix design. With the gradation details, the INDEX Model film thickness value FT(INDEX) and VIRTUAL Model film thickness FT(VIRTUAL) were computed for each of the 40 mix designs in the sample database. The sample is plotted in Figure 4 with the FT(std) values as the basis (x-axis value) for comparison. Overall, the trends between the four film thickness values are very similar to trends observed in the generic mix sensitivity analysis in Phase I.

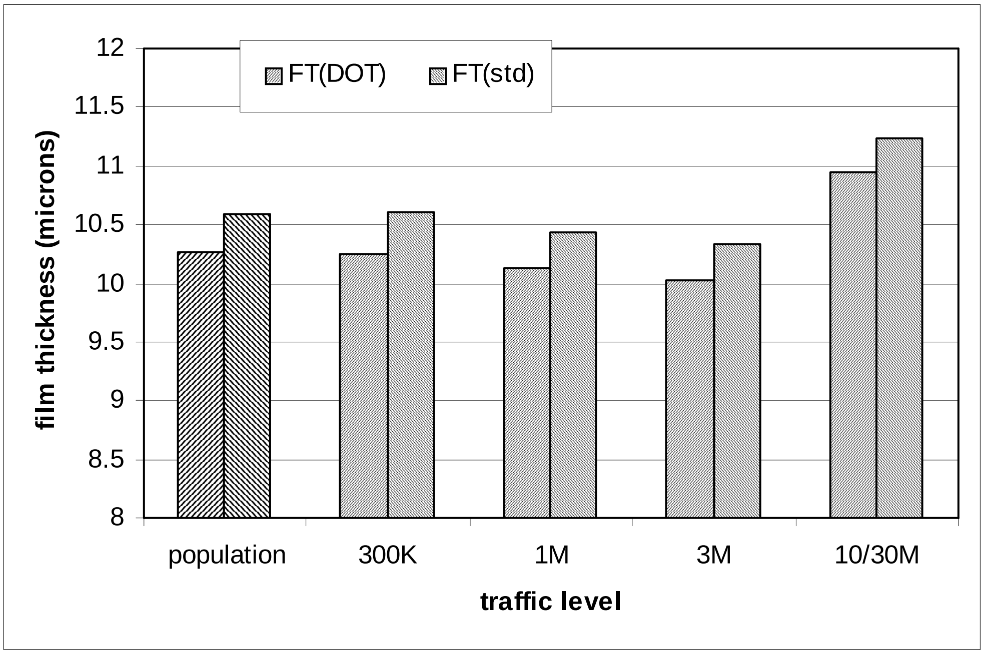


Figure 3. Population and traffic level mean film thickness

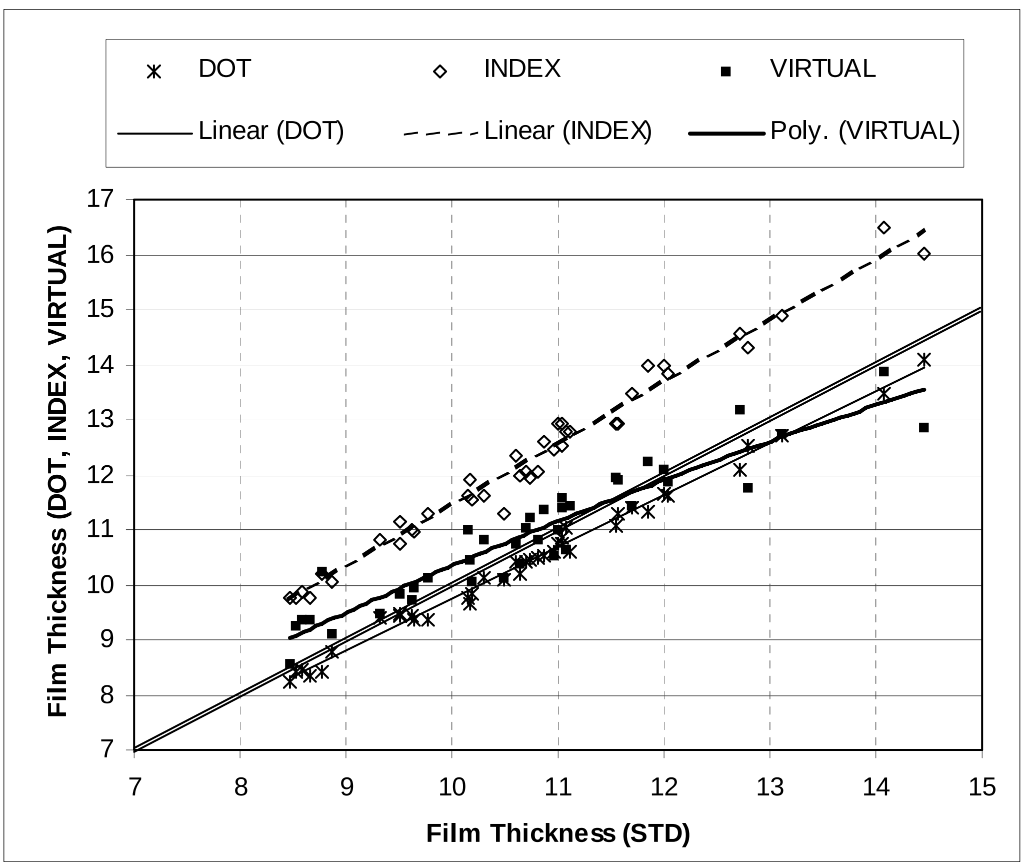


Figure 4. Film thickness comparison for 40-mix sample

Like the 268-mix population, the 40-mix sample was analyzed for general descriptive statistics. The summary of the sample's descriptive statistics is shown in Table 3. The mean values differ for each film thickness model. The FT(DOT) is lower than the FT(std) as expected. The FT(INDEX) mean value is 1.5 microns above the FT(std) as expected. And the FT(VIRTUAL) is between the FT(std) and FT(INDEX), which fits the general trend for the VIRTUAL model. The median values are very close to the mean for three of the four models, but all four sets of data shown some degree of positive skew (0.36 to 0.57). The range of the 40 values is lowest for the VIRTUAL model (5.3 microns) and highest for the INDEX model (6.7 microns). Only the range of film thickness values for the INDEX Model (9.7 to 16.5) would not fit within the Iowa DOT specification limits of 8 to 15 microns.

Table 3. Film thickness 40-mix sample statistics

	FT(DOT)	FT(std)	FT(INDEX)	FT(VIRTUAL)
Mean	10.42	10.72	12.25	10.88
Median	10.45	10.72	12.06	10.82
Standard Deviation	1.37	1.45	1.66	1.19
Skew	0.57	0.55	0.55	0.36
Minimum	8.23	8.48	9.75	8.57
Maximum	14.08	14.45	16.48	13.86
Count	40	40	40	40

The 40-mix sample set is further divided into four 10-mix subsets for each of the traffic (ESAL) levels. While it is possible to create a list of descriptive statistics for each subset, the number of values in each set is too small to realistically measure the shape of the distribution. It is appropriate to limit the examination to the mean and median values for general trends between the film thickness models. Table 4 provides a summary of the sample subset means and median values. The median values are lower than or equal to the mean values. The trend between the means of the film thickness equation models shows the FT(DOT) always as the lowest values and the FT(INDEX) as the highest values. The FT(VIRTUAL) values are higher than the FT(std) values. The trend across traffic levels shows the 1M mixes have the lowest film thickness, but the ranking of the FT(INDEX) and FT(VIRTUAL) values varies for each traffic level.

Table 4. Film thickness sample 10-mix subset statistics

	FT(DOT)	FT(std)	FT(INDEX)	FT(VIRTUAL)
	Mean / median	**Mean / median**	**Mean / median**	**Mean / median**
300K ESAL	10.64 / 10.55	11.07 / 10.99	12.81 / 12.69	11.55 / 11.39
1M ESAL	9.84 / 9.74	10.08 / 10.03	11.45 / 11.45	10.43 / 10.46
3M ESAL	10.42 / 10.44	10.68 / 10.69	12.16 / 12.17	10.77 / 10.69
10/30M ESAL	10.80 / 10.52	11.06 / 10.83	12.58 / 12.26	10.77 / 10.77

With this information about the population and sample of film thickness values, can we use the results of the sample to hypothesize the impact of the INDEX Model and VIRTUAL Model on the film thickness of the population of mix designs? The first question that must be answered is whether the sample is a representative subset of the population. The mean values of the FT(DOT) sample and FT(std) sample are slightly higher than the population values. The median values of the sample are almost equal to the sample mean, unlike the population, but the sample shows a similar level of data skew (population ~0.64 and sample ~0.56). The amount of standard deviation is comparable between the population and sample statistics, and the standard deviation is substantially greater than the difference between the means. Using a standard two-tailed t-test for equal means of the FT(std) population and FT(std) sample, the probable variation of the mean (t = 0.53) falls well below the critical variation (t = 1.97) for 95% confidence. From this comparison, we can conclude that the sample is representative of the population. Therefore, the INDEX Model and VIRTUAL Model values from the sample can be viewed as representative of their impact on the population of film thickness values.

How would the INDEX Model and VIRTUAL Model impact Iowa's film thickness values? One approach looks at the differences between the new models and the FT(std) values. This difference can be expressed as FT(x)-FT(std). Figure 5 shows the histogram of the differences for the Iowa DOT, INDEX, and VIRTUAL values against the FT(std) values. As expected, the FT(DOT) values are very tightly grouped and slightly lower than the FT(std) values, generally in the range of –0.6 to 0.0 microns. The FT(INDEX) values are also tightly grouped and positively skewed in the general range of 1.2 to 2.0 microns. The FT(VIRTUAL) values show a much broader distribution with typical values between –0.6 and +1.0 microns. These FT(VIRTUAL) differences demonstrate the power of the VIRTUAL Model to reflect the uniqueness of each mix design better than the standard two-dimensional approach currently used.

Another approach to examine the impact starts with the mean values of the sample subsets. The trend of mean values in the sample subsets is not the same as the trend in the population subsets, but the range of mean values for FT(DOT) and FT (std) are generally the same (10 to 11 microns). Since the number of values in the sample subset is only 10, the standard deviation values are not strong indicators. To establish a reasonable range for the sample subset, the standard deviations from the total sample are used. Figure 6 shows the nominal range of all four film thickness models based on the sample subset means plus/minus twice the sample standard deviation. Superimposed over the shaded box representing the specification range (8 to 15 microns), the INDEX Model would require an increase in the upper limit of the specification range for three of the four traffic levels. The VIRTUAL Model ranges are within the current specification range and are generally between 8.5 and 13 microns in three of the four levels. Taking a conservative approach, a specification range of 8 to 14 microns would be sufficient to cover the expected range of VIRTUAL Model film thickness values based on the sample database. The range for the INDEX Model would be 8 to 16 microns.

A similar approach compares the population of FT(DOT) and FT(std) values to the sample FT(INDEX) and FT(VIRTUAL) values. The analysis already demonstrated that the sample database is statistically similar to the population. Since the Iowa DOT specification range for film thickness does not distinguish between traffic levels, the analysis based on the range of each model is a reasonable approach. In addition, the statistics are stronger for the sample (40 data points) than for the sample subsets (10 data points each). Figure 7 shows the range of each model based on the mean value plus/minus two standard deviations. The background shading represents the current film thickness specification limits. The figure shows that the current range of film thickness values for the Iowa DOT and standard models generally falls between 8 and 13 microns. The INDEX Model ranges between 9 and 15.5 microns. The VIRTUAL Model is a slightly narrower range from 8.5 to 13.5 microns.

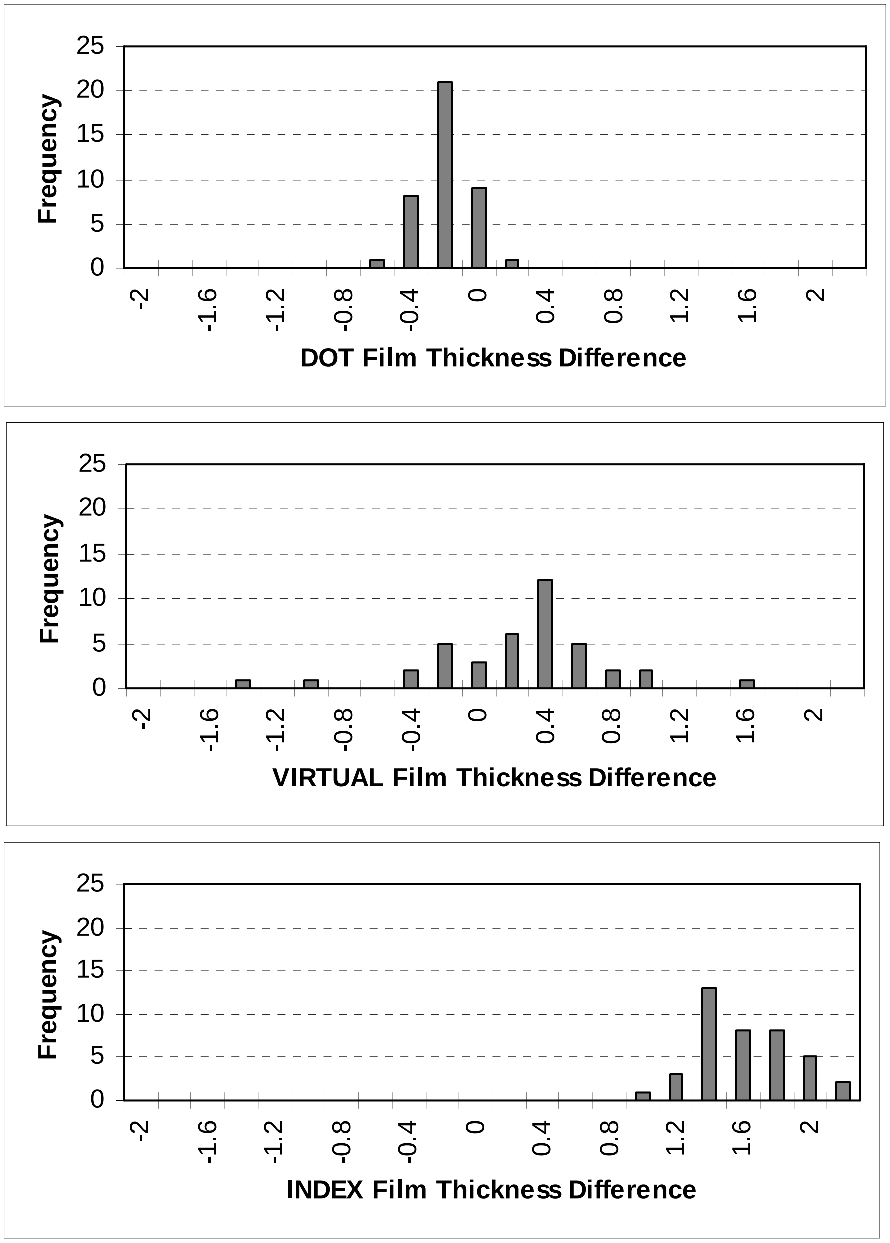


Figure 5. Film thickness change

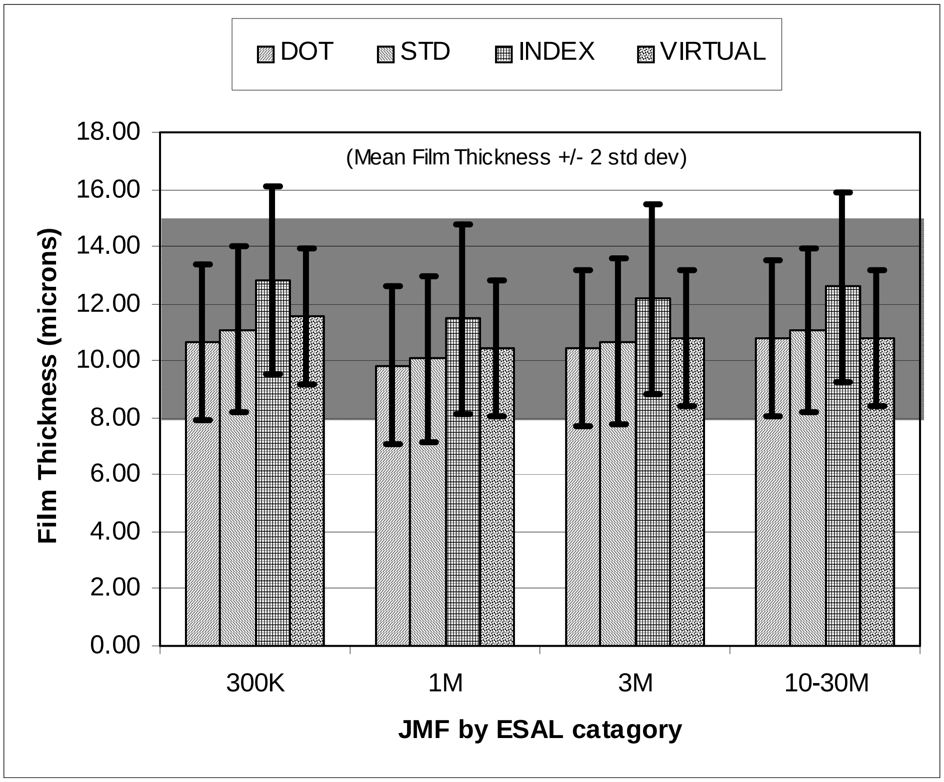


Figure 6. Statistical range of film thickness for sample traffic level subgroups

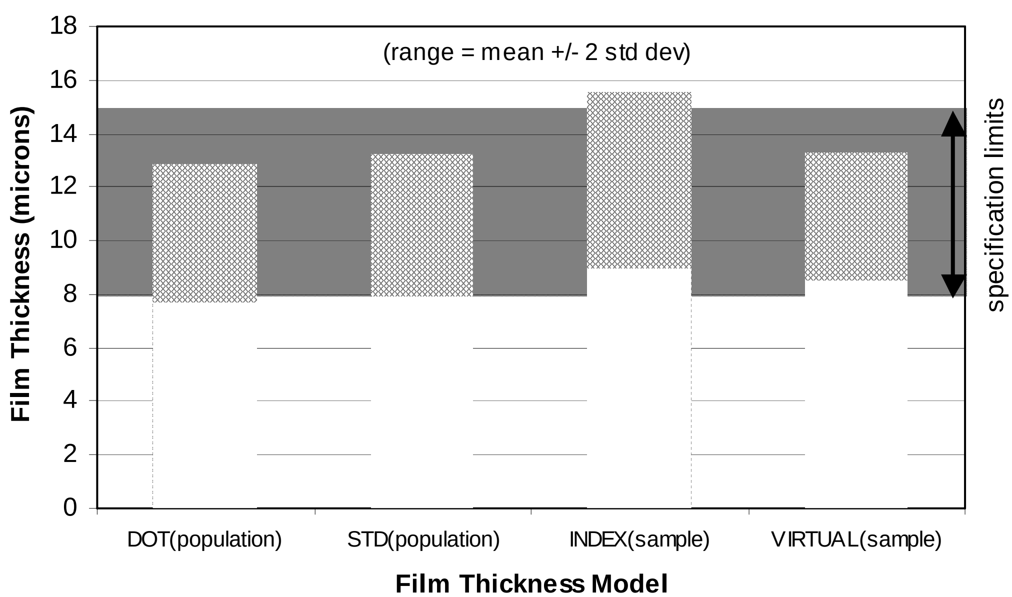


Figure 7. Film thickness range on population basis

As the plotted sample data in Figure 4 shows, we would expect the INDEX range to be higher and broader than the Iowa DOT and standard model. The linear trend is 1.5 microns higher at lower film thickness values and 2.0 microns higher near the upper film thickness range. The extent of the INDEX range is reflected in the higher standard deviation (1.66) and the increased slope of the linear trend. The shape of the VIRTUAL trend in Figure 4 is not linear. Similar to the sensitivity study in Phase I, the VIRTUAL Model computes film thickness values above the standard model at lower film thickness levels but tends to flatten and generate lower values at the higher film thickness level. The standard deviation for the VIRTUAL Model sample (1.19) is smaller than the standard model and is reflected in the flatter slope of the VIRTUAL second-order polynomial trend.

CONCLUSIONS

The third phase of the study examined the impact of the new film thickness equations on field mix designs to determine if the range of film thickness specification criteria would change. The study started with a population of good or excellent mix designs. The distribution of film thickness values for the population was compared to a randomly selected sample set of 40 mixtures. Film thickness values using the INDEX Model and VIRTUAL Model were computed for the 40 sample mix designs.

The trends for the 40-mix sample set between the film thickness values of all four models are very similar to trends observed in the generic mix sensitivity analysis in Phase I.

The standard model film thickness sample data set was determined to be statistically representative. Therefore the INDEX Model and VIRTUAL Model values from the sample can be viewed as representative of their impact on the population of film thickness values. The film thickness values of the INDEX model are tightly grouped and higher than the standard model, and the VIRTUAL values are much broader distributed above and below the standard model. Two other approaches compared the current specification limits to the statistical range of the new film thickness models. The INDEX Model would require an increase in the upper limit of the specification range. The VIRTUAL Model range is generally narrower than the current specification range. The current range of film thickness values for the standard model generally falls between 8 and 13 microns. The INDEX Model ranges between 9 and 15.5 microns. The VIRTUAL Model is a slightly narrower range from 8.5 to 13.5 microns.

The VIRTUAL Model reflects the uniqueness of each mix design better than the standard two-dimensional approach currently used. A specification range of 8 to 14 microns would be sufficient to cover the expected range of VIRTUAL Model film thickness values.

FUTURE STUDY

Durability is a key component of HMA performance but does not receive sufficient research effort to better define differences between good and poor mixtures. Meaningful durability studies must isolate the rate of binder aging (durability). This will require careful mixture preparation, short-term and long-tern aging, nondestructive mixture testing, careful asphalt binder extraction and recovery, and performance grade binder testing. Laboratory studies should include multiple binder contents for each mix to best measure the value of film thickness as an indicator of mixture durability.

Film thickness is a key component of mixture durability. New durability studies should apply the INDEX Model and VIRTUAL Model to better account for differences between mixtures.

The INDEX Model and VIRTUAL Model should be validated by laboratory studies. The sensitivity study described in the model development phase should be replicated with a series of real mixtures. A better understanding of the gradation of mineral filler would improve the computed film thickness value and better distinguish between coated particles and binder extender. The shape factors applied to the INDEX Model would be better quantified using the Aggregate Imaging System.

It is possible that new durability studies of multiple mixtures will still conclude that film thickness alone does not adequately quantify mixture durability. Previous studies have hypothesized that mixture durability relative to coarse aggregate is different from mixture durability relative to fine aggregate. The VIRTUAL Model suggests that there are relationships between the fine aggregate gradation and the binder content. New studies should take an in-depth look at defining the mastic component of a mixture (asphalt binder and a portion of the fine aggregate). How does mixture durability relate to the coarse aggregate fraction? To the fine aggregate fraction? And to the mastic fraction? Does one of these fractions dominate durability, or does each play a role with different criteria?

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Cold In-Place Recycling Forensic Study on U.S. Highway 34 Union County, Iowa

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ABSTRACT

Forensic studies of pavement failures require a clear understanding of the distress observed, a logical approach to examining potential assignable causes, a reasonable connection between the distress and assignable causes, and recommendations to minimize the potential for future failures. The recommendations must be careful to address the cause and to not unduly restrict technology that has a proven record of success. This case study on cold in-place recycling demonstrates this process.

Cold in-place recycling is a successful rehabilitation strategy. As we learn more about this technology, we also learn about its limitations. The dramatic failure of the cold in-place recycled layer under traffic on the U.S. Highway 34 project was investigated. Eleven separate factors were analyzed as possible assignable causes. It was concluded that the level of truck traffic was the primary cause of the failure. The application of cold in-place recycling technology is currently restricted to less than 400 trucks per day until changes are made to the mix design to address higher truck traffic levels.

Key words: cold in-place recycling—rutting

INTRODUCTION

Cold In-place Recycling (CIR) is one of several rehabilitation strategies for hot mix asphalt (HMA) pavements. The criterion for applying CIR includes having a thick HMA pavement that is structurally sound. The primary benefit of CIR is the ability of the recycled layer to delay reflective cracking in the HMA overlay placed over the CIR. The CIR process involves milling the top three to four inches of the HMA pavement, crushing the milled material to a specified size, adding asphalt stabilizing agent to the processed reclaimed asphalt pavement (RAP), placing the CIR mix back onto the pavement, and compacting the CIR layer. Once the layer is compacted and allowed to gain strength, a HMA overlay is placed as a new wearing surface.

On Iowa Department of Transportation (Iowa DOT) projects (and in many states), the CIR layer is used as the driving surface during the period of time between the CIR rehabilitation and the HMA overlay. This part of the paving project process provides two benefits. First, the level of traffic on many Iowa DOT routes would not permit lengthy detours for periods of a week or more. Second, past experience with CIR has demonstrated that opening the CIR layer to traffic generally improves the compaction, particularly in the wheel paths. As the Iowa DOT gains more experience with the proper application of CIR, this rehabilitation strategy is considered to be a viable option on more HMA projects.

On occasion, a phase of a construction project fails. Projects that use CIR are no exception to this rule. The surface of the CIR layer on early projects sometimes ravels under traffic. This has been attributed to a lack of sufficient asphalt stabilizing agent in the CIR to bind the particles together. Other projects have shown distress when the HMA overlay was placed too soon. The CIR layer was not able to support the heavy construction traffic. With each failure, we better understand the key components of the CIR rehabilitation process. The application of CIR on the U.S. Highway 34 (US-34) Union County project added another key component to our understanding of this rehabilitation strategy.

PROJECT DESCRIPTION

The design for the US-34 project included CIR for the existing HMA surface of the composite pavement (HMA over concrete) and placement of a three-inch HMA overlay. There were no unique project features. The project called for 14 lane miles of CIR. As part of the CIR specification, existing HMA was removed by the contractor and sent to the Iowa DOT Central Laboratory for development of the CIR mix design. The mix design process showed that 2.0% foamed asphalt should be added as part of the CIR mixture. This amount was adjusted down to 1.8% as the CIR process started. The mix design decision on the amount of asphalt stabilization is primarily based on the measured wet strength of the mixtures prepared and cured in the lab.

The construction staging for the CIR and HMA overlay was very typical for Iowa DOT rehabilitation projects. Both the CIR and HMA overlay are constructed under traffic. The two-lane section is closed in one direction for an allowable length (generally up to two miles per day), and both directions of traffic are controlled by a pilot car operation. All lanes are open to traffic at night. The CIR subcontractor processes one direction, reaches the end of the CIR limit, then processes the other direction. In this case, the CIR subcontractor started two miles from the west end of the project, proceeded east to the end of the project, turned and continued processing west to the end of the project, and turned east again to finish the project. Due to the curing criteria for CIR, the HMA paving contractor does not bring the HMA plant in until the CIR subcontractor completes his/her rehabilitation of the existing HMA surface.

The CIR process started on May 16 and took 19 calendar days to complete. After each day of CIR, the lanes were opened to traffic. No problems were noted by the construction inspection staff regarding the stability of the CIR layer under traffic.

In a typical CIR process, the curing period requires four to seven days of moderately hot temperatures and no rain events. It is common for the HMA contractor to schedule the HMA overlay two weeks after the CIR is placed. For the US-34 project, the HMA paving crew began placing an HMA shoulder widening unit about one week after the CIR was completed.

FORENSIC STUDY

As early as four days after completing a two-mile length of CIR, severe loss of stability began occurring in isolated 100- to 200-foot sections of the CIR layer under traffic. As the problem increased across the project, an investigation was initiated to determine the cause of the problem. The loss of stability was a serious problem that impacted the progress of the project and, more importantly, the safety of the traffic. The field inspection staff described the problem as a dramatic loss of strength over a two-hour period that resulted in deep wheel path rutting and shoving of the CIR material. To maintain a safe driving surface, motorgraders were used to blade the rutted and heaved CIR material onto the shoulder.

The balance of this paper discusses the field investigation actions applied to examine the CIR failure and determine the cause of the acute rutting and shoving. The first phase of an investigation is to obtain a clear understanding of the condition observed. Once the material distress is properly defined, the investigation looks for assignable causes. Finally, the investigation must determine the changes needed to eliminate or reduce the causes. For example, HMA rutting is an observed distress condition. Rutting can be caused by poor compaction of the mixture. The compaction process (rolling pattern) may be one of the activities that should be changed to achieve higher density. In this example, the observed distress (rutting) may have several assignable causes. If there is no clear connection between the cause and the distress, other causes must be explored.

On the US-34 project, the observed problem was severe rutting and shoving of the CIR layer under traffic. Pictures of the CIR surface and interviews with the field inspection staff confirmed that the observed problem was rutting and shoving. The difficulty with the investigation was that the rutting and shoving was not consistently observed over the project length. With this background, the investigation needed to determine what cause or combination of causes led to the sporadic problem.

A number of causes are associated with CIR layer loss of stability. Since CIR mixture is similar to HMA, the causes of HMA rutting became the baseline for the investigation. Each of the potential causes was listed, and the design and construction details associated with each cause were examined. The list of potential causes for CIR layer loss of stability is as follows, each of which are explained below:

- Moisture
- Compaction
- Temperature
- Changes in CIR aggregate (RAP age)
- Changes in CIR process (RAP sizing)
- Asphalt stabilizer content
- Asphalt stabilizer type
- Traffic (wheel load)
- Wheel load due to steep grade

- Construction staging (traffic queue)
- Air voids

Moisture

Free moisture is a known problem for CIR layers. The adhesive bonds from the CIR process are susceptible to high pore pressures. CIR mix design focuses on the indirect tensile strength of wet-conditioned specimens. During construction, the cured condition of the CIR layer is *loosely* measured as the moisture content in the layer. CIR layers have failed under traffic combined with heavy rains. However, the failures in the field are typically associated with surface raveling. On the US-34 project, there were no significant rain events during the CIR process or during the subsequent traffic.

Compaction

Insufficient compaction of the CIR layer reduces the strength of the layer. Traffic placed on the layer will knead and compact the layer further. This additional compaction can create rutting. However, the compaction created by the traffic normally strengthens the CIR material. On the US-34 project, the field compaction complied with the density specification. Even when the sections of CIR with the lowest density are isolated, these sections do not correspond with the sections that experienced rutting.

Temperature

High summer temperatures are the most common cause of rutting in HMA. For CIR layers, high temperatures are desirable to rapidly cure the mixture (reduce the moisture content). On the US-34 project, there appeared to be some correlation between high temperature and the occurrence of the CIR rutting and shoving. However, if high temperature was the primary cause, we would expect to find continuous rutting. The observed rutting is sporadic along the project length.

Age of the CIR

The CIR process mills, sizes, mixes, places, and compacts the existing HMA surface as a continuous in-place recycling operation. The processed RAP is not removed from the project to a central stockpile to be uniformly blended. It is possible that isolated maintenance overlays were placed in recent years to strengthen the pavement and maintain a smooth ride. If new HMA was processed as part of the CIR, then the use of a soft asphalt stabilizing agent could soften the mixture and create instability in high summer temperatures. Normally, the existing HMA surface material is more than 15 years old and the asphalt binder is aged. On the US-34 project, there was no record or knowledge of recent thin maintenance overlays along the length of the project.

Size of the CIR

Speed of the CIR recycling train is controlled by the speed of the milling unit that tows the remainder of the equipment units. The speed of milling can impact the size of the RAP. Previous CIR studies have not identified major changes in the RAP gradation, but a significant change in the gradation could affect the stability of the CIR layer. Very fine-graded RAP may not have sufficient coarse aggregate to maintain stability under traffic. On the US-34 project, there was no observed changed in the speed of the CIR process.

Asphalt Stabilizer Content

The amount of asphalt in HMA and CIR has similar impacts on each mixture. Dry (low asphalt content) mixes are generally stiff and provide good rutting resistance. Wet (high asphalt content) mixes are softer and more prone to rutting. The CIR process uses material weigh belts to proportion the foamed asphalt and RAP. If there are fluctuations in the operation of the proportioning system, the rutting resistance of the CIR layer will vary. On the US-34 project, cores were taken along the project length to obtain samples of CIR mixture with good and poor performance. The results from laboratory ignition oven extraction testing did not indicate any significant variation in total binder content, and there was no trend between the binder content of sections with good and poor performance. The lab results agreed with the quality control foamed asphalt yield checks computed during the CIR process.

Asphalt Stabilizer Type

The asphalt binder used for CIR is a PG 52 -34. This soft binder is specified for CIR to rejuvenate the aged binder in the existing HMA surface. The larger fraction of light oils in the soft binder will be absorbed by the aged binder. On the US-34 project, a unique situation occurred. About the last 25% of the CIR process was performed with a stiffer PG 58 -28 binder for the foamed asphalt. Although the change in binder does not account for the sporadic occurrence of the rutting, it would provide a clue if there are no distressed locations in the sections processed with the stiffer asphalt binder. Some locations of rutting were observed in the CIR section with PG 58 -28 foamed asphalt binder, so the grade of the asphalt stabilization agent is not a cause of the distress.

Traffic

Typical asphalt pavement rehabilitation projects extend over five to ten miles. Along that project length, it is possible to have changes in traffic frequency and load. Traffic load is a factor in any rutting distress. Significant changes in traffic load can cause one part of a project to perform differently from another. On the US-34 project, the overall traffic frequency and load are heavier than the traffic for a typical CIR project. However, there were no identifiable point sources of load that matched the rutted locations.

Steep Grades

Like traffic frequency and traffic loading, pavement on a steep grade experiences higher stress from the traffic. It is common to observe differences in pavement performance between the incline lanes and the decline lanes. On the US-34 project, the terrain across the project length is moderately rolling. However, the locations of rutting and shoving did not always correspond with incline grades.

Construction Staging

Looking at one more aspect of the load applied on the CIR layer, construction staging is sometimes a factor in early pavement distress. The actual load applied on the pavement surface is a combination of the size of the load and the speed at which it passes. The stress on a pavement surface is greater at slow or stopped conditions, like on the approach to intersections. During the CIR process, the traffic is queued at both ends of the construction length and a pilot car directs the queue of traffic across the open lane. At the point of the queue, the traffic is stopped, which applies a high stress on the pavement. On the US-34 project, the inspector’s field log identifies the start and end locations for each day of the CIR process. However, the locations of the rutting did not coincide with the locations of the traffic queues.

Air Voids

Since many of the *common* causes for CIR rutting distress did not provide a strong explanation for the rutting, the amount of air voids was examined. This potential cause is the opposite of the compaction factor. The compaction factor examined low density mixtures (with high air voids). This factor is related to low air voids (high density mixtures). Measuring the air voids in the CIR layer is an approximation at best. Unlike with HMA, it is difficult to measure the specific gravity of the CIR material because the particles are not completely coated with asphalt. Air voids play a role in some instances of HMA rutting and shoving, particularly on the approaches to intersections. As the traffic slows and the stress increases, the HMA further compacts. The traffic compaction lowers the air void content of the mixture. At very low air void contents, the asphalt binder in the mix begins to fill all the available void space. When this occurs, the surface of the mix begins to flush and the mix stability decreases. On the US-34 project, there were observations of flushing in the wheel paths on short sections of the CIR surface. The description of the severe failures noted rapid displacement rutting and shoving. This would agree with a low air void condition. However, there is no explanation for the sporadic location and timing of the distress.

CONCLUSION

Eleven separate factors were examined to determine the cause of the CIR layer distress. No single, specific factor clearly established a cause for the rutting and shoving. The next step in the investigation looked at the potential impact of a combination of the factors. Of the eleven factors, seven were eliminated from further consideration based on the previous analysis (moisture, changes in CIR age and size, asphalt stabilizer content and size, steep grade, and construction staging). The remaining factors were plotted against the location of the distress. From this analysis, a relative correlation between the individual density measurements and the location of the patches was seen, as shown in Figure 1. The apparent tie to the distress is a function of high CIR construction compaction. Generally, high density due to CIR compaction is not a problem and is encouraged. On this project, however, the high CIR density combined with the heavy truck traffic and higher temperatures pushed the mixture into a low air void condition and created the unstable CIR layer in those sections. Other sections with less CIR compaction density (i.e., higher air voids) were also being further compacted by the heavy truck traffic, but these had not reached the point of low air voids.

Based on this conclusion, the Iowa DOT Office of Materials issued a restriction on the use of CIR. Because there is good performance on many CIR projects, the restriction does not avoid all application of the CIR rehabilitation strategy. The restriction focuses on the key factor on the US-34 project that was unique: heavy truck traffic. The truck traffic on the US-34 project was approximately 600 trucks per day. Previous successful CIR projects with traffic on the CIR layer carried up to 400 trucks per day. The restriction does not recommend the use of CIR on projects that would open the CIR layer to more than 400 trucks per day.

Can CIR rehabilitation technology apply to heavier truck routes in Iowa? A number of states apply CIR technology on heavy truck routes, but their intended CIR mix design and performance criteria are different than Iowa's. To increase the stability of the CIR mix, the process will require a dry stabilizing agent like cement or fly ash. However, making the CIR mixture stiffer will also reduce the value of the mix to retard reflective cracking. Another option may be the use of a polymer modified asphalt emulsion instead of foamed asphalt. Laboratory testing to compare various mix alternatives using a repeated load axial–permanent deformation performance test should be used. Initial test results with this test protocol showed differences between mixtures.

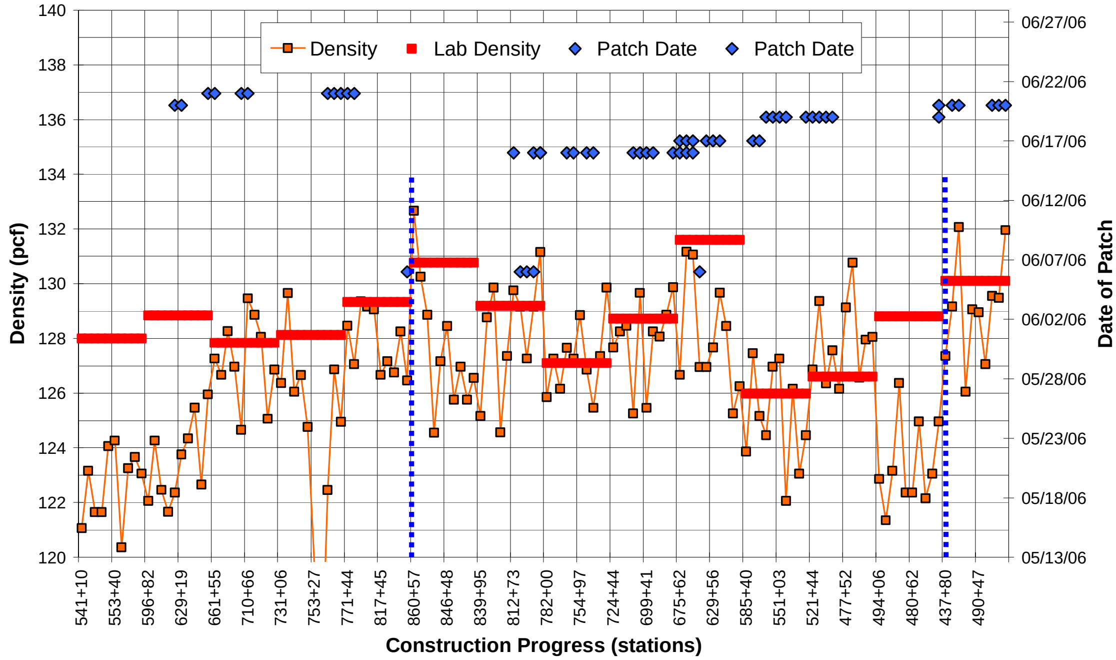

Figure 1. Actual density and location/date of patching

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Safety Effects of Offset Right-Turn Lanes at Rural Expressway Intersections

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ABSTRACT

A rural expressway is a high-speed, multilane, divided highway with partial access control. It is typically divided by a wide, depressed median and consists of both at-grade intersections and grade-separated interchanges. Converting undivided rural two-lane highways into expressways is a popular highway safety improvement, as expressways make passing easier and drastically reduce the likelihood of head-on and opposite-direction sideswipe collisions. However, at-grade intersection collisions on rural expressways are reducing the safety benefits that should be achieved when converting rural two-lane highways into expressways. The underlying problem seems to be that expressway intersections present challenges to minor road drivers attempting to select gaps in the expressway traffic stream. State transportation agencies have experimented with several intersection safety treatments at problematic two-way stop controlled (TWSC) rural expressway intersections to improve their safety performance while avoiding costly grade separation. One of these treatments is the offset right-turn lane. The assumed safety benefit of offset right-turn lanes is that they eliminate the sight distance obstruction created by the presence of right-turning vehicles leaving the expressway, thereby allowing minor road drivers to make better gap selection decisions when entering expressway intersections. However, no studies have been conducted to determine the crash reduction potential of this countermeasure. Therefore, this research examines offset right-turn lane implementation at three TWSC rural expressway intersections and documents their safety performance using naïve before-after crash data analysis. The results show that offset right-turn lanes can be effective in reducing the frequency of near-side right-angle collisions occurring at TWSC rural expressway intersections.

Key words: expressway intersection safety—median intersection design—offset right-turn lanes

INTRODUCTION

A rural expressway is a high-speed (50 mph), multilane, divided highway with partial access control. It is typically divided by a wide, depressed turf median and may have intersections that are at-grade or grade-separated. Converting undivided rural two-lane highways into expressways is a popular highway safety improvement used by many state transportation agencies (STAs) because, by providing an extra lane of travel in each direction and a physical separation between opposing traffic flows, expressways make passing easier and drastically reduce the likelihood of dangerous head-on and opposite-direction sideswipe collisions (AASHTO 2004). In addition, these facilities improve the connectivity between cities while promoting economic growth. Overall, the assumption is that expressways are able to provide most of the mobility, capacity, and safety benefits of an interstate, while being constructed at a lower cost (Maze, Hawkins, and Burchett 2004). The popularity of expressway conversion is evidenced by the fact that rural expressway mileage in the U.S. increased by more than 2,600 miles between 1996 and 2002. This expansion is expected to continue, as 26 out of 28 STAs recently surveyed indicated that they plan to expand their state expressway systems over the next ten years (Maze, Hawkins, and Burchett 2004).

The typical rural expressway at-grade intersection, as shown in Figure 1, is a two-way stop controlled (TWSC) intersection, with the stop control on the minor, usually two-lane, roadway. A number of studies have shown that right-angle collisions account for the majority of crashes at these intersections (Maze, Hawkins, and Burchett 2004; Harwood et al. 1995; NDOR 2000; Preston et al. 2004). In addition, an unpublished study conducted by the Nebraska Department of Roads (NDOR) Highway Safety Division (NDOR 2000) revealed that right-angle intersection collisions on their rural expressway system are nullifying the safety benefits that should be derived from converting a number of their rural two-lane highways into expressways. In an effort to develop a better understanding of the causes of these right-angle collisions, Preston et al. (2004) reviewed three years (2000–2002) of crash data at three high–crash frequency TWSC rural expressway intersections in Minnesota and discovered that 87% of the right-angle collisions at these intersections were due to the inability of minor road drivers to recognize oncoming expressway traffic and/or select safe gaps in the expressway traffic stream (i.e., minor road drivers did not see or misjudged the time-to-arrival of an approaching expressway vehicle). Other intersection design features (horizontal/vertical curvature on the expressway, intersection skew, median width, presence of right-turn lanes on the expressway, etc.) may make the task of gap selection more difficult for the minor road driver.

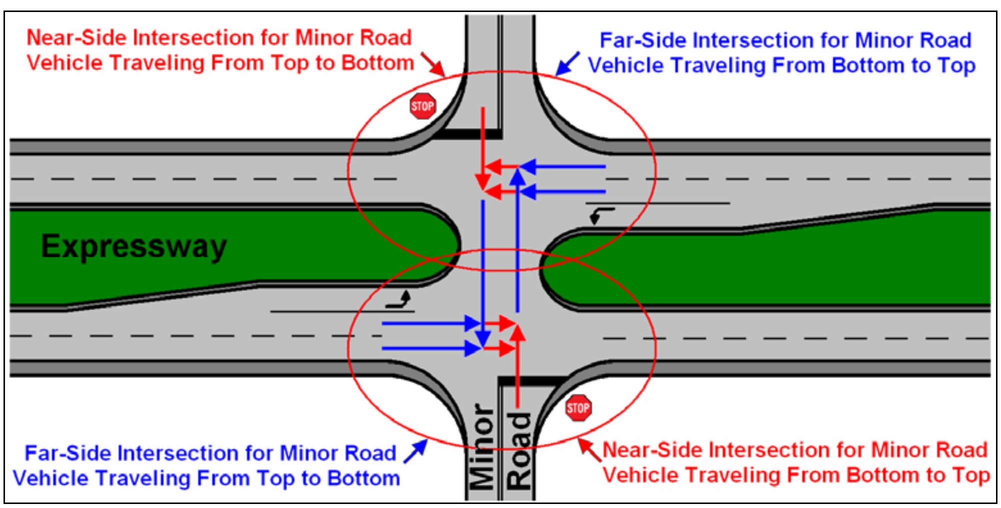

Figure 1. Typical rural expressway at-grade intersection

PROBLEM STATEMENT

The purpose of providing exclusive right-turn lanes on expressway intersection approaches is to remove the deceleration and storage of right-turning vehicles from the through traffic lanes, thereby enabling through traffic to pass by with little conflict or delay and improving the overall safety and capacity of the intersection (AASHTO 2004). It is generally thought that the presence of exclusive right-turn lanes on the divided highway contributes to intersection safety by reducing speed differentials in the through lanes, consequently diminishing the potential for rear-end collisions, particularly on high-speed, high-volume approaches where right-turn volumes are substantial. However, the limited research assessing the safety effects of providing right-turn lanes at rural expressway/divided highway intersections revealed that conventional right-turn lanes may actually increase crashes (Maze, Hawkins, and Burchett 2004; Van Maren 1980).

A crash model developed by Van Maren (1980) for 39 randomly selected multilane divided highway intersections in rural Indiana showed that intersection crash rates increased with the presence of a right-turn deceleration lane on the divided highway. In a more recent study, a rural expressway intersection safety performance function developed by Maze, Hawkins, and Burchett (2004) using 644 TWSC expressway intersections in rural Iowa revealed a similar trend. Although this result was statistically significant, the authors speculated that the higher crash rates at locations with right-turn lanes was not due directly to their presence, but was instead due to the fact that right-turn lanes had been installed at high crash locations. However, another explanation of these findings might be that vehicles using a conventional right-turn lane to exit the expressway are obstructing the adjacent minor road driver's view of oncoming expressway traffic, as shown in Figure 2. This can lead to an increase in collisions involving vehicles turning left, turning right, or crossing from the minor road, thus creating a more dangerous intersection environment.

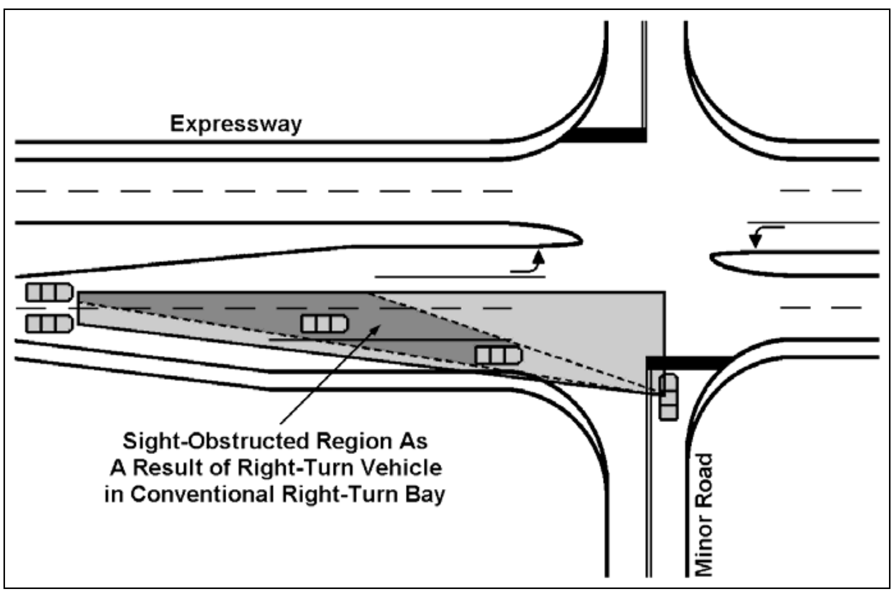

Figure 2. Sight obstruction created with conventional right-turn lane

The offset right-turn lane design alternative shown in Figure 3 helps to alleviate the sight distance obstruction created by the presence of right-turning vehicles in a conventional right-turn lane. In this design, the right-turn lane is moved laterally to the right as far as necessary so that right-turning vehicles no longer obstruct the view of minor road drivers positioned at the adjacent stop bar. Offset right-turn lanes should improve rural expressway intersection safety by enhancing intersection sight distance and making it easier for minor road drivers to select safe gaps in the near-side expressway traffic stream. As such, they are expected to reduce near-side right-angle collisions between vehicles turning or crossing from the minor road and through vehicles on the divided highway; however, no research has been conducted to determine the safety benefits of applying this strategy at rural expressway intersections (Neuman et al. 2003).

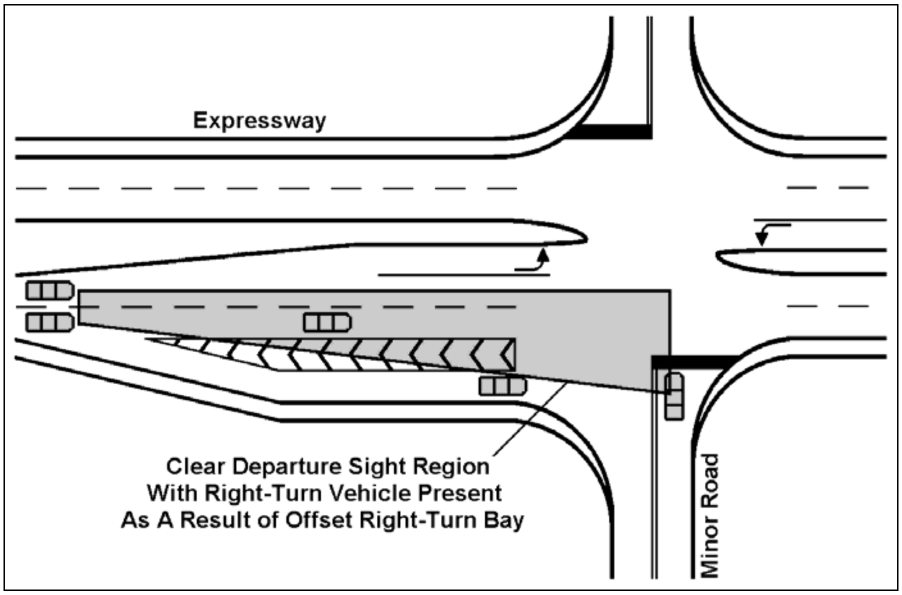

Figure 3. Offset right-turn lane design concept

RESEARCH OBJECTIVES

STAs have experimented with a wide range of intersection safety treatments at problematic rural expressway intersections to improve their safety performance while avoiding costly grade separation. However, a recent survey of STAs conducted by Maze, Hawkins, and Burchett (2004) revealed that only five of the twenty-eight responding agencies had used offset right-turn lanes as a corrective measure at rural expressway intersections. This is probably due to the fact that no guidance on the use or design of offset right-turn lanes is provided in the AASHTO Green Book (2004). In addition, no studies have been conducted to determine the crash reduction potential of this strategy. Therefore, the objective of this research is to investigate the safety effectiveness of the offset right-turn lane design alternative. Examples of offset right-turn lane implementation were found in Iowa and Nebraska, and these case studies are presented herein.

BEFORE-AFTER CRASH DATA ANALYSIS

US-61 and Hershey Road, Muscatine, IA

Two examples of offset right-turn lane installations on rural expressways were found in Iowa. The first example is located at the intersection of US-61 and Hershey Road near the western edge of Muscatine, IA. An aerial photo of this intersection is shown in Figure 4. US-61 through this area was originally built to expressway standards in 1984. The construction of the US-61/Hershey Road intersection that resulted from this project did not provide any right-turn lanes for vehicles exiting US-61. The intersection remained this way until July of 2003, when offset right-turn lanes were installed on both the northbound and southbound US-61 approaches. Photographs of each offset right-turn lane are shown in Figure 5.

Subsequently, the intersection was signalized in November of 2005 and remains that way today. However, this intersection has been a consistent safety problem and is likely to be converted to an interchange sometime in the near future.

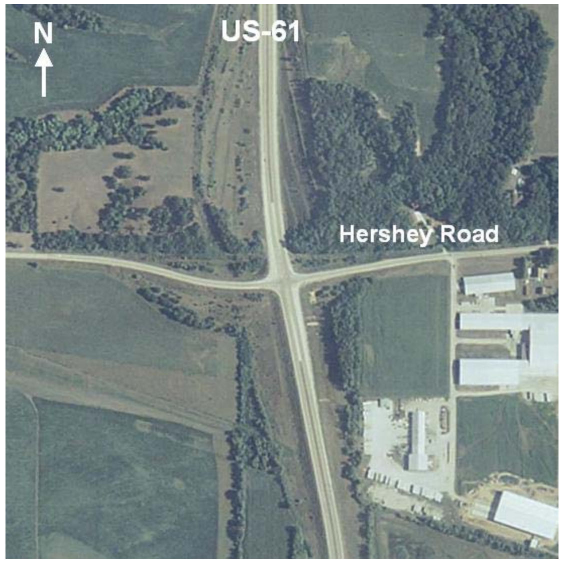

Figure 4. Intersection of US-61 and Hershey Road, Muscatine, IA

Figure 5. Offset right-turn lanes installed at US-61 and Hershey Road, Muscatine, IA

Before-after crash data for this offset right-turn lane implementation was obtained from the Iowa Traffic Safety Data Service (ITSDS) and is shown in Table 1. In the 3 1/2 year before period (January 1, 2000 through June 30, 2003), the intersection experienced a total of 15 intersection-related crashes (1 fatal, 10 injury, and 4 property damage-only [PDO]), giving an average crash frequency of 4.3 crashes per year. In the 2 1/4 year after period (August 1, 2003 through October 31, 2005), there were a total of 11 intersection-related crashes (2 fatal, 5 injury, and 4 PDO), resulting in an average of 4.9 crashes per year; an overall increase of 14%. However, in order to gauge the true effectiveness of the offset right-turn lane installation, a closer examination of the crash types targeted by the improvement, namely near-side right-angle collisions, is necessary (Neuman et al. 2003).

Table 1. Before-after crash data for offset right-turn lanes at US-61 and Hershey Road

Data category	Before	After	% Change
Years	3.50	2.25	
Total intersection-related crashes	15	11	
Crash frequency/year	4.29	4.89	+14.1
Fatal	1 (0.29)	2 (0.89)	+211.1
Injury	10 (2.86)	5 (2.22)	-22.2
PDO	4 (1.14)	4 (1.78)	+55.6
Right-angle/broadside	13 (3.71)	9 (4.00)	+7.7
Left turn leaving	2 (0.57)	0 (0.00)	-100
Rear-end	0 (0.00)	2 (0.89)	+Infinite
Near-side right-angle	6 (1.71)	6 (2.67)	+55.6
Far-side right-angle	7 (2.00)	3 (1.33)	-33.3
% Near-side/right-angle	46.15	66.67	+44.4
% Near-side/total crashes	40.00	54.55	+36.4

Note: Values in parenthesis are the average crash frequencies per year and were used to compute the percent change. Statistical analysis was not performed because there were less than three years of after data.

Because offset right-turn lanes are meant to reduce near-side right-angle collisions, and because offset right-turn lanes were installed on both mainline approaches at this location, a before-after comparison of total near-side right-angle collisions was conducted. In the before period, this site averaged 1.71 near-side right-angle collisions per year. In the after period, the site averaged 2.67 near-side right-angle collisions per year, an increase of approximately 56%. In both the before and after periods, this site averaged approximately four right-angle crashes per year; therefore, it appears that after the installation of the offset right-turn lanes, the distribution of far-side versus near-side crashes switched in favor of near-side crashes, which is an unexpected outcome.

Because there were two offset right-turn lanes installed at this location, one on northbound US-61 and one on southbound US-61, a separate analysis of near-side right-angle collisions was conducted for each offset right-turn lane. The results of this analysis are shown in Table 2. Table 2 shows that neither offset right-turn lane was effective in reducing the frequency of near-side right-angle collisions at this location. In both the before and after periods, five of the six near-side right-angle crashes involved southbound traffic on US-61 colliding with eastbound traffic on Hershey Road. This distribution can possibly be explained by the fact that southbound traffic on US-61 is rounding a horizontal curve and coming down a relatively steep grade as it approaches Hershey Road. These alignment issues could be causing eastbound drivers on Hershey Road to have problems seeing and/or judging the speed of southbound traffic on US-61, regardless of the presence of the offset right-turn lane. The view of an eastbound driver on Hershey Road can be seen in the top portion of Figure 5. These alignment issues may explain why the southbound offset right-turn lane was not beneficial; however, they do not explain why the northbound offset was ineffective.

Table 2. Before-after analysis of individual offset right-turn lanes at US-61 and Hershey Road

Data category	Before	After	% Change
Years	3.50	2.25	
Southbound offset right-turn lane; near-side right-angle (SB and EB traffic)	5 (1.43)	5 (2.22)	+55.6
Northbound offset right-turn lane; near-side right-angle (NB and WB traffic)	1 (0.29)	1 (0.44)	+55.6

Note: Values in parenthesis are the average crash frequencies per year and were used to compute the percent change. Statistical analysis was not performed because there were less than three years of after data.

Another issue at this intersection which may explain why both the northbound and southbound offset rights were ineffective is that the median width is very narrow (14–16 ft.). This geometry does not allow a minor road passenger car to be stored fully within the median; therefore, minor road drivers are forced to make a one-stage crossing or left-turn maneuver. As a result, the crossing/left-turning task for the minor road driver becomes increasingly complex, as they must simultaneously search for an acceptable gap in expressway traffic coming from both the left and the right.

West Junction of US-18 and US-218, Floyd, IA

The second example of an offset right-turn lane installation on a rural expressway in Iowa was found at the west junction of US-18 and US-218, just to the south of Floyd, IA. An aerial photo of this intersection is shown in Figure 6. US-18 was originally built to expressway standards sometime during the 1990s. The construction of the US-18/US-218 intersection that resulted from this project included a conventional right-turn lane for northwest-bound traffic on US-18 turning right onto US-218 toward Floyd. The intersection remained this way until late September of 2003, when Iowa Department of Transportation District 2 converted this conventional right-turn lane into an offset right-turn lane, as shown in Figure 7.

Figure 6. West junction of US-18 and US-218, Floyd, IA

Figure 7. Offset right-turn lane at west junction of US-18 and US-218, Floyd, IA

The offset right-turn lane at this intersection was installed due to a heavy volume of truck traffic exiting US-18 to access the truck stop located in the north quadrant of this intersection. The offset right-turn lane was constructed with district maintenance funds, and the intent was to keep the cost of the improvement to a minimum; therefore, the offset right-turn lane was designed as a normal parallel right-turn lane that flares out at a 30:1 taper in order to achieve the desired offset. During the design process, a minimum departure sight triangle was determined and used to decide how much the right-turn lane needed to be offset. However, during pavement marking, a decision was made in the field to extend the 2 ft. paved shoulder on the mainline throughout the offset right-turn lane. As a result, the outer edge of the gore area was painted 12 ft. from the striped right-turn lane edge-line, and the offset distance was reduced in size from what the designers had initially intended. As these markings wore off over time, the district attempted to increase the size of the offset (gore area) by positioning the right-turn lane closer to the edge of pavement. David Little, Assistant District 2 Engineer, stated, “The offset seems to have been an improvement, but the overall consensus is that the right-turn lane is still not offset far enough.” Therefore, District 2 is currently working on a project that will offset this right-turn lane by 3 or 4 more ft. In conjunction with this project, the district plans to place rumble strips within the gore area to encourage right-turning drivers to use the full offset. Another means of increasing the offset at this location may also include moving the stop bar, stop sign, and divisional island on southwest bound US-218 closer to the mainline. Currently, they are positioned too far back (as shown in Figure 8), and as a result minor road drivers stopped at the stop bar do not get the full sight distance advantage provided by the offset right-turn lane.

Before-after crash data for this offset right-turn lane conversion was obtained from ITSDS and is shown in Table 3. In the 3 3/4 year before period (January 1, 2000 through September 24, 2003), the intersection experienced a total of 10 intersection-related crashes (6 injury, 4 PDO), giving an average crash frequency of 2.7 crashes per year. In the 2 1/4 year after period (October 15, 2003 through December 31, 2005), there were a total of 6 intersection-related crashes (all injury), resulting in an equivalent crash frequency of 2.7 crashes per year. Therefore, on the surface, it appears that the offset right-turn lane installation at this location made no difference at all. However, an examination of near-side right-angle collisions (the crash type targeted by the improvement) shows much more positive results.

Figure 8. Stop bar location on southwest bound US-218

Table 3. Before-after crash data for offset right-turn lane at west junction of US-18 and US-218

Data category	Before	After	% Change
Years	3.73	2.21	
Total intersection-related crashes	10	6	
Crash frequency/year	2.68	2.71	+1.14
Fatal	0 (0.00)	0 (0.00)	0
Injury	6 (1.61)	6 (2.71)	+68.6
PDO	4 (1.07)	0 (0.00)	-100
Right-angle/broadside	8 (2.14)	2 (0.90)	-57.9
Left-turn leaving	1 (0.27)	0 (0.00)	-100
Rear-end	1 (0.27)	0 (0.00)	-100
Right-turn leaving	0 (0.00)	3 (1.36)	+Infinite
Sideswipe (same direction)	0 (0.00)	1 (0.45)	+Infinite
Near-side right-angle	6 (1.61)	2 (0.90)	-43.8
Far-side right-angle	2 (0.54)	0 (0.00)	-100
% Near-side/right-angle	75.00	100.00	+33.3
% Near-side/total crashes	60.00	33.33	-44.4

Note: Values in parenthesis are the average crash frequencies per year and were used to compute the percent change. Statistical analysis was not performed because there were less than three years of after data.

In the before period, this intersection experienced a total of 8 right-angle collisions. Of these 8, 6 were near-side collisions involving vehicles on southwest-bound US-218 colliding with vehicles on northwest-bound US-18 (the approach where the offset right-turn lane was eventually installed), giving a “preventable” near-side right-angle crash frequency of 1.61 crashes per year. In the after period, only two near-side right-angle crashes occurred that involved vehicles on southwest US-218 and northwest US-18, giving a near-side right-angle crash frequency of 0.90 per year and an overall near-side right-angle crash reduction of approximately 44%. Therefore, according to this naïve before-after crash data comparison, it appears that the offset right-turn lane installation at this location has been a safety improvement in terms of reducing near-side right-angle collisions. However, it is interesting to note that in the after period, there were three “right-turn leaving” crashes involving a right-turning vehicle on northwest US-18 that used the offset right-turn lane, turned at a high rate of speed, lost control, slid through the intersection, and collided with a vehicle on southwest US-218 that was stopped at the stop sign waiting to enter the intersection. This unexpected consequence may be an indication that drivers are interpreting the tapered offset right-turn lane design used at this location as a high-speed right-turn exit ramp, which is consequently encouraging drivers to make the right-turn at a higher rate of speed than is safe for the conditions. Some possible fixes to correct this problem include (1) paving the shoulder adjacent to the offset right-turn lane to keep excess gravel out of the turning lane, (2) increasing the turning radius for the exiting offset right-turn lane, (3) using a parallel offset right-turn lane design (see Figure 3) as opposed to the tapered type design used by the Iowa DOT (see Figures 5 and 7), and (4) posting an advisory speed plaque with the message “EXIT XX MPH” along the deceleration lane far enough in advance so that the exiting driver can make a safe slowing and turning maneuver.

N-2 and 148th Street, Lincoln, NE

A third example of an offset right-turn lane installation on a rural expressway was found at the intersection of Nebraska Highway 2 (N-2) and 148th Street, located a few miles to the southeast of Lincoln, Nebraska. The conversion of N-2 from a two-lane undivided highway to expressway standards was completed in late 1997. An aerial photo of the N-2/148th Street intersection that resulted from this project is shown in Figure 9. This initial design did not provide any right-turn lanes for traffic exiting N-2.

Figure 9. Intersection of N-2 and 148th Street, near Lincoln, NE

In late 1998, an NDOR traffic engineering study identified the need to install a right-turn lane on westbound N-2 for traffic turning northward onto 148th Street. 148th Street is a two-lane undivided paved county road that essentially functions as a bypass on the east edge of Lincoln, NE. The study indicated that (1) current right-turn traffic volumes at the intersection met NCHRP 279 (Neuman 1985) volume warrants for a full-width right-turn lane, (2) westbound right-turning traffic often used the paved shoulder to complete the turn, (3) a heavy volume of truck traffic was using 148th Street, and (4) although intersection sight distance was adequate, the intersection is placed on a crest vertical curve such that westbound traffic on N-2 does not see the intersection until just over the crest. As a result of these observations, a decision was made to construct an offset right-turn lane. The parallel offset right-turn lane, shown in Figure 10, was constructed and opened to traffic in late June 2003. NDOR personnel estimated that the offset distance is 12 ft. In addition, a divisional (splitter) island was installed on southbound 148th Street, and an additional stop sign was placed there, as shown in the lower portion of Figure 10.

Figure 10. Offset right-turn lane at N-2 and 148th Street, near Lincoln, NE

Crash data for this intersection was obtained from NDOR and is summarized in Table 4. In the 5 1/2 year before period (January 1, 1998 to June 30, 2003), there were a total of three reported PDO crashes that occurred at the intersection. Therefore, the average crash frequency was 0.55 crashes per year (recall that the offset right-turn lane installation at this location was based on a volume warrant, not poor safety performance). In the 2 1/2 year after period (July 1, 2003 to December 31, 2005), there were a total of five intersection-related collisions (1 fatal, 4 PDO), giving an average crash frequency of 2.0 crashes per year. Therefore, the crash frequency at this intersection increased by approximately 267% after the offset right-turn lane was installed; however, a further examination of near-side right-angle crashes shows more positive results.

Table 4. Before-after crash data for offset right-turn lane at N-2 and 148th Street

Data category	Before	After	% Change
Years	5.5	2.5	
Total intersection-related crashes	3	5	
Crash frequency/year	0.55	2.00	+266.7
Fatal	0 (0.00)	1 (0.40)	+Infinite
Injury	0 (0.00)	0 (0.00)	0
PDO	3 (0.55)	4 (1.60)	+193.3
Right-angle/broadside	2 (0.36)	1 (0.40)	+10.0
Rear-end	1 (0.18)	3 (1.20)	+560.0
Other	0 (0.00)	1 (0.40)	+Infinite
Near-side right-angle	1 (0.18)	0 (0.00)	-100.0
Far-side right-angle	1 (0.18)	1 (0.40)	+120.0
% Near-side/right-angle	50.00	0	-100.0
% Near-side/total crashes	33.33	0	-100.0

Note: Values in parenthesis are the average crash frequencies per year and were used to compute the percent change. Statistical analysis was not performed because there were less than three years of after data.

Of the three crashes that occurred during the before period, only one was a near-side right-angle collision involving a vehicle on southbound 148th Street colliding with a westbound vehicle on N-2 (the approach where the offset right-turn lane was eventually installed), giving a near-side right-angle crash frequency of 0.18 crashes per year. It was noted in the crash report that the southbound driver's sight distance was obstructed by an uninvolved right-turning vehicle on N-2; therefore, this collision may have been prevented had the offset right-turn lane been in place at that time. In the after period, even though the overall crash frequency dramatically increased, no near-side right-angle crashes occurred at the intersection, giving a 100% reduction for this crash type. Therefore, it appears that the offset right-turn lane was a safety improvement in terms of preventing near-side right-angle collisions. However, it should be mentioned that the collision classified as "other" in the after period was a single-vehicle, run-off-road, PDO crash under daylight and dry conditions in which a westbound vehicle on N-2 took evasive action to prevent a near-side right-angle collision with a southbound vehicle on 148th Street, which had pulled out in front of the westbound vehicle. It was not stated whether a right-turning vehicle was present at the time of this collision.

CONCLUSIONS

The assumed safety benefit of offset right-turn lanes is that they eliminate the sight distance obstruction created by the presence of right-turning expressway vehicles positioned in a conventional right-turn lane, thereby allowing minor road drivers to make better gap acceptance decisions when entering the near-side intersection. Expressway intersections most likely to benefit from offset right-turn lanes include (1) intersections with a history of near-side right-angle collisions resulting from right-turning expressway vehicles obstructing minor road driver sight lines and (2) intersections with large right-turn volumes (especially trucks) leaving the expressway, in combination with large volumes of minor road and expressway traffic on the corresponding approaches.

Two of the three before-after case studies presented here revealed a reduction in the frequency of near-side right-angle collisions, while the overall crash frequency at each intersection increased. Table 5 summarizes these results. However, the naïve before-after comparison approach used has two major limitations. First, it does not take regression to the mean into account. (I.e., because two of the three sites were high-crash locations, there would likely have been a reduction in crashes during the after period even if nothing had been done due to simple chance). Second, it is not known what part of the change in safety can be attributed to the treatment and what part is due to various other influences, such as changes in traffic volume, vehicle fleet mix, weather, driver behavior, etc. Finally, it must also be stated that offset right-turn lanes are only meant to enhance sight distance and reduce the possibility of near-side collisions when right-turning vehicles are present. By only examining the crash data, there is no way of knowing whether or not right-turning vehicles were present at the time of these collisions. Therefore, a better means of determining the safety effectiveness of the offset right-turn lane treatment may be to conduct an observational before-after conflict analysis.

Table 5. Offset right-turn lane safety effectiveness summary

	% Change		
	US-61 and Hershey Rd.	**US-18 and US-218**	**N-2 and 148th St.**
Total crash frequency	+14 %	+1 %	+267 %
Right-angle crash frequency	+8 %	-58 %	+10 %
Near-side right-angle Crash frequency	+56 %	-44 %	-100 %

Note: Statistical analysis was not performed because there were less than three years of after crash data in each case.

Given the limited number of sites, the fact that there were less than three years of after data at each site, and the limitations of the naïve before-after analysis, the specific results shown in Table 5 may not be transferable to other expressway intersections. However, the lessons learned from these case studies are important for other STAs to take away as they begin to implement this countermeasure.

Offset right-turn lane design guidance should be included in the AASHTO Green Book (2004). The most important design aspect of an offset right-turn lane is that it should provide the minor road driver with a clear departure sight triangle to the left (i.e., sufficient sight distance along the near-side expressway lanes) when right-turning vehicles are present on the mainline. Meeting this design criterion should aid minor road drivers in judging the suitability of available gaps in the near-side expressway traffic stream when making turning or crossing maneuvers. The recommended dimensions for the legs of a clear departure sight triangle are described in Chapter 9 of the AASHTO Green Book (2004). The required right-turn offset distance may vary from intersection to intersection based on each intersection's unique geometry (skew, horizontal curvature, approach grades, design speed, stop bar placement, etc.); therefore, intersection design plans should be checked to ensure that adequate intersection sight distance is provided. In addition, rumble strips may be placed in the gore area to ensure that the offset right-turn lane is used properly, and additional precautions should be taken to prevent “right-turn leaving” collisions.

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Evaluation of a Timber Bridge for the Secondary Road System Using FRP Reinforced Glued-Laminated Girders

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ABSTRACT

One of the goals that the Innovative Bridge Research and Construction (IRBC) program, sponsored by the Federal Highway Administration (FHWA), wishes to achieve through sponsored research projects is the concept of an economical, durable, short-span bridge that reduces construction time. Delaware County, Iowa took a step in that direction through the construction of a 64 ft. long bridge comprised of fiber-reinforced polymer (FRP) reinforced glued-laminated timber girders and a transverse glued-laminated timber deck. Prior funding for the design, construction (including materials), and monitoring/evaluation of this project was obtained through the IBRC program with the assistance of the Iowa Division of the FHWA. Although strengthening of timber bridges with FRP is not a new topic, limited information exists on the short- and long-term effectiveness of using FRP reinforcement for these types of structures. Thus, the objective of this project was to evaluate the in-service structural performance of a glued-laminated timber bridge strengthened with FRP plates. Field load tests and inspections were performed immediately after construction and each year for two years after construction of the bridge. The purpose was to establish a database of information to address both short- and long-term performance using FRP plates to reinforce glued-laminated timber girders. Structurally, the bridge performed within current specified limits, and no significant increase in stiffness was identified in the girders due to the presence of the FRP plates. Test results collected in 2006 show no noticeable difference compared to those collected two years earlier in 2004, and the FRP/timber bond showed no signs of deterioration. Cost still appears to be the limiting factor for this bridge design, although, as this design gains familiarity, that cost may decrease, making it a more attractive alternative.

Key words: FRP—glued-laminated timber—timber-FRP composite

INTRODUCTION

In recent years, the concept of an economical, durable, short-span bridge which reduces construction time has become more of a necessity to the county engineer than a goal. In addition, county engineers are seeking these same qualities in methods to repair and strengthen existing bridges on the secondary road system. This is just one of the goals that the Innovative Bridge Research and Construction (IRBC) program, sponsored by the Federal Highway Administration (FHWA), wishes to achieve through sponsored research projects.

Delaware County, Iowa is taking a step in that direction through the construction of a 64 ft. long bridge comprised of fiber-reinforced polymer (FRP) reinforced glued-laminated timber girders and a transverse glued-laminated timber deck. The bridge was designed by Matt Smith of Laminated Concepts, Inc. of New York state with technical guidance provided in the design and fabrication of the FRP reinforced girders by the University of Maine. The bridge was fabricated by Alamco Wood Products, Inc. of Albert Lea, MN. Prior funding for the design, construction (including materials), and monitoring/evaluation of this project has been obtained through the IBRC program with the assistance of Curtis Monk, Division Bridge Engineer with the Iowa Division of the FHWA.

This report summarizes the monitoring and evaluation of the structure conducted immediately following the construction of the bridge and throughout the first couple years after construction.

BACKGROUND

Significant research has been conducted in the past by various researchers and engineers concerning the design, construction, implementation, and effectiveness of using timber as a bridge construction material. Timber has been proven to be a viable option for today’s transportation structures and, given the advancements in glued-laminated timber products, tomorrow’s transportation structures as well.

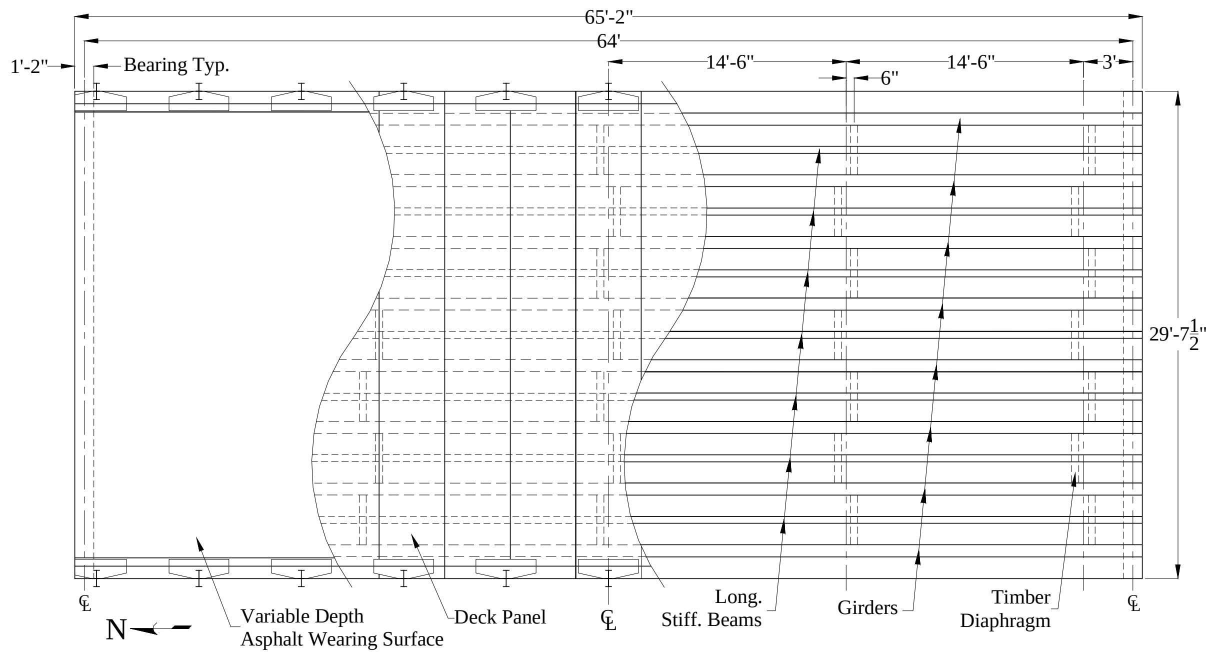
For centuries, engineers have been using timber bridges to span short crossings on our nation’s secondary roads. However, changes in the dynamics and demand of traffic, hydraulics, aesthetics, and economics have forced today’s engineer to be more creative and to develop methods to carry increased traffic across longer spans in a more efficient and cost-effective manner. In addition, engineers are seeking methods to improve or increase the strength and performance of current structures to handle the aforementioned changes in traffic, hydraulics, economics, and so forth.

The benefits of using FRP materials to strengthen and repair bridges has been realized and accomplished very effectively on various steel and concrete bridges in the past. Although strengthening of timber bridges with FRP is not a new topic, limited information exists on the short- and long-term effectiveness of using FRP reinforcement for these types of structures. Thus, the objective of this project was to evaluate the structural performance of the bridge in service. Additional field load tests and inspections will be performed over approximately the next two years to establish a database of information to address both short- and long-term performance using FRP plates to reinforce glued-laminated timber girders on a 64 ft. simple span bridge.

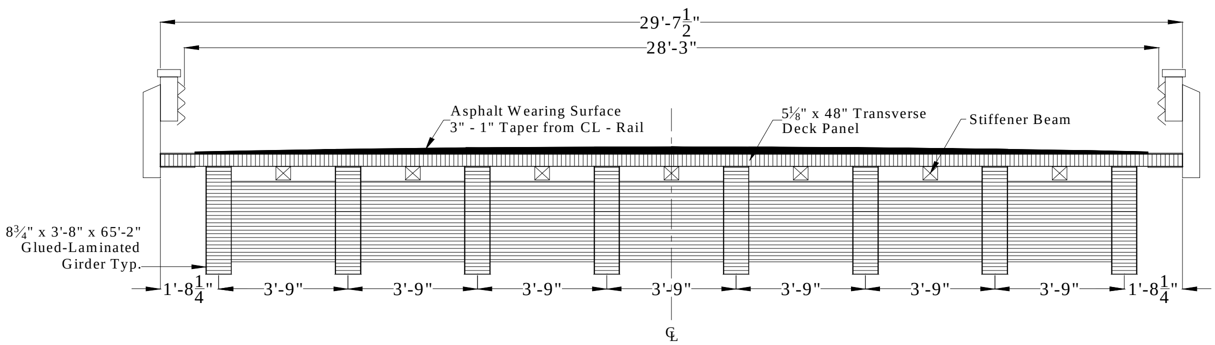
BRIDGE DESCRIPTION

The Delaware Co. Bridge is a two-lane, simple span, longitudinal glued-laminated girder bridge located on a low-volume gravel road spanning Lime Creek east of Ryan, IA. The glued-laminated timber girders and panels were fabricated at Alamco in Albert Lea, MN.

The bridge spans 64 ft., center to center of abutment, is 29 ft. 7.50 in. wide with zero degrees of skew and a roadway width of 28 ft.,3 in., as shown in Fig. 1. The superstructure consists of eight glued-laminated timber girders strengthened with FRP plates and a transverse glued-laminated timber deck. The eight girders measure 65 ft. 2 in. in length and consists of an 8.75 in. by 3 ft. 7.50 in. cross section of glued-laminated timber with a 0.50-in. thick by 8.75 in. wide FRP plate bonded to the tension laminate (see Figure 2). The girders are supported on either end by a 14 in. by 8.75 in. by 0.50 in. neoprene pad on the abutments' 14 in. seats. The supporting substructure consists of timber piles, timber abutment caps, and a timber plank abutment back wall.



a. Plan view



b. Cross section view

Figure 1. Delaware County Bridge

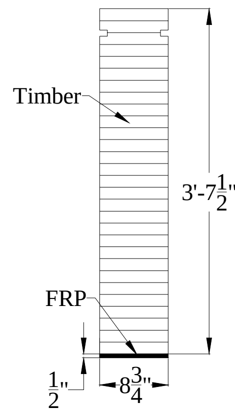


Figure 2. Girder cross section

The exterior girders on either side of the bridge are inset 1 ft. 8.25 in., measured from the outside edge of the deck to the centerline of the girder, and the center-to-center girder spacing is 3 ft. 9 in. Glued-laminated timber diaphragms are located at the abutments, 1/4 span, and midspan and longitudinal timber stiffener beams are bolted to the bottom of the deck midway between each girder.

The glued-laminated timber deck spans transverse to the girders and consists of nominal 5 in. by 48 in. panels measuring 29 ft. 7.50 in. in length. The panels are set against one another and are attached to the girders with metal s-clips bolted to the deck and inset into a groove in the girders. The guardrail is composed of a three-beam rail and steel posts attached to the deck with bolts; a variable depth asphalt wearing surface provides protection for the timber deck panels and also serves as the riding surface. Figure 3 illustrates the wearing surface utilized on this structure. No curbs are present on this bridge.

Figure 3. Delaware County Bridge wearing surface

EVALUATION METHODOLOGY

Girder and deck deflections were recorded at critical locations with the use of an Optim Megadec data acquisition system (DAS), a Dell laptop computer running TCS software for communication with the Megadec, and ratiometric displacement transducers. In addition, strains were recorded at critical locations with the use of the Bridge Diagnostics, Inc. (BDI) Intelliducers and Structural Testing System (STS).

Using the global deflection data collected, differential deflections and lateral load distribution factors were calculated during post-processing of the data. Differential deflections have been known to be a problematic area on these types of bridges, and approximate distribution factors provide some insight to the actual load distribution characteristics of the bridge compared to those assumed in the design.

INSTRUMENTATION

Figures 4 and 5 illustrate the positioning of the displacement and strain transducers on the Delaware Co. Bridge, respectively. Displacement transducers were installed on the underside of all eight girders at mid-span and on the underside of girders G1-G4 at quarter-span. Transducers were installed on the underside of the deck approximately 1 in. from the panel joint on both sides of the joint at multiple locations. The transducers on the girders were used to evaluate the global deflection performance of the structure, while those on the underside of the deck panels were used to determine the localized deflection performance of the deck panels.

Given the variable localized material properties of timber and the orthotropic nature of the material, there is some difficulty accurately measuring strains in the components of a timber bridge. However, with careful planning and gage application, strains in timber members may be measured and approximated to a degree that provides useful information about the behavior of the members and the structure as a whole.

Strain gages were positioned on girders G1 and G4 at mid- and quarter-spans in locations such that the distribution of stress over the girder cross section and the effectiveness of the bond between the glued-laminated timber and the FRP laminate could be evaluated. As shown in Figure 5, two gages were installed on one side of each girder several inches from its top and bottom, and one gage was installed on the bottom face of the FRP. In addition, strain gages were installed on the underside of two longitudinal stiffener beams in locations adjacent to the strain gages on the girders.

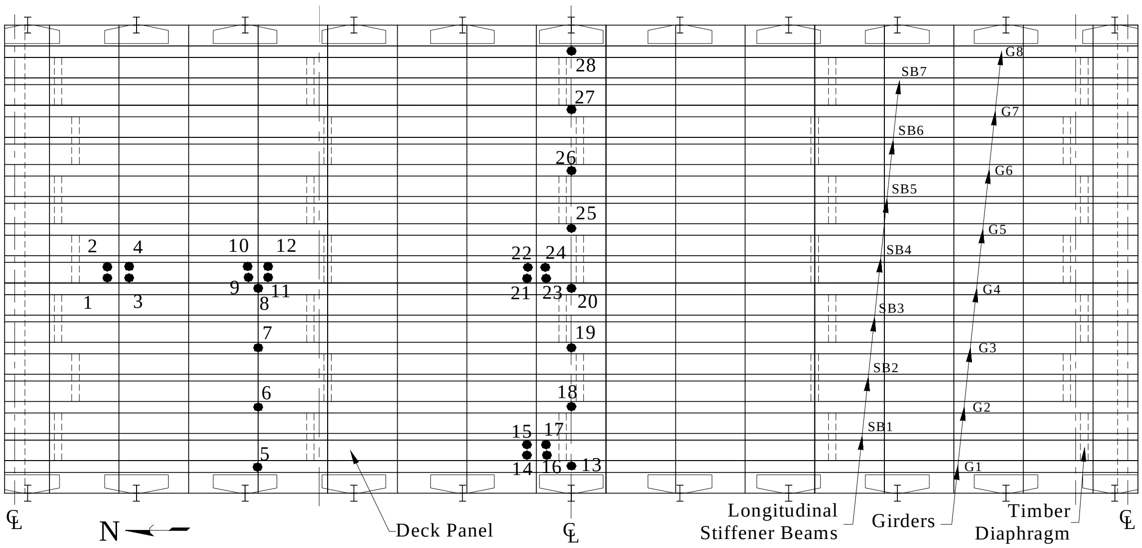


Figure 4. Delaware County Bridge displacement transducer locations

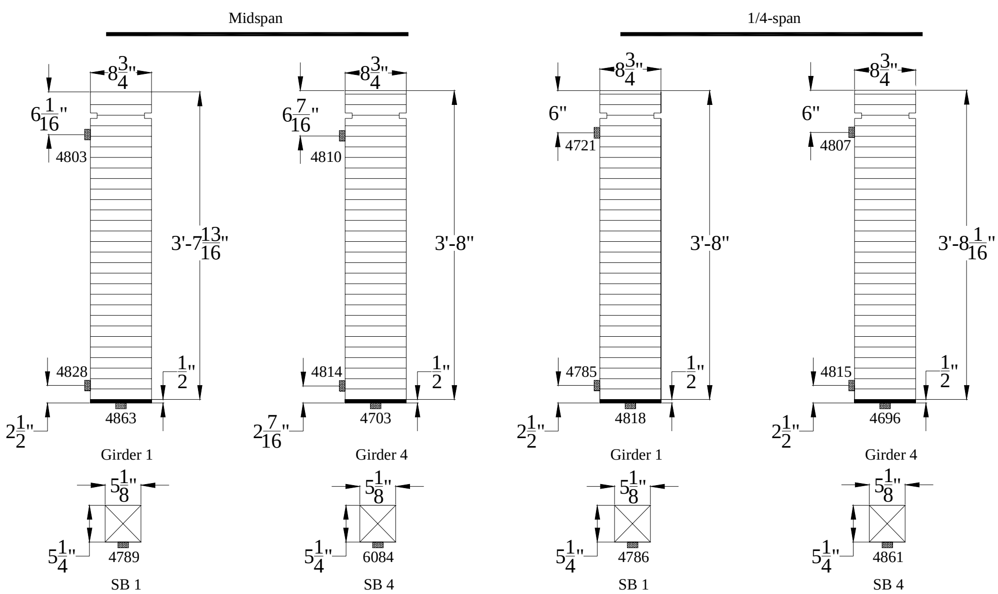
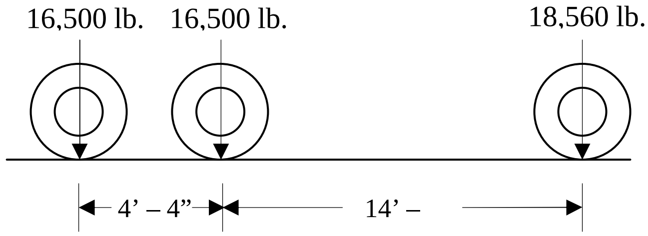


Figure 5. Delaware County Bridge strain gage locations

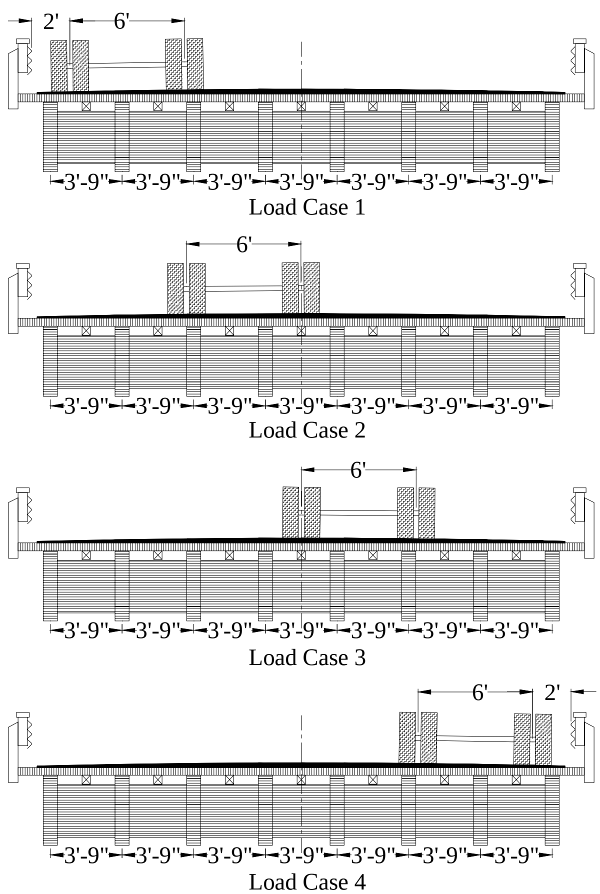
STATIC LOADING

The bridge was loaded with a fully loaded (51,560 lb.) tandem axle dump truck provided by the Delaware County Secondary Roads Department. The vehicle configuration and axle loads are illustrated in Figure 6a. Two runs were completed for each of the four load cases investigated, which are illustrated in Figure 6b. The first load case consisted of the load truck driving southbound at crawl speed with the passenger-side wheel line offset 2 ft. from the west guardrail. The second load case consisted of the load truck driving southbound at crawl speed with the driver-side wheel line centered over the longitudinal centerline of the bridge. The third load case consisted of the load truck driving southbound at crawl speed with the passenger-side wheel line centered over the longitudinal centerline of the bridge. The fourth and final load case consisted of the load truck driving southbound at crawl speed with the driver-side wheel line offset 2 ft. from the east guardrail.

The use of transversely symmetric load cases allows for verification that the load distribution characteristics are consistent across the width of the bridge and that the bridge is behaving as expected.



a. Vehicle configuration and axle loads



b. Load cases

Figure 6. Delaware County Bridge

CONDITION ASSESSMENT

Girders

Three condition assessments were conducted on the girders prior to load testing of the structure: first, pre-construction at the laminating plant; second, post-construction at the bridge site once the girders and deck panels had been set in place; and third, prior to load testing.

Pre-Construction

The initial inspection, conducted at the Alamco Wood Products, Inc. plant in Albert Lea, MN, found the glued-laminated girders were in excellent condition upon removal from the clamps. The girders were then planed, drilled, routed, and prepared for FRP installation.

Immediately following the fabrication of the girders, representatives from Iowa State University and Delaware County traveled to the plant site to witness the FRP installation. Mark Nahra, the Delaware County Engineer, was onsite to observe the installation of the FRP plate on the eight girders, which was performed by Justin Crouse, a research associate for the University of Maine. Installation of the 0.50 in. FRP plate transpired with no complications.

Figures 7 and 8 illustrate the condition of the glued-laminated girders before and after installation of the FRP, respectively.

Figure 7. Glued-laminated girders prior to FRP installation

Figure 8. Glued-laminated girders after FRP installation

Post-Construction

Delivery and installation of the girders went well with only one minor incident noted. During installation, girder G1 was apparently bumped, causing delamination of approximately the first 1 ft. of the FRP plate at the north end of the girder. Inspection revealed that the damage, which is difficult to identify, actually occurred in the timber itself and not the bond between the timber and the FRP. It appeared that approximately 1 in. of the girder was delaminated from the rest of the girder but not completely detached. It is anticipated that this will have little to no effect on the effectiveness of the FRP reinforcement, given the location of the damage.

Prior to Testing

There was no change found in the condition of the glued-laminated timber girders at the time of testing since the last inspection. All eight girders were well seated and no defects, other than the delamination which occurred during installation, were evident in the FRP reinforcement or glued-laminated timber portions of the girders. In addition, moisture content readings were taken, using an Elmhurst Moisture Meter with a 2 in. pin in both the deck panels and the girders prior to testing. In all locations the moisture content was approximately 15%.

Deck Panels

The deck panels are standard panels for this type of bridge construction, thus inspection prior to construction was not warranted; however, inspection of the deck panels was conducted post-construction and again prior to load testing.

Post-Construction

The initial inspection of the deck panels, immediately following placement on the girders, found no defects or problems in the glued-laminated deck panels themselves. The deck panels appeared to be relatively well seated on the girders in most locations; however, there were minor problems evident in the placement and attachment of the panels to the girders which may be significant in the future.

There were several locations where the deck panels were installed with gaps ranging from 0.25 in. to 0.50 in. between adjacent deck panels. However, there were also areas where the deck panels were set tightly against one another as desired.

Prior to Testing

Similar to the girders, there was no obvious change in the condition of the deck panels from the previous inspection. The deck panels had no areas of discoloration or damage and looked to be well-seated on the girders. However, the gaps between the girders were still evident at the edges of the deck where the panels were not covered by the wearing surface, and some of the s-clips had not yet been tightened in several locations.

Asphalt Wearing Surface

Prior to Testing

Currently, only one inspection has been completed on the asphalt wearing surface on the Delaware County Bridge. This initial inspection, conducted immediately before load testing of the bridge, resulted in some interesting findings.

First, there appeared to be no moisture-blocking membrane placed between the glued-laminated deck and the wearing surface to prevent moisture from coming in contact with the deck and underlying girders should there be a break in the wearing surface. Second, the wearing surface did not cover the entire deck surface but, instead, terminated just inside the metal base plates used to attach the guardrail posts to the deck, leaving approximately 1 ft. of the deck surface exposed. Third, the wearing surface was measured as approximately 3 in. in depth at centerline and tapered to approximately 1 in. in depth at the guardrail. Lastly, evident at each panel joint and at both abutments were small, noticeable, transverse cracks in the asphalt wearing surface. Figure 9 attempts to illustrate these cracks, although they are difficult to identify. As alluded to previously, these cracks will allow for infiltration of moisture to both the deck panels and girders, which may lead to further problems in the future.

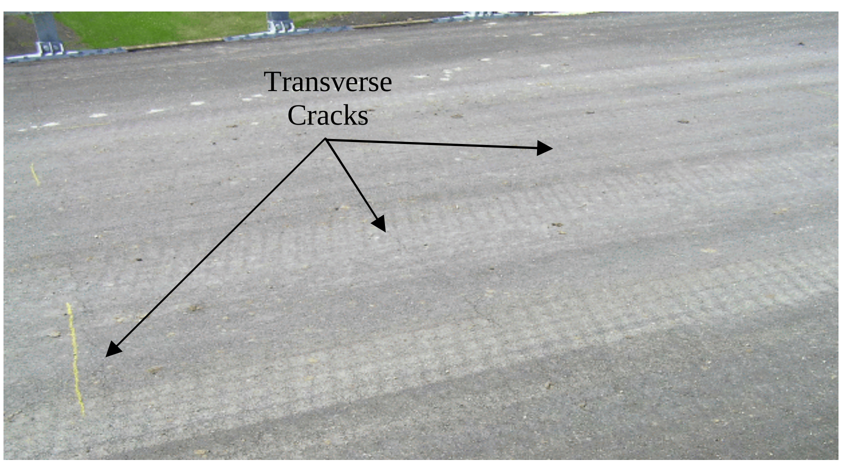


Figure 9. Transverse cracks at panel joints on Delaware County Bridge

RESULTS AND DISCUSSION

Global Deflection Performance

Illustrated in Figure 10 is the transverse global deflection of the bridge for all four load cases when the load truck is positioned longitudinally near mid-span. The maximum deflections were approximately -0.90 in. to -1.00 in. for load cases 1 and 4 at the exterior girders. When the load truck was positioned near the longitudinal centerline of the bridge; as in load cases 2 and 3, the maximum deflections were approximately -0.55 in. The decrease in the maximum deflection from load case 1 and 4 to load case 2 and 3 is likely the result of transverse load distribution characteristics associated with load position.

Current design specifications call for deflection checks of the form L/n, L being the clear span of the bridge. For this bridge, the America Association for State Highway Transportation Officials (AASHTO) Standard Specification (1996) specifies that global deflection of timber bridges be limited to 1.51 in.; the AASHTO LRFD (1998) specification limits the same deflection to 1.77 in.; the timber design manual, *Timber Bridges: Design, Construction, Inspection and Maintenance* (Ritter 1990), published by the Forest Service, limits this deflection to 2.09 in. The maximum deflections for the Delaware Co. Bridge for each load case are 0.72 in., 0.68 in., 0.41 in. and 0.40 in. after normalization to the standard HS-20 load truck for comparative purposes. This indicates that the deflection performance of the bridge is within the limits of the current design specifications and manual.

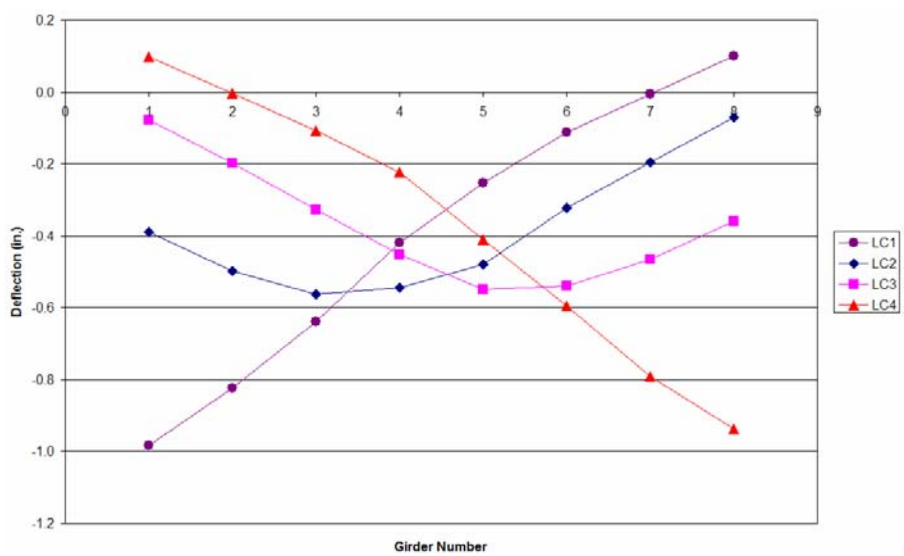


Figure 10. Girder deflection for load cases 1-4, load truck near mid-span

To investigate the level of girder end restraint, deflections were approximated from equations based on standard beam theory for a pinned-pinned condition and a fixed-fixed condition. These deflections were then compared with the measured deflections. Illustrated in Figure 11 are the calculated and measured deflections for one girder, which is representative of all girders for all four load cases. Since the experimental data trend line is closer to the fixed-fixed trend line than the pinned-pinned trend line, there is apparently some level of rotational restraint at the girder ends of the Delaware Co. Bridge.

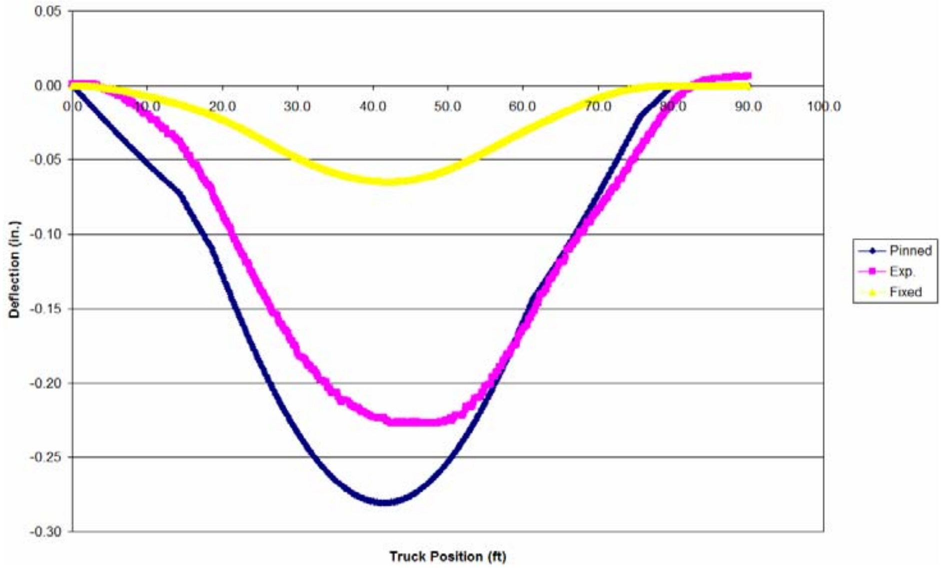


Figure 11. Calculated and experimental deflections for Delaware County Bridge

To investigate the load distribution characteristics of the bridge, approximate distribution factors were calculated from the measured deflections and compared with those computed with code equations. Using the assumption that all girders are of equal stiffness, an approximation of the distribution factors for each load case can be obtained from Equation. 1 using the physical test data. To obtain a more accurate estimate of the load distribution characteristics of the bridge, the distribution factors from two load cases were added together to obtain the approximate load distribution for the bridge with both lanes loaded, as assumed in the code.

$$DF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i} \qquad (1)$$

where

DF_i	= distribution factor of the *i*th girder (lanes/girder)
D_i	= deflection of the *i*th girder
SD_i	= sum of all girder deflections
n	= number of girders

Illustrated in Figure 12 are the codified and experimental distribution factors for the Delaware County Bridge. From Figure 12 it is clear that the equations in [1], [2], and [3] used to calculate distribution factors are conservative.

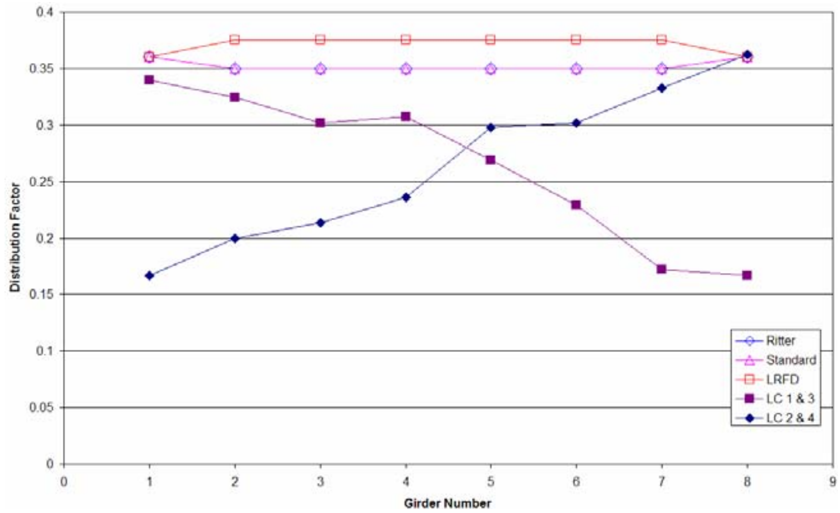


Figure 12. Experimental and codified distribution factors

Deck Panel Deflection Performance

In general, the deflection performance of the deck panels was satisfactory. Transversely, the deflection of the panels was as would be expected, with the deflection pattern of the panels following the same deflection pattern as the girders. This indicates that the deck panels possess adequate stiffness to effectively span between the girders. In previous research, it was determined that differential panel deflections were partially responsible for the deterioration of the wearing surface, specifically the cracking above the panel joints. Thus, the differential panel deflections and the effect the longitudinal stiffener beams have on these deflections are of particular interest.

The only specified limitation on this type of deflection is given in [3]. The manual states that relative deflection of the panels should be limited to 0.10 in., with a further reduction in this limit for deck panels supporting a pedestrian walkway or an asphalt wearing surface. Any reduction in the 0.10 in. limit is left up to the designer/engineer's judgment and based solely on experience and user preference, not on the actual structural or serviceability performance of the bridge.

Listed in Table 1 are the maximum differential panel deflections for all eight locations for all four load cases. As shown in Table 1, the calculated differential panel deflections are significantly less than the 0.10 in. limit for all load cases investigated. The largest differential panel deflection calculated was 0.027 in. In addition, there appears to be little difference between the differential panel deflections calculated adjacent to the stiffener beams and those calculated midway between the stiffener beams and the girders. These small differential panel deflections suggest that the stiffener beams are limiting the magnitude of the differential panel deflections between the girders.

Table 1. Differential panel deflections of the Delaware County Bridge

Load Case	1-3 (in.)	2-4 (in.)	9-11 (in.)	10-12 (in.)	14-16 (in.)	15-17 (in.)	21-22 (in.)	23-24 (in.)
1	0.005	0.006	0.002	0.011	0.027	0.022	0.007	0.004
2	0.018	0.016	0.004	0.014	0.019	0.018	0.015	0.017
3	0.020	0.017	0.007	0.013	0.016	0.017	0.020	0.017
4	0.003	0.007	0.002	0.008	0.013	0.013	0.008	0.010

Strain

Using the measured strains from the live load tests, strain magnitudes were evaluated and compared with the code specified yield or ultimate strains (or in the case of the timber manual, the tabulated strains), and the location of the neutral axis and the strain distribution across the girder cross section were approximated.

Strain Magnitudes

The maximum measured strain at mid-span for girders G1 and G4 for all four load cases are listed in Table 2. The calculated strains from [3] are listed in Table 3 and were based on the tabulated bending stresses and modulus of elasticity for glued laminated timber. The tabulated bending stresses are 640 psi (C), 2,400 psi (T), and the tabulated modulus of elasticity is 1,800 ksi for the combination symbol 24F-V3. In addition, the ultimate strains of the FRP are also listed in Table 3. These values were calculated based on the published ultimate tensile strength and tensile modulus of elasticity and the ultimate compressive strength and compressive modulus of elasticity which are 130 ksi, 5,600 ksi, 102 ksi, and 5,400 ksi, respectively.

Based on the strain values listed in Tables 2 and 3, the strains in the timber load tests were below the tabulated tensile strain and less than approximately 60% of the tabulated compressive strain. Likewise, the measured strains in the FRP are below the specified ultimate strain of the FRP.

Table 2. Maximum measured strains (microstrain)

		Load Case (see Fig. 9)			
	Girder	**1**	**2**	**3**	**4**
Timber	G1 (Top)	228 (c)	110 (c)	26 (c)	34 (t)
	G1 (Bottom)	240 (t)	96 (t)	32 (t)	8 (c)
	G4 (Top)	93 (c)	178 (c)	195 (c)	102 (c)
	G4 (Bottom)	104 (t)	162 (t)	141 (t)	81 (t)
FRP	G1	340 (t)	123 (t)	22 (t)	-32 (c)
	G4	156 (t)	227 (t)	194 (t)	87 (t)

(c) – Compression
(t) – Tension

Table 3. Code specified tabulated and ultimate strains (microstrain)

	e_t	e_c
Timber (tabulated)	1,333	356
FRP (ultimate)	23,000	20,000

Neutral Axis Location

Using linear interpolation between the measured strains at the top of the girder and the bottom of the FRP laminate (Ref. Fig. 13 and Equation 2), the approximate neutral axis location for each instrumented location and each load case are listed in Table 4. Due to the deck connection detail, these calculations assume there is no composite action between the glued-laminated deck and the glued-laminated girders.

For load cases 1 and 2, the neutral axis is typically between 1 in. above and 2 in. below the mid-depth of the girder. For load cases 3 and 4, the neutral axis is typically between 1 in. and 6 in. below the mid-depth of the girder.

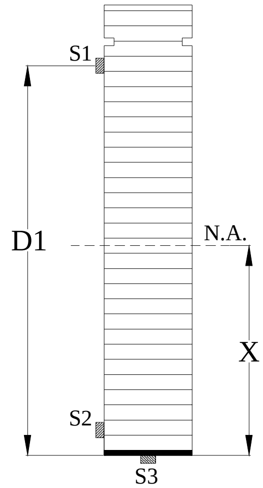


Figure 13. Girder neutral axis diagram

$$X = D1 * \left[\frac{S3}{(S3 - S1)} \right] \qquad (2)$$

Table 4. Delaware County Bridge, neutral axis location (X) measured from the bottom of the FRP

Load Case	G1 – ¼ Span (in.)	G4 – ¼ Span (in.)	G1 – Midspan (in.)	G4 - Midspan (in.)
1	22.74	23.49	22.56	23.50
2	20.51	23.25	20.09	21.63
3	16.07	21.56	16.41	18.90
4	18.45	21.65	18.19	17.55

SUMMARY

This report summarizes work completed in three primary phases: (1) inspection of glued-laminated girder fabrication including the FRP installation, (2) condition inspections throughout construction of the bridge, and (3) the static field load testing of the bridge.

Phase 2 of the project was completed in two steps. The initial inspection of the structure was completed in early July 2004 prior to placement of the asphalt wearing surface. Overall, the structure looked sound, although there were two minor items worth noting. First, there were locations where there were gaps as wide as 0.50 in. between adjacent deck panels. There were also several s-clips (used to attach the deck panels to the girders) underneath the deck panels which were not completely tightened. The second inspection was completed immediately before testing of the structure in mid August 2004. This inspection revealed noticeable transverse cracks in the asphalt wearing surface above each of the panel joints. In addition, several of the s-clips under the deck were still found to be not completely tightened. The loose s-clips did not appear to be a significant problem at the time of testing but may allow the deck panels to become cupped in the future, leading to further deterioration of other parts of the structure including the wearing surface, girders, and guardrail.

Phase 3, static load testing of the bridge, was completed immediately following final inspection. The two-lane, single span Delaware Co. Bridge, composed of eight glued-laminated girders reinforced with FRP on the tension side and a transverse glued-laminated deck, performed well under static loading. Global deflection of the structure was within current limits specified in AASHTO (1996), AASHTO LRFD (1998), and Ritter (1990). In addition, experimental lateral load distribution factors calculated from the field data suggests that the bridge distributes load more effectively than assumed in design.

From the load test data, there appeared to be no noticeable change in stiffness of girder G1, which had approximately the first foot of FRP on one end delaminated during bridge erection. The damage to the girder is only slightly obvious and only visible under close inspection of the girder. Since the damage was concentrated near the abutment, away from the tension zone of the girder, it is no surprise this had little effect on the response of the bridge during service loading.

The performance of the structure was equally adequate and within the limitations of AASHTO (1996), AASHTO LRFD (1998), and Ritter (1990) with regard to relative deflections such as differential panel deflections. For all load cases investigated, the calculated differential panel deflections were less than 0.03 in., which is approximately one-third of the 0.10 in. limit specified in Ritter (1990) for this type of deflection. The magnitudes of these deflections were found to be similar at locations adjacent to the longitudinal stiffener beams as well as midway between the stiffener beams and the girders. Based on differential panel deflections from similar bridges tested previously under similar loading configurations, the magnitude of these deflections appear to have been significantly reduced due in large part to the presence of the longitudinal stiffener beams.

Inspection prior to testing revealed transverse cracks in the asphalt wearing surface only weeks after placement of the wearing surface. The presence of these cracks along with small differential panel deflections indicates that there may be other factors affecting the condition of the asphalt-wearing surface on the bridge.

CONCLUSIONS

- The overall bridge structural performance under static live loading is adequate and within specified limits.
- Initial test results do not indicate a significant increase in stiffness resulting from the presence of the FRP reinforcement on the girders.
- There does not appear to be a decrease in the stiffness of girder G1 due to the delamination of the FRP which occurred during erection of the bridge.
- Regardless of the initial performance of girder G1 with the delaminated FRP, this girder should be inspected on a regular schedule to identify any further delamination or other changes resulting from the initial damage.
- The relative deflection performance of the deck panels with the longitudinal stiffener beams was well within specified limits; however, transverse cracking of the wearing surface above the deck panel joints appears to be occurring and is the result of unidentified factors.

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Understanding Traffic Impacts of Large-Scale Traffic Events

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ABSTRACT

The Iowa Department of Transportation has initiated a project to support local agencies in providing safe and efficient travel to and from the largest traffic events held on an annual basis within the state. These include college football games, the state fair, high school state wrestling and basketball tournaments, and large business seminars. This study provides insight into the planning and staffing for these events and the vehicle and pedestrian field observations per event and identifies the short- and long-term improvement strategies developed for each venue. These findings are applicable to events of all sizes and not just within the state of Iowa. The results are being used to promote the management of operational information as an asset in accommodating customer peak demands on the transportation system.

Key words: congestion—events—football—state fair—traffic—vehicles

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Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

An Overview and Update on Transportation Engineering and Road Research Alliance (TERRA)

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ABSTRACT

The Transportation Engineering and Road Research Alliance (TERRA) is a research governance structure formed in 2004 to foster a comprehensive road research program. TERRA brings together government, industry, and academia in a dynamic partnership to advance innovations in road engineering and construction. TERRA's partnering efforts reach beyond Minnesota to include transportation organizations in other states and in Europe.

TERRA was conceived as a new road research governance structure that would facilitate a comprehensive research program, with a strategic focus to take advantage of the MnROAD test facility and associated resources. TERRA was created after a task force of government, industry, and academic representatives investigated road research governing structures and evaluated ways to broaden the use of the unique capabilities of MnROAD.

TERRA's mission is to develop, sustain, and communicate a comprehensive program of research on pavement, materials, and related transportation engineering challenges, including issues related to cold climates.

For more information on the TERRA organization, visit www.TerraRoadAlliance.org or contact Laurie McGinnis, 612-625-3019, mcgin001@cts.umn.edu or Maureen Jensen, 651-366-5507, maureen.jensen@dot.state.mn.us. An article on the TERRA organization was also published in a recent issue of *TR News.* To read the article, visit http://onlinepubs.trb.org/onlinepubs/trnews/trnews249.pdf.

Key words: MnROAD—research organization—TERRA

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Evaluation of Long-Term Field Performance of Cold In-Place Recycled Roads

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ABSTRACT

Cold in-place recycling (CIR) has become an attractive method for rehabilitating asphalt roads that have good subgrade support and are suffering distress related to nonstructural aging and cracking of the pavement layer. Although CIR is widely used, its use could be expanded if its performance were more predictable. Transportation officials have observed roads that were recycled under similar circumstances perform very differently for no clear reason. Moreover, a rational mix design has not yet been developed, design assumptions regarding the structural support of the CIR layer remain empirical and conservative, and there is no clear understanding of the cause-effect relationships between the choices made during the design/construction process and the resulting performance.

The objective of this project is to investigate these relationships, especially concerning the age of the recycled pavement, cumulative traffic volume, support conditions, aged engineering properties of the CIR materials, and road performance. Twenty-four CIR asphalt roads constructed in Iowa from 1986 to 2004 were studied: 18 were selected from a sample of roads studied in a previous research project (HR-392), and 6 were selected from newer CIR projects constructed after 1999.

This report summarizes the results of a comprehensive program of field distress surveys, field testing, and laboratory testing for these CIR asphalt roads. The results of this research can help identify changes that should be made with regard to design, material selection, and construction in order to lengthen the time between rehabilitation cycles and improve the performance and cost-effectiveness of future recycled roads.

Key words: asphalt pavement performance—asphalt pavement rehabilitation—cold in-place recycling—recycled asphalt pavements

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

BACKGROUND

Cold in-place recycling (CIR) is an attractive rehabilitation method for asphalt roads that have good subgrade support and are suffering distress related to nonstructural aging and cracking of the pavement layer. The process is accomplished by milling three to four in. off the top of the pavement, screening and crushing the milled asphalt pavement to size, mixing the processed recycled asphalt pavement (RAP) with a stabilizing and/or rejuvenating agent, and relaying and compacting the processed material near its original location. New material that must be hauled in is usually limited to the stabilizing and/or rejuvenating agent and water. Heating is also not required for the milled asphalt material. The entire process is usually performed by a recycling train operating on the rehabilitated lane in close proximity to the location where the material is milled (Figure 1). A typical recycling train includes a milling machine, tanks for water and the rejuvenating/stabilizing agent, a mobile screening/crushing/pug mill unit, a paving machine with a windrow pickup unit, and one or more pneumatic and steel wheeled rollers. Variations are possible for smaller jobs, for areas where it is impossible to maneuver a recycling train, and for processes that require special equipment to handle special recycling agents. Further information on CIR and related recycling methods is available in the *Basic Asphalt Recycling Manual* (ARRA 2001).

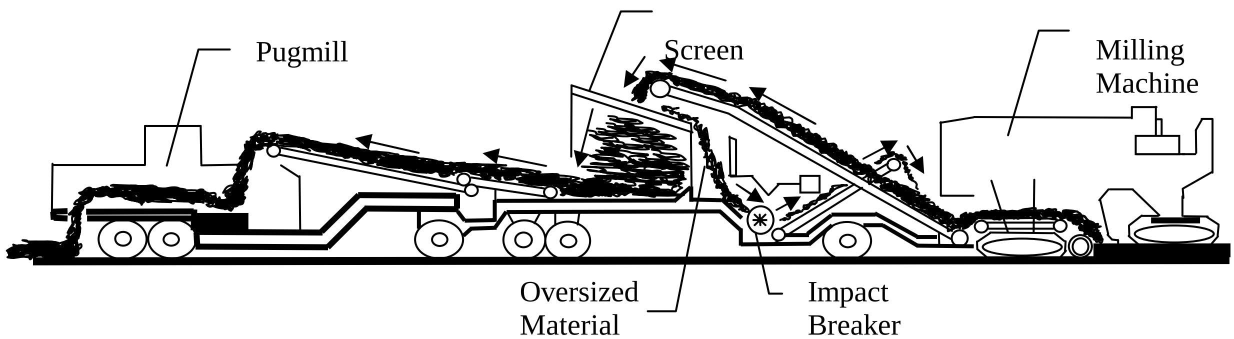


Figure 1. Diagram of a typical CIR milling, screening, crushing and pugmill unit, traveling left to right (paving and compaction units not shown; based on Jahren et al. 1998)

RESEARCH OBJECTIVE

Although CIR is a widely used pavement rehabilitation technique, its use could be expanded if its performance were more predictable. Transportation officials have observed roads that were recycled under similar circumstances perform very differently for no apparent reason. Additionally, although several attempts have been made, a rational mix design has not been developed, and design assumptions regarding the structural support of the CIR layer remain empirical and conservative. There is also no clear understanding of the cause and effect relationships between choices that are made during the design and construction process and the resulting performance.

The objective of this research project is to investigate these relationships, especially the aged properties of the CIR layer and the performance of the roads.

RESEARCH METHODS

Two major investigative approaches were used: a field investigation to assess actual road performance and a laboratory investigation to assess the CIR material properties associated with the performance. A test matrix (Table 1) was devised to aid in selecting a sample of roads with a wide range of characteristics thought to influence performance. These characteristics included average annual daily traffic (AADT, low < 800, high > 800), CIR age (old, 1986 and before to 1991; medium, 1992 to 1998; new, 1999 to 2004), and subgrade support condition (strong > 5000 psi; weak < 5000 psi). Research methods and results are described in greater detail in Jahren et al. (2007).

Table 1. Twenty-six test sections classified as a function of CIR age, traffic, and subgrade support/drainage

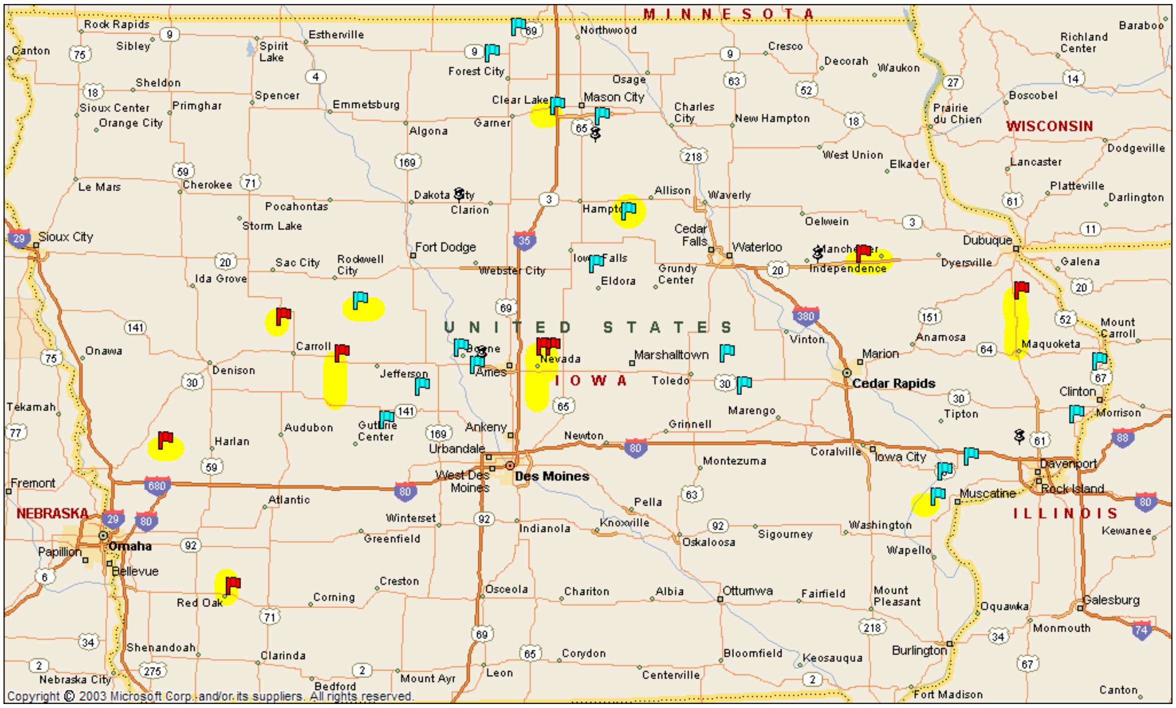
	Good support (>Subgrade modulus of 5,000 psi)		**Poor support (< Subgrade modulus of 5,000 psi)**	
Age	**High traffic (>800)**	**Low traffic (0–800)**	**High traffic (>800)**	**Low traffic (0–800)**
Young (1999–)	IA-44, Harrison	US-20, Delaware US-61, Jackson IA-48, Montgomery	N-58, Carroll N. of Breda, Carroll S-14, Story	S-27, Story
Medium (1992–1998)	-	IA-175, Calhoun IA-4, Guthrie F-70, Muscatine	V-18, Tama E-52, Boone T-16, Butler	G-28, Muscatine D-35, Hardin
Old (1986–1991)	R-34, Winnebago B-43, Cerro Gordo R-60, Winnebago	S.S.L., Cerro Gordo Z-30, Clinton E-66, Tama	198th St., Boone E-50,Clinton	Y-14, Muscatine IA-144, Greene

The 20 roads originally sampled by Jahren et al. (1998) were first entered into the matrix. It was considered desirable to include these roads because considerable historical information had already been collected for them. Also, because their performance had been assessed in 1996, the opportunity existed to provide a longitudinal performance comparison in 2004. New roads were then added to the matrix with the goal of filling empty test cells. These included high-volume primary routes that used foamed asphalt and engineered emulsion as a rejuvenating/ stabilizing agent, and a few secondary roads whose construction was directly observed by researchers. In all, 26 roads were included in the sample: 18 from the 1996 sample and 8 new roads. For each sample road, one representative test section was selected that was 1,500 ft. long. For the roads that had been included in the 1996 study (Jahren et al. 1998), the sample roads selected for the present investigation coincided with the test section selected for the previous investigation. The sample roads are listed in the classification matrix in Table 1, and their locations are shown in Figure 2.

The final sample ranged from 1 to 19 years in CIR age and from 130 to over 5,000 AADT in traffic. Evidence, obtained from researcher observations and questionnaires sent to the agencies in charge of each of the sample roads, indicated that there were a variety of support conditions.

Images of field distresses for the sample were digitally captured by University of Iowa researchers (Lee and Kim 2007) and analyzed using a partially automated, technician-assisted procedure. Images were digitally captured using a vehicle-mounted digital camera that was computer-synchronized with the vehicle's speed to capture overlapping images of the road at highway speed. The images were then analyzed on a personal computer, where the operator traced cracks using a mouse and outlined areas of other distresses. Distresses included transverse, longitudinal, block, and alligator cracking; patching; raveling; and bleeding. To determine crack severity, the operator estimated crack width by scaling. The resulting measurements were then processed to calculate a pavement condition index (PCI) according to the method outlined by Shahin and Walther (1990). While this approach was more automated than the approach used in the 1996 study (Jahren et al. 1998), the University of Iowa research team conducted a side-by-side comparison of the two methods and found that they produced equivalent results (Lee and

Kim 2007). Rut measurements were taken at 50 ft. intervals by the University of Iowa researchers using a dipstick measuring device on a 4 ft. straightedge.



Blue (lighter) flags: Older roads
Red (darker) flags: Newer roads

Figure 2. Location of sample roads

The extent of structural support and layer stiffness were inferred from the calculated results of a falling weight deflectometer (FWD) testing program. Iowa State University researchers (Chen and Jahren 2007) engaged the Iowa Department of Transportation Special Investigations Team using a JILS-20 unit manufactured by Foundation Mechanics, Inc. For each 1,500 ft. test section, the load plate was dropped 16 times, (once every 100 ft., including the locations at the beginning and end of the test section). Deflections were collected from an array of eight sensors. The FWD testing was conducted in December of 2004 and March of 2005. The low temperatures and freezing conditions may have had some effects on the results. Since most of the tests were conducted under similar weather conditions, it was expected that FWD results would provide a comparison of subgrade and pavement stiffness amongst the sample roads.

For the purpose of backcalculating the pavement layer strength moduli, the pavement structure was assumed to have three layers: an HMA layer, which represented the HMA overlay above the CIR layer; the CIR layer; and a layer (designated FDN) that included everything below the CIR layer, including the remaining unrecycled HMA, base layers, and subgrade. This foundation layer was purposefully generalized as one layer in order to accommodate the limitations of back calculation programs, even though the actual field condition usually included three discrete layers (remaining HMA, base, and subgrade). Researchers selected the CIR layer as one layer to isolate because it was the focus of this investigation. A computer application named BAKFAA (FAA 2003) was used to back calculate the moduli of the HMA, CIR, and FDN layers.

A laboratory investigation of the material properties of the CIR layer was conducted by Iowa State University researchers (Chen and Jahren 2007). The samples of cold in-place material were obtained by coring the test section at six locations approximately 300 ft. apart. The cores alternated between the right wheel track and the centerline of the lane. The cores were trimmed to remove the non-CIR portions and to produce the correct sample size (4 in. diameter, 2 in. height) and were then weighed and measured. Then the cores were subjected to several tests:

- Bulk specific gravity (AASHTO T166-93)
- Indirect tensile strength (wet and dry)
- Theoretical maximum specific gravity (ASTM D 6857-02)
- Ignition extraction of binder (ASTM D 6307) for binder percentage
- Chemical extraction and recovery of binder (AASHTO T 164)
- Aggregate gradation (AASHTO T27-93)
- Penetration (AASHTO T49-96)
- Dynamic shear rheometer (AASHTO T315-02)
- Bending beam rheometer (AASHTO T313-02)

The results of these tests and measurements were collected and analyzed in several ways. The PCI measurements were plotted against time, and predictions were made regarding the service life of the roads. Separate analyses were conducted to determine the ways traffic volume and subgrade support affected the performance of these roads. The changes in each of the individual distresses were plotted against time to determine whether further findings could be gleaned.

For the investigation of the material properties, it was desired to identify material properties that were associated with unusually good and unusually poor performance. An appropriate statistical analysis method was developed to accomplish this goal. The results of the analysis were examined and conclusions were drawn.

FIELD PERFORMANCE ANALYSIS

The following sections describe the results obtained from the previously described methodologies.

Pavement Condition Index

The field performance measurements resulted in PCIs that ranged from 100 to 48, with 100 representing a pavement in excellent condition and a range of 40 to 60 representing a pavement that should be considered for rehabilitation (Tables 2 and 3). Several pavements that received ratings of 99 to 100 had been recycled within the past one to three years. However, some pavements that had been recycled 10, 14, and 12 years ago received ratings of 98, 97, and 96, respectively. The lowest rating was for Clinton county E50. This was the first road in recent history that was recycled in Iowa, and it was scheduled to be rehabilitated a year after this investigation.

Plots of PCI vs. age were produced for all of the sample roads. The PCI from the 1996 survey (Jahren et al. 1998) and the PCI from this survey were both plotted. A PCI of 100 was selected for the year of CIR rehabilitation. Visual review of the plots indicates that the rate of deterioration has likely diminished for the segment of time between the first and second survey compared to the segment of time between initial recycling and the first survey. Figure 3 illustrates this trend.

Table 2. Performance data including PCI, 18 older sample roads by 1st and 2nd surveys

Road	Subgrade modulus (ksi)	1996 survey			2004 survey		
		Traffic	Age	PCI	Traffic	Age	PCI
IA4	19.81	820	2	100	1850	10	98
IA144	13.16	1110	7	62	1770	15	54
IA175	22.05	1920	3	100	1560	11	63
Y14	13.03	990	9	86	1490	18	60
F70	23.78	950	3	100	1250	12	92
E66	11.9	1080	6	94	1170	15	93
SSL	23.53	600	6	81	1140	15	54
G28	19.96	940	5	98	1100	14	73
D35	10.69	665	4	85	930	13	78
Z30	18.5	850	7	99	890	16	70
T16	10.39	470	3	100	610	12	96
V18	16.7	550	5	100	570	14	97
R60	19.86	340	6	72	550	15	70
E50	13.21	520	10	81	540	19	48
B43	22.21	570	7	82	450	16	61
R34	15.94	620	6	90	400	15	89
E52	8.94	290	5	95	390	14	85
198th ST.	12.63	300	8	71	130	17	54

Table 3. Performance data including PCI, 8 newer sample roads by 2nd survey

Road	Subgrade modulus (ksi)	Traffic (AADT)	Age	PCI
US-61	32.61	6200	3	87
IA48	18.93	1980	3	100
S27	12.11	1000	1	100
US-20	46.12	900	3	91
IA44	19.53	770	3	100
S14	14.04	740	1	100
N58	15.78	340	1	100
North of Breda	11.58	190	3	99

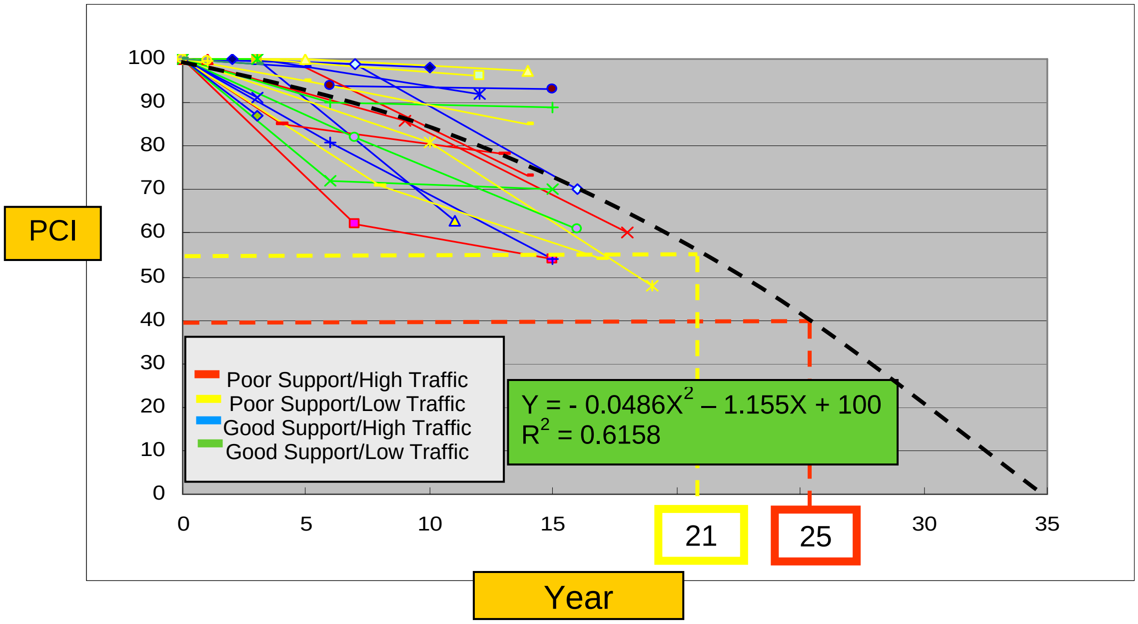


Figure 3. PCI performance over several years based on distress surveys

Pavement Distress

Transverse cracking and rutting accounted for the highest distress density, with densities ranging from zero to 5% for most roads. Distress density is the ratio of the quantity of distress in a section of road to the area of that section. For distresses that are measured by area, it is the percent of pavement in the test section affected by that particular distress. If the distress is measured in lineal feet (e.g., transverse cracking), the distress density is the number of feet of cracking observed divided by the area of the test section. Two roads exceeded that density in transverse cracking, and one exceeded it in rutting. Longitudinal and alligator cracking exhibited densities above 5% on five projects, but these types of cracking did not exist on other projects. No more than five sample roads exhibited distress densities above 1% in block cracking, edge cracking, or patching.

Service Life Predictions

Several statistical analyses were conducted to predict the service life of CIR roads. A regression analysis provided an expected performance curve based on data from the sample roads. For the purposes of this analysis, the acceptable service life was predicted to end when the PCI reached between 40 and 55. An analysis of all the roads indicated a predicted service life range from 21 to 25 years (see Figure 3). Separate analyses were conducted for roads with good and poor subgrade support, resulting in a predicted service life range of 26 to 34 years and 18 to 22 years, respectively. Separate analyses comparing service life to traffic level revealed little difference.

Lab Characterization Analysis

The CIR materials properties measured as part of this investigation were, in some cases, different than those that would be expected from typical HMA materials. Table 4 compares some typical HMA and CIR properties. The percentage of air voids (V_a) for CIR ranged from 4.5% to 14.3%, in comparison to 5% to 9% for HMA. Higher V_a values would be expected from the CIR mix because it is cold compacted, and obtaining a density that is as high as HMA's density is unlikely. The indirect tensile test (wet) yielded results that ranged from 9.60 to 28.70 psi. This compares to a typical minimum of 100 psi for HMA.

Table 4. Comparison of typical HMA and CIR properties

Type of Property	Property	Typical HMA	CIR in this study
Mix	V_a (field measured, %)	5 ~ 9	4.5 ~ 14.3
Binder	G*/sin(delta)	> 2.2	230 ~ 4,700
Binder	G*sin(delta)	< 5,000	170 ~ 3,600
Binder	Penetration (dmm)	20 ~ 30	0 ~ 30.3
Binder	S(t) (Mpa)	< 300	204 ~ 962
Binder	m-value	> 0.3	0.16 ~ 0.32
Pavement Layer Structural	Pavement modulus (ksi)*	100 ~ 6,000	1,000 ~ 18,000
Subgrade Layer Structural	Subgrade modulus (ksi)*	1 ~15[#]	10 ~ 70[@]

* Backcalculated value from FWD testing results
[#] Range of typical values for readings taken in summer
[@] Range of values of results on this project, some subgrades were frozen because testing was conducted in the winter months.

Relative PCI

One goal of this investigation was to identify material properties associated with good and poor performance. The concept of relative PCI was developed to separate good from poor performance, and it became the response variable for subsequent statistical modeling efforts. A regression relationship between the actual PCI and age of CIR roads was developed to identify the expected performance of CIR roads (expected PCI). Then, road performance (actual PCI) was compared to this regression relationship (expected PCI). Roads that performed better than expected at a particular age were assigned positive relative PCI values, while roads performing worse than expected were assigned negative relative PCI values (Figure 4). The magnitude of the relative PCI was the difference between the expected and actual PCI. The relative PCI values ranged from -22 to +18. The road with the lowest relative PCI was IA 175 in Calhoun County, which was suffering distress from rutting and longitudinal cracking. The road with the highest relative PCI was Tama County V18.

Subsequent analyses attempted to identify material properties that were associated with low and high relative PCI values. The following independent variables were initially considered:

- Cumulative traffic
- Modulus of the HMA layer (psi)
- Modulus of the CIR layer (psi)
- Modulus of the FND layer (psi)
- Indirect tensile strength of the mixture for wet samples (IDT_{wet}, psi)
- Air voids (V_a, %)
- Complex shear modulus (G*, kPa) of CIR binder
- Flexural creep stiffness (S(t), MPa) of CIR binder
- m-value of CIR binder
- Type of aggregate

As will be described below, the resilient modulus of the CIR layer, IDT_{wet}, V_a, and cumulative traffic proved to have statistical significance in influencing road performance.

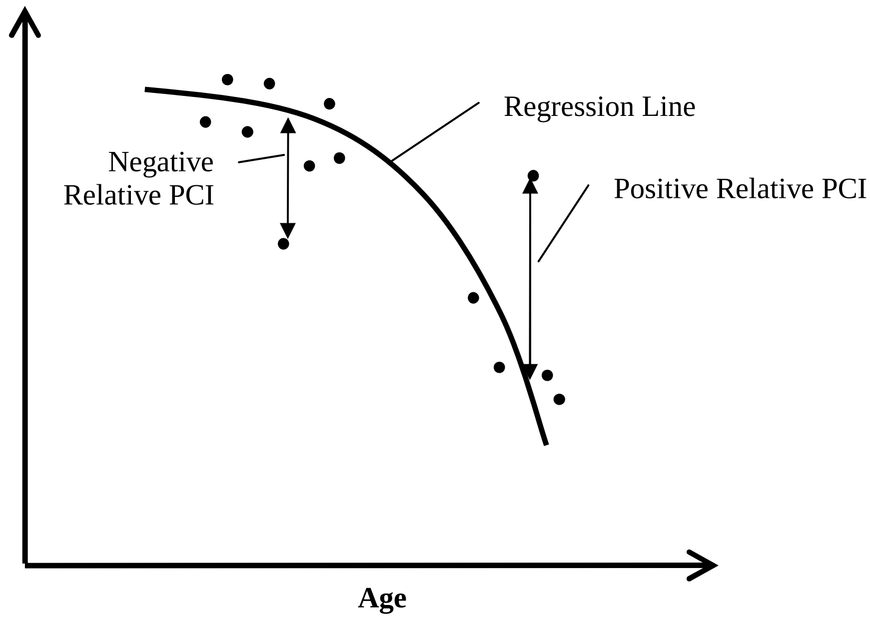


Figure 4. Relative PCI

First, a descriptive statistical analysis was conducted by visually reviewing scatter plots of each of the variables vs. relative PCI. This visual review of the plots suggested that a trend may exist between relative PCI and V_a, and between relative PCI and CIR modulus, with higher relative PCI values associated with higher V_a values and a lower CIR modulus (Figure 5).

Next, three separate multivariable models were developed and tested for roads in three categories:

1. All 24 CIR roads
2. Low-traffic roads (AADT < 800 VPD)
3. High-traffic roads (AADT > 800 VPD)

For category (2), the model that was developed was not statistically significant at the α=0.05 level. This suggests that variables not selected for this study, rather than the variables selected for this study, may have had more influence on pavement performance for this category of roads. However, the analysis indicated that higher IDT_{wet} values positively influenced performance on lower volume roads. Models that are statistically significant at α=0.05 were developed for categories (1) and (3). For category (1), the model indicated that road performance was influenced by V_a, CIR modulus, and cumulative traffic, with better performance associated with roads that had a higher V_a value, a lower CIR modulus, and lower cumulative traffic (cumulative traffic is the product of traffic volume and the project age). For category (3), the models indicated that road performance was influenced by V_a and CIR modulus, with better performance associated with higher V_a values and a lower CIR modulus.

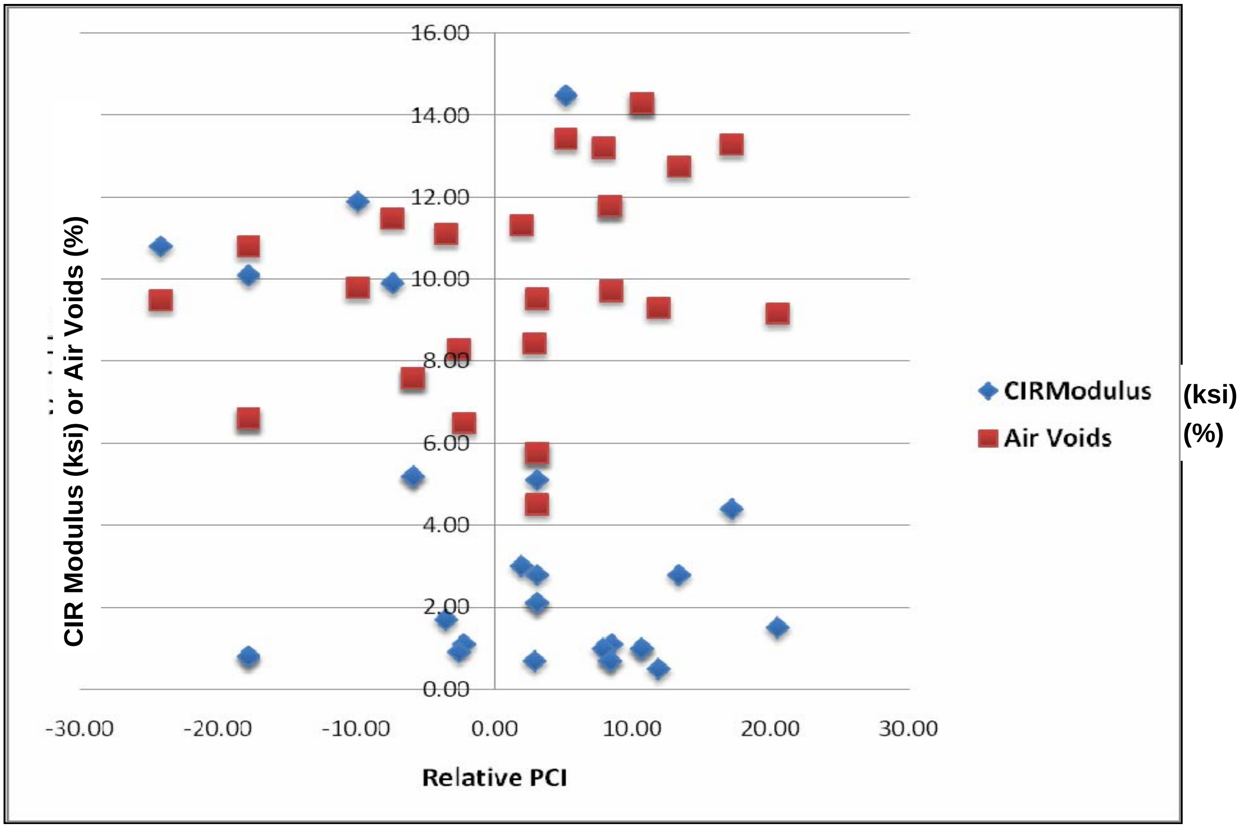


Figure 5. Selected material properties vs. relative PCI

DISCUSSION

The results of the analysis indicated that better performance is associated with CIR layers that are more flexible, have a higher percentage of air voids, and are less moisture-sensitive.

The CIR modulus is a measure of stiffness, with higher values associated with greater stiffness. Within the range of the data in this study (500 to 14,500 ksi), better performance was associated with roads that had CIR layers with lower moduli. V_a indicates the percent volume of air voids in an asphalt mix. Within the range of data in this study (4.52% to 14.30%), roads that had CIR layers with higher V_a percentages were associated with better performance. Thus, within the range of the data in this study, a more flexible, more porous CIR layer was associated with better performance.

This finding supports the theory that the CIR layer acts as a stress relieving layer that slows the propagation of cracks from the layers below and reduces the tendency of the HMA overlay to crack (Abd El Halim 1985; Abd El Halim 1986). In terms of the range of values included in this study, if the modulus of the CIR layer falls below this range or if the V_a percentage exceeds this range, poor performance will result at some point, because some minimum level of stiffness and some maximum value of V_a are required for good road performance.

CONCLUSIONS

The following conclusions can be drawn as a result of this investigation:

- The predicted service life for CIR roads is 21 to 25 years, based on the predictions of best-fit regression models that determine when the pavements would reach a fair condition, which is defined as a PCI value between 40 and 55.
- The additional performance information collected since the previous study (Jahren et al. 1998) allows for a better understanding of service life predictions than previously possible.
- The predicted service life of the test sections with good subgrade support was longer than that of the test sections with poor subgrade support. The average service life of CIR roads with good subgrade support is predicted as being up to 34 years, whereas that of CIR roads with poor subgrade support is predicted as being up to 22 years.
- The service life of the test sections under low traffic volumes is very similar to that of the test sections with high traffic volumes. Traffic level (all less than 2,000 AADT) did not seem to affect the performance as much as subgrade support. Particularly, the performance of pavements with good subgrade support was not affected by traffic level.
- Longitudinal and alligator cracking increased, whereas transverse cracking did not change much over time.
- Rutting, patching, and edge cracking increased primarily in sections with poor subgrade support, whereas block cracking converted to alligator cracking in some sections.
- The results of this study support the theory that the CIR layer acts as a stress relieving layer. Therefore, within the range of data analyzed, a smaller CIR modulus value (less stiff) and a higher V_a value for the CIR layer (more porous) indicates that better performance is expected. However, it is certain that if the CIR modulus is too low or the V_a is too high, poor performance will result.
- Within the range of data analyzed, a higher IDT_{wet} value significantly and positively affected the pavement performance of low-traffic roads.
- As expected, a higher amount of cumulative traffic is associated with lower relative pavement performance in both the models for high-traffic roads and for all 24 CIR roads.
- The first CIR road in Iowa, constructed in 1986, was rehabilitated in 2005 after reaching its fair condition in 2004, with a PCI value of 48. Its 19-year service life supports previously stated conclusions regarding expected service lives.
- One section with a very high traffic level of 6,200 AADT has performed reasonably well, although rutting started to develop after three years.

RECOMMENDATIONS

The following recommendations were made from this study:

- The service life predictions provided in the conclusions should be considered for use in economic analysis and network planning.
- Special attention should be directed toward providing proper subgrade support and proper CIR mixture materials for high-volume CIR roads, because the performance of such roads is most affected by these items. Decision makers should consider using FWD or dynamic cone penetrometer testing to evaluate the ability of the subgrade to provide proper support.
- Efforts should be focused on avoiding deterioration caused by environmental factors for lower volume CIR roads. High IDT_{wet} strength is considered to be an indicator that the mix is resistant to environmental attack. It is generally accepted that low-volume roads are more likely to fail due

to environmentally induced distress than traffic load-induced distress. Therefore, this conclusion is in alignment with generally accepted views regarding the failure modes of low-volume roads.

- Design and construction specifications and procedures should be considered that are in alignment with the conclusion that a CIR layer that has lower stiffness and higher air voids exhibits better performance. However, this conclusion/recommendation is limited to the range of data analyzed, and including materials with measurements outside these ranges will likely result in poor performance. Investigators should consider developing a concept of preferred ranges of air voids and CIR stiffness values that are likely to be different from those of HMA.
- More refined limits for V_a and CIR modulus should be developed through further investigations.
- Based on the range of results obtained from this investigation, future investigators should reevaluate the required sample size necessary for refining the conclusions of this investigation. Analyzing the results from more cores, FWD tests, and performance surveys may also provide statistically significant results in other areas of investigation.
- In future studies, efforts should be made to isolate the effects of the CIR layer from other layers in the pavement system. This will be a considerable challenge and will likely require more sampling, as noted in the previous recommendations.
- Phase angles should to be considered in future studies to account for the elastic and viscous properties of asphalt binders.
- The field performance of the sample roads examined in this study should be reevaluated in approximately 2010 (five years from the last survey) to confirm the predictions of service life.

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Lessons Learned in Developing a Highway Condition Assessment and Maintenance Quality Assurance Program

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ABSTRACT

Compass is the maintenance quality assurance program developed and implemented statewide by the Wisconsin Department of Transportation (WisDOT) since 2002. This program provides a comprehensive overview of highway operations conditions based on analysis of data from field reviews, condition surveys, and expenditures. Compass reports are tools to visualize and communicate trends and conditions, to prioritize resources, and to set targets for future condition levels. At the executive level, Compass demonstrates accountability of expenditures and illustrates consequences of funding and policy shifts.

Researchers at the Midwest Regional University Transportation Center at the University of Wisconsin-Madison have been working with the Compass program managers of WisDOT since 2002 to develop the analysis, reporting, and quality assurance procedures for the annual *Compass Operations and Executive Overview* reports. During these development years, several design and procedural adjustments improved the data collection and performance analysis. Some of these adjustments include redesigning the rating sheet for field data collection and revising the approaches to define, measure, and calculate maintenance backlog. After experimenting with alternative methods, one recent and significant change concerns the method for calculating trends in pavement condition.

One of the most critical success factors for a maintenance quality assurance program is communicating results to maintenance managers, program managers, agency executives, the legislature, and the public. With this in mind, the charts, tables, and outlines of the annual Compass reports have undergone successive revisions to make them meaningful, relevant, and easily understood.

The development of maintenance quality assurance programs is still in its infancy. As agencies implement these programs, a peer community and vocabulary have evolved. As the members of that community discuss results of data analysis, new ideas emerge leading to improvements in their programs and the maintenance quality of roadways. In this paper, we present our lessons learned in the evolving development of a maintenance quality assurance programs.

Key words: maintenance quality assurance—asset management—condition assessment

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BACKGROUND

Compass is the Wisconsin Department of Transportation's (WisDOT) annual quality assurance and asset management program that was implemented statewide in Wisconsin starting in 2002. Compass is a comprehensive overview of the quality of highway operations from the customer's perspective. Information gathered from this program is used as a tool to help understand trends and conditions. Development of the program is intended to assist decision makers in prioritizing resources, in setting targets for future condition levels for the highway system, in evaluating tradeoffs when allocating maintenance budgets to individual maintenance activities, and in understanding the consequences of funding and policy shifts. Furthermore, Compass illustrates impact of budgets and demonstrates accountability to the legislature and the public.

Table 1 lists the various report sections and the elements, features, and distresses and measures included. Feature conditions are reported as backlog percentages at the county, region, and statewide levels. Backlog is not as much a reflection of inadequate work, but instead impact of fiscal constraints. Backlog is the percentage of a particular feature in a condition where maintenance work is required, and thus would have been worked on, if sufficient budget was available. Compass reports the condition of pavements, signs, bridges, drainage, roadsides, and winter operations, The agency has inventory data to assess bridges, pavement, routine maintenance of signs, and winter maintenance. For other features, Compass assesses maintenance condition based upon sampled data.

Table 1. Content of the Compass report

Section	Element /Type	Feature/Distress Type
Field Review	Traffic	Delineators, Special Pavement Markings, etc
	Shoulders	Cracking, Potholes, etc
	Drainage	Culverts, Ditches, etc
	Roadsides	Fences, Mowing, etc
Winter Maintenance		Winter by the numbers
		Time to Bare/wet Pavement
		Winter crashes per VMT
		Winter Severity Index vs. Cost per lane mile
Pavement	Flexible	Flushing, Rutting, etc
	Rigid	Slab breakup, Distress, etc
Sign	Regulatory/warning Sign	Missing, damaged, not visible
	Non-regulatory	Missing, damaged, not visible
Bridge		Condition (NBI-based)
		Maintenance Needs
		Inspection Compliance

LESSONS LEARNED

Compass analysis and reporting have been developed and revised during its six years of implementation to improve the quality of the Compass program. These revisions include improvements in the scope of the program, data collection process, formulation for backlog, the analysis methodology, and the report. In this paper, we discuss the significant revisions.

1. Incrementally Grow the Scope of the System

The scope of Compass grows each year as more databases and business units of highway maintenance are included. The first statewide implementation in 2002 reported the sampled field review of shoulders, litter, mowing, markings and drainage. In 2003, analyses of the pavement and sign databases were added. Table 2 details the evolutionary development of the system scope. Incremental development allows the system to be up and running quickly because the scope is small. Incremental development may be the only alternative if the development team is only one or two people. As development progresses, the team becomes more experienced at defining the performance measures for analysis, and the overall process gets easier. As each new element is added, the development team must work with the maintenance managers to help them understand the purpose of the program and to gain access to the relevant databases. These maintenance managers may not have analyzed their data for maintenance quality assurance so there may be several iterations until they are comfortable that the measures are consistent with the assumptions and scope of their data.

Table 2. Evolutionary system development

Year	System Increment
2001	Pilot project - eight counties reporting field review of shoulders, mowing, drainage, markings, and litter
2002	Statewide implementation of field review (all 72 counties)
2003	Add Pavement and Sign reports
2004	Add Winter report and introduce the use of "% backlog" as the performance measure
2005	Add Bridge report
2006	Modification of pavement backlog calculation formula.

2. Provide Training and Institutionalize the Program

Data consistencies have improved significantly compared to the early implementation of Compass. Most of the improvements are a result of better training for the data collection team, including better clarification of measurement technique, revisions to the rating manual, and the development of the training DVD. As the program gets institutionalized there is more buy-in across the agency. The program gets institutionalized if it is linked to a business mandate. For example, Compass is used to establish maintenance targets and as an input to the state budget process.

3. Improve the Sampling Strategy of Highway Maintenance Features

As the only part in Compass using sample data to calculate backlogs, highway maintenance features have been using different strategies along the years. In the beginning, 25 segments were sampled from each county, with the total of 1,800 segments to be sampled. The next year, the sampling procedure was changed to sample 240 segments for each district. This method increased the amount of sample needed to be taken statewide, but because of the different size of counties, the results are believed to be more statistically representative. More recently, when WisDOT reorganized the counties in Wisconsin to be grouped in five regions from the eight districts it was before, another adjustment to the sampling strategy was needed. It was decided to sample 240 segments from each region, for a total of 1,200 segments sampled. This reduced the number amount of samples to be taken, and therefore, reduced the burden to the counties who are doing the work, but on the other hand, it still maintains the significance of the data at the region level.

4. Revisit Approaches for Defining, Measuring, and Calculating Maintenance Backlog

Several modifications have been applied along the years to the way features are measured and the maintenance backlog is defined. One of the examples is when early on Compass was set up to report highway maintenance feature conditions in detail and reported both the extent and severity of the deficiencies. It was then discovered that since these highway maintenance data are samples instead of the full inventory, not a lot of meaningful conclusions can be had with the results. Thus afterwards backlog percentage was exclusively used. It is a way to simply show a percentage of segments that are deficient out of the total segments observed, regardless of the severity of the deficiencies.

Another recent modification is the way pavement backlog is calculated. Inspections of state-maintained highway pavements in Wisconsin are done regularly in two-year cycles, with half of the state's pavements inspected in one year and the other half in the next year. In previous Compass reports, a two-year rolling average of all pavement segments conditions was used to calculate statewide conditions. It was determined in 2006 that the rolling average method doesn't accurately represent the actual condition at any one year and can dilute the condition of one or both halves of the state. Therefore, in the 2006 pavement report, the trends for pavement backlogs were separated into two separate trends, one for odd years and one for even years. This makes sure that the trends that are shown are from the exact same pools of pavement segments, and, therefore, the trends data will be more meaningful.

5. Clearly Communicate Results

Findings of a maintenance quality program will not have impact unless they are communicated well to the intended audience. Because of this, the Compass report itself has gone through many revisions and redesigns, making sure that each time it will be more successful in conveying the result of the data analysis accurately and thoroughly, but yet easily understood.

In 2004, trend data was added at the statewide level to show trends in the backlogs from one year to another. At the same time, grade curves were defined, and grades were given to the features based on their backlog percentage. Creating grade curves based on the critical importance of the feature helped make the critically important features stand out and get the attention that they need.

In 2006, additional tables were added to show the result of the analysis in the form of a report card. These tables contain backlog trends for all the features that are organized in groups according to their contribution, whether it is critical safety, safety, comfort, stewardship, or aesthetics. Table 3 shows a sample of a report card showing the three-year trends of grades for four features that contribute to "Critical Safety." With this new way of showing the grades based on their contributions, it is easier to identify which feature has more importance and, therefore, needs more attention.

Table 3. Report card of Compass report for critical safety features

Feature	2006	2005	2004	Element
Hazardous debris	B	B	B	Shoulders
Rutting	B	C	D	Traveled way, asphalt
Centerline markings	B	B	B	Traffic and safety devices
Regulatory/warning signs (emergency)	A	A	A	Traffic and safety devices

6. Prepare Process Documentation and Keep it Current

Data analysis usually involves a certain amount of data sorting, cross referencing, summary, and even cleaning. There may be certain assumptions that must be accommodated. For example, if the backlog for pavement cracking measures unfilled cracks and ignores filled cracks, then this must be dealt with in the data processing. The purpose of the process document is to provide guidelines for data analysis so that the information is consistent from year to year. The process document is extremely valuable to new staff on the program.

7. Institute a Quality Assurance Program

This is important if data are sampled in the field by human inspectors. The quality assurance (QA) program should verify that inspectors can consistently identify the features and consistently determine the maintenance condition. Measuring and recording the severity of deficiency for many features (length of cracking, height of drop-off) requires considerable effort on the part of inspectors. In the early years of Compass, the QA analysis showed that inspectors were not consistently measuring this information. The method has since been changed; now inspectors can simply check if a feature is backlogged according to predefined threshold standards.

The rating sheet itself may affect data quality. The QA for 2003 indicated a potential ambiguity in the rating sheet used for field review data collections. Inspectors could not agree on the existence of several features. Additionally, there was apparently some confusion in identifying whether or not a segment of highway is paved or unpaved. There was also an ambiguity in some of the features whether a number zero means that feature does not exist or whether it means it exists but it is in perfect condition. Closer examination of the collected data set and the rating sheet showed some things that can be improved to minimize the confusion. As an example, Figure 3 shows the changes made to the rating sheet in 2003 and 2006 regarding the yes-no checkbox for cracking distress on paved shoulders. In the 2006 rating sheet, that checkbox was removed, and the question was simplified, giving only a choice of inputting the number of linear feet of cracking found on the segment or zero if none is found. The figure also shows that the threshold for what is considered as backlogged has changed in 2006.

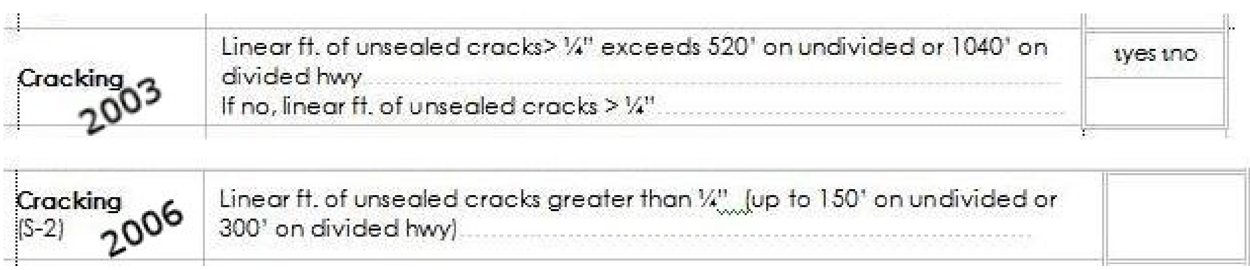

Figure 3. Rating sheet sample for cracking on paved shoulders, 2003 vs. 2006

Another effort to improve consistency is removing some problematic features from the program. With the more experienced rating teams, after doing field review inspection for several years and the recent development of automated data reporting process, overall data consistency has been significantly improved.

CONCLUSION

Many lessons were learned during the six years of implementation of the Compass program. Features were added, and the methodologies were refined. Every year feedback was taken, and adjustments were made to improve the report. It is very important to be responsive to the feedback received after each report. Evaluate and learn from the feedback, and use it to make the report better. Always be ready to adjust things if needed and modify the procedures if necessary. Evaluate the report each year and try to find the strengths and the weaknesses of the report and figure out how to improve it. A small and seemingly insignificant change like how the tables are shown in the report may help a lot in telling the story better. With all the continuing changes based on the lessons learned each year, the Compass program is constantly evolving, and with each step of this evolution the program is getting better.

Discussion of the Iowa Department of Transportation's Linear Referencing System

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ABSTRACT

The Iowa Department of Transportation (Iowa DOT) and its business partners have been working over five years to implement a version of the NCHRP 20-27 model using Intergraph's GeoMedia Professional, Bentley's LDMX, and Oracle. In spring 2006, version 1.0 of Iowa's Linear Referencing System (LRS) was successfully implemented.

This presentation will discuss the successes and challenges of Iowa's LRS implementation. The presenters will also review the technical architecture and describe how Iowa's LRS system will be maintained by the Iowa DOT.

Key words: Iowa—linear referencing system

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Privatization of Transportation Services in Developing Countries

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ABSTRACT

Developing countries are often characterized by insufficient revenues and weak governmental structure, which often lead to difficulties in provision of public facilities as well as a loss of regulatory control. This is particularly true of transportation infrastructure and services, important but often overlooked issues with contributions and impacts across the social and economic spectrum.

In developed and developing countries worldwide, privatization is seen as a means to transfer state-owned enterprises to the private sector and reduce government spending. This paper focuses on privatization of transportation services in developing countries. As market liberalization policies become more prevalent across these countries, privatization is often seen as a tool to shed government risk and boost the private sector. Capital constraints limit the effectiveness of the privatization of the transportation infrastructure; however, transportation services have been privatized with varying degrees of success. Paradoxically, the privatization of transportation services can often reassert government control through improved policies, regulation, and quality of service requirements in contracts.

First, relevant literature to the issue of privatization in general is summarized, using case studies to identify impacts and lessons learned through privatization in developing and developed countries. Next, a framework for evaluating a privatization project is developed. This framework is then applied to the privatization of transportation systems in developing countries, with a particular focus on bus rapid transit. The policy and planning recommendations gleaned from these experiences apply in developing and developed countries alike.

Key words: developing countries—privatization—transit

Design of Buchanan County, Iowa, Bridge Using Ultra High Performance Concrete and PI Girders

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ABSTRACT

Buchanan County, Iowa, was granted funding through the TEA-21 Innovative Bridge Construction Program (IBRC), managed by the Federal Highway Administration (FHWA), to construct a highway bridge using an optimized PI girder section with ultra high performance concrete (UHPC). UHPC is a relatively new structural material that is marketed by Lafarge, Inc. under the name Ductal. The PI girder section was developed to optimize the amount of material used in a girder, since currently the cost is relatively expensive. The Buchanan County project will be the first time the PI section has been used for a highway bridge in the United States. The girders will be pretensioned longitudinally and the deck will be post-tensioned transversely.

The Office of Bridges and Structures at the Iowa Department of Transportation and the Bridge Engineering Center at Iowa State University are currently working on the bridge design, which is a challenge since currently there is no design specification available in the United States for UHPC. The basis for the design will be conventional and finite element analysis, which is validated by prior laboratory testing at the FHWA's Turner-Fairbank Laboratory in Washington, DC. In addition, the paper will cover the design and analysis effort by the Office of Bridges and Structures at the Iowa Department of Transportation and the Bridge Engineering Center at Iowa State University.

Keywords: Ductal concrete—LaFarge North America—ultra high performance concrete—steel fibers—PI section

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INTRODUCTION

Developed in France during the 1990s, ultra high performance concrete (UHPC) has seen limited use in North America. UHPC consists of sand, cement, and silica fume in a dense, low water-cement ratio (0.15) mix. Compressive strengths of 18,000 psi to 30,000 psi can be achieved, depending on the curing process. The material has a low permeability and high durability. To improve ductility, steel or fiberglass fibers (approximately 2% by volume) are added, replacing the use of mild reinforcing steel. For this project, the patented mix Ductal developed by LaFarge North America was used.

Research has been conducted at Ohio University (Lubbers 2003), Michigan Technological University, Iowa State University, and Virginia Polytechnic Institute and State University to help better understand UHPC properties. Testing is ongoing at the FHWA's Turner-Fairbank Laboratory near Washington, DC, on the optimized prestressed bridge girder cross section. In addition, an IBRC project by the Virginia Department of Transportation using UHPC in prestressed beams for a highway bridge is underway.

PROJECT BACKGROUND

Iowa was first introduced to UHPC with a bridge project in Wapello County, completed 2006. See Figures 1, 2, and 3. Wapello County and the Iowa Department of Transportation (Iowa DOT) were granted funding through the TEA-21 Innovative Bridge Construction Program (IBRC). The UHPC mix was used in three 110 ft. long prestressed concrete bulb tees for a bridge replacement project south of Ottumwa, Iowa.

Figure 1. Casting of 110 ft. UHPC beam

Figure 2. UHPC beams prior to shipping to bridge site

Figure 3. Completed bridge

As a continuation of this work, Buchanan County and the Iowa DOT were granted funding in 2005 through the TEA-21 IBRC for an additional project using UHPC. The UHPC mix will be used in three 51 ft. optimized PI sections developed by the FHWA's Turner-Fairbank Laboratory and the Massachusetts Institute of Technology (MIT).

BRIDGE DESCRIPTION

The replacement bridge project is located on a county road (136th Street) over the east branch of Buffalo Creek in northeast, Buchanan County, Iowa. See Figures 4 and 5. The bridge will be 24 ft. 3 in. wide by 112 ft. 4 in. long. The center span will be 51 ft. 2 in. from center to center of the pier caps, and plain neoprene bearing seats will be provided for the 50 ft. 0 in. simple span PI section. See Figure 7 for cross section details. The beam ends will be encased in normal strength cast-in-place concrete diaphragms at the bridge site. End spans will be cast-in-place reinforced concrete slabs with integral abutments and pier caps. See Figure 8.

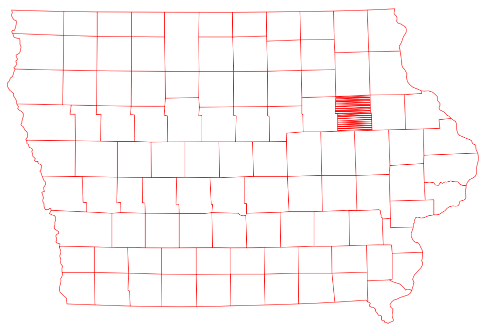

Figure 4. Location of Buchanan County

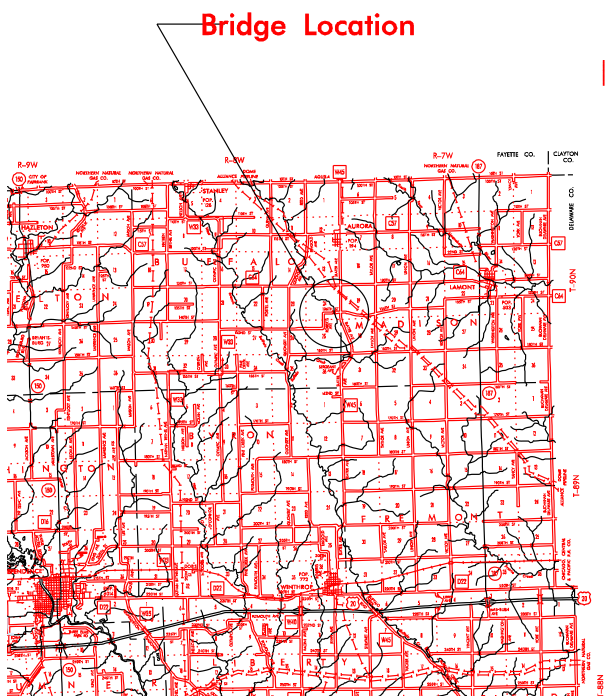

Figure 5. Bridge location

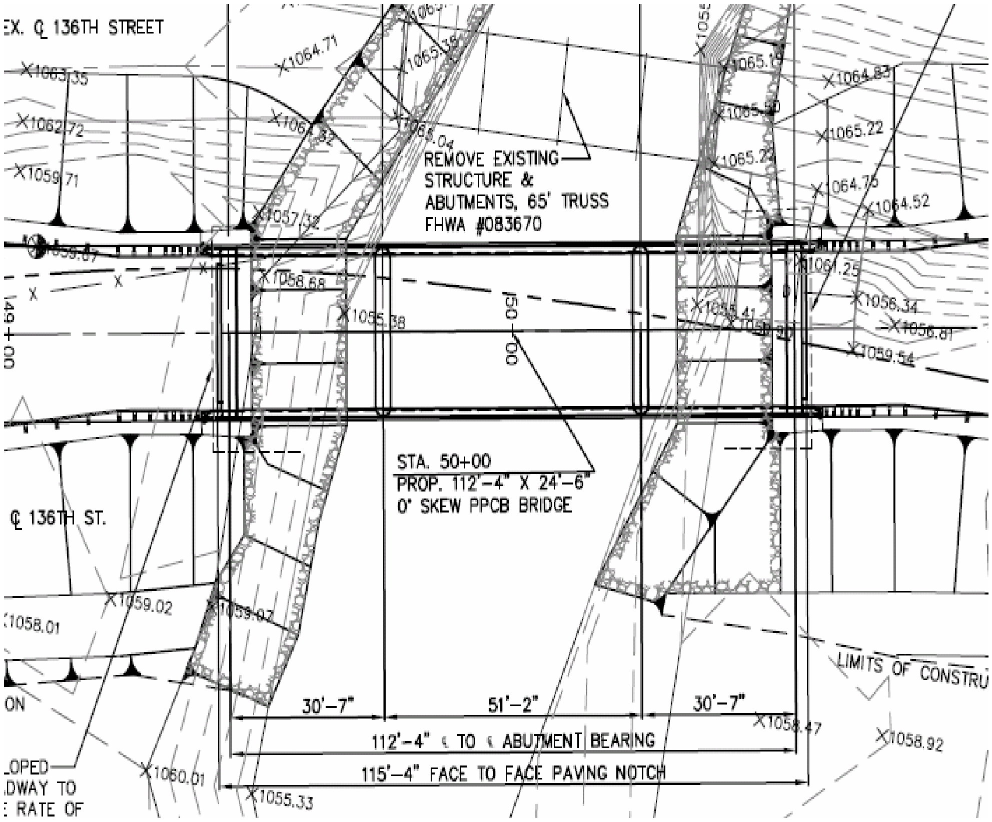

Figure 6. Situation plan

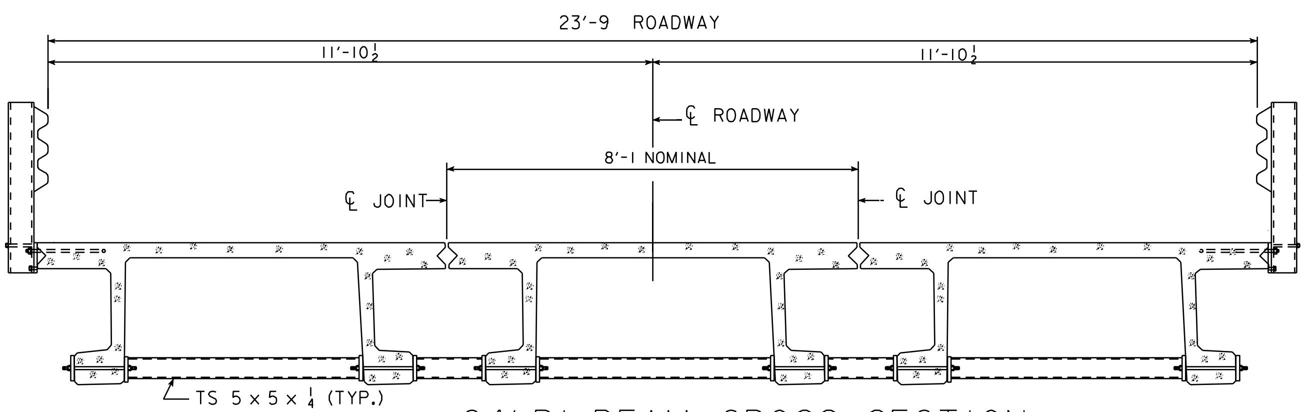

Figure 7. Proposed bridge cross section

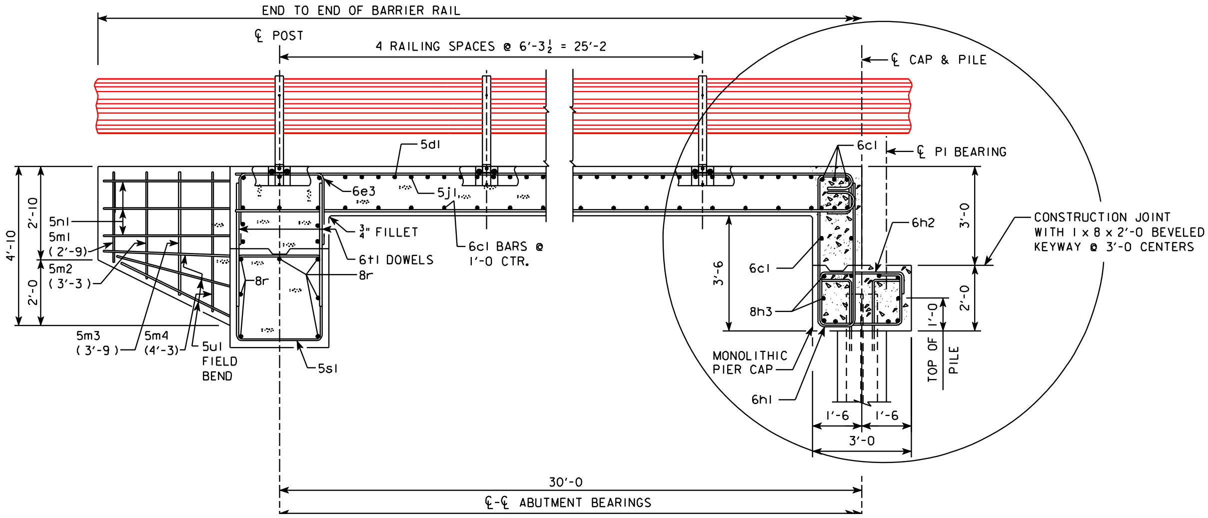

Figure 8. End span details

DESIGN

Materials Properties and Design Stresses

Material properties and design stresses of the Ductal mix were based on experience with the Wapello County project, FHWA testing, and recommendations by LaFarge. Values are shown below:

Modulus of elasticity at release	5,800	psi
Modulus of elasticity final	8,000	psi
Design compressive strength at release	14,500	psi
Design compressive strength final	24,000	psi
Tensile strength	1,200	psi
Allowable compressive release stresses 0.6 (14,500 psi)	8,700	psi
Allowable compressive stress at service 0.6 (24,000 psi)	14,400	psi
Allowable tensile stress at service 0.7 (1200 psi)	840	psi

Design Guidelines

For the design, the team took advantage of the design work that was done for the bridge project in Wapello County, along with the testing that was performed by Turner-Fairbank Laboratory on UHPC concrete and the PI section. In addition, research reports and guide specifications listed below were used, as well as discussions with Ben Graybeal and Vic Perry:

1. Research and design recommendations from Dr. Ulm of MIT (Ulm 2004)
2. Design Guidelines for RPC Prestressed Concrete Beams, University of New South Wales (Gowripalan and Gilbert 2000)
3. Structural Behavior of Ultra High Performance Concrete Prestressed I-Girders, Publication No. FHWA-HRT-06-115 (Graybeal 2006a)
4. Material Property Characterization of Ultra High Performance Concrete, Publication No. FHWA-HRT-06-103 (Graybeal 2006b)

Beam Design

The design started with the PI section (see Figure 9) that was developed by the FHWA's Turner-Fairbank Laboratory and MIT. During testing of the section at Turner-Fairbank, some problems were found with the initial shape. These problems were addressed during the design process, and changes to the section are planned. The areas that were addressed during the design process were as follows:

1. Transverse strength of deck
2. Live load distribution between beams
3. Fiber distribution in web areas

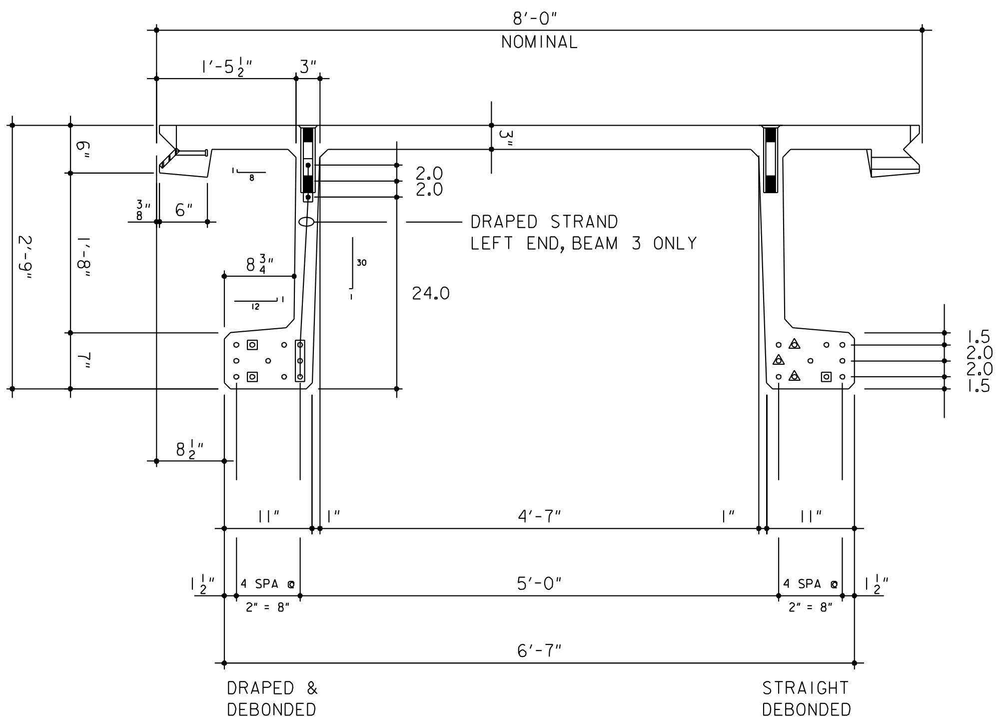

Figure 9. Initial PI section

Transverse Strength of Deck

A large amount of the design time was spent on improving the transverse strength of the deck system. Based on testing at Turner-Fairbank (see Figures 10 and 11) the 3 in. deck did not meet the requirements for a service loading of 16 kip wheel load with 33% impact.

To strengthen the deck, a number of options were studied using a maximum tensile stress for a service design of 0.84 ksi. To complicate the process, options considered would need to use or modify the existing forms.

Figure 10. Transverse deck test

Figure 11. PI test bridge

Design options that were considered for strengthening the deck were as follows:

1. Adding ribs under the deck with or without mild reinforcing or post-tensioning
2. Adding structural steel diaphragm after casting
3. Thickening the deck with or without reinforcing

These design options were analyzed in the Office of Bridges and Structures at the Iowa DOT and verified by Iowa State University using finite element analysis (FEA). See Figure 12. After all options were considered, a decision was made to use a constant 4 in. deck with transverse post-tensioning. This decision was based on keeping the changes as simple as possible and limiting the cost of modifying the beam forms to keep the research budget within allowable limits. A final FEA analysis of the bridge is ongoing and has not been completed at this time.

A decision has not yet been made on the type of post-tensioning to use. The team is considering 5/8 in. high strength rods or 0.6 in. diameter strands at approximately 18 in. spacing in a grouted plastic post-tensioning duct. Additional discussion is still ongoing whether to post-tension each individual section in the precast plant to eliminate field post-tensioning or to post-tension the entire section at the site. Advantages and disadvantages are being discussed for both methods.

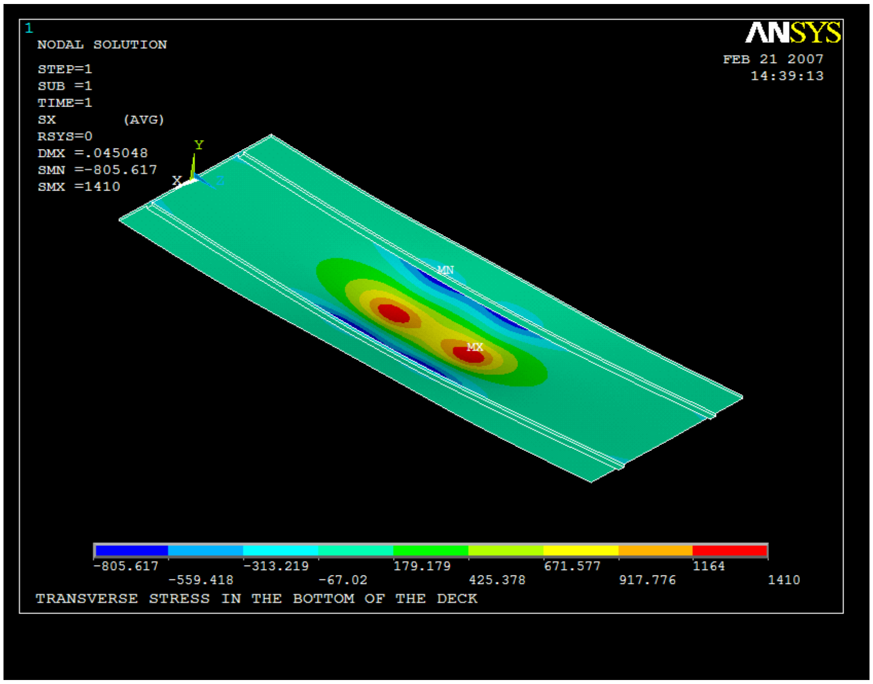

Figure 12. Finite element analysis

Live Load Distribution between Beams

Testing revealed poor distribution between the beam sections, possibly due to the flexibility of the beams, the connection between the beam sections, and the lack of diaphragms. To improve distribution, steel diaphragms were added to the bottom of the section. See Figure 7 for details. In addition, there has been discussion for providing continuous post-tensioning across the joints between the sections in the deck. At the writing of this report, no final decision had been made.

Fiber Distribution in Web Area

During the testing of the original section, it was found that testing wires were disrupting the proper flow of fibers through the web and may have caused planes of weakness in the web sections where the shear failures took place. To resolve this problem, the webs were thickened by 1/2 in. to improve the flow of the steel fiber in case testing wires are cast in the web.

Final Cross Section

Based on the work completed to date, the final PI cross section is shown in Figure 13. Additional changes may be made as final details are completed.

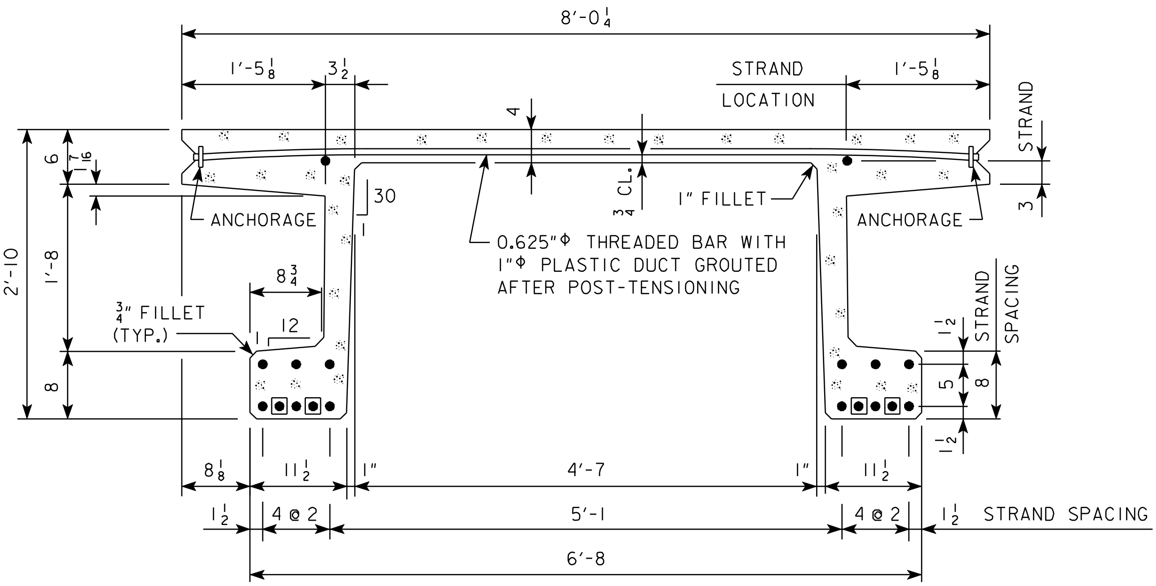

Figure 13. Modified PI section

CONSTRUCTION SCHEDULE

Details are currently being finalized for the beams, with plans to let the casting of the beam for contract this fall. If bids are reasonable, casting will be in late fall or winter. Letting of the bridge project will take place after the beams are successively cast, with construction to begin spring 2008.

CONCLUSION

This brief report covered the design work that has been completed to date using the UHPC PI section for a bridge project in Buchanan County, Iowa. The design work to date is adapting the optimized test section that was developed by the FHWA and MIT into an actual bridge project. Based on the results of testing by the FHWA, changes were made to the optimized PI section. Evaluation of the final modified PI section bridge will take place during construction, and a load test will be conducted once the bridge is complete. If successful, the next step will be the use of this revised section on a primary highway with higher traffic and loading. By using the optimized section, the people involved in this project hope to take better advantage of the properties of this unique material and help reduce the cost of using it in future projects.

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Centerline Curbing Treatment at Railroad Crossings for Improved Safety

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ABSTRACT

The main objective of this paper is to report on unsafe actions of motor vehicle drivers at railroad-highway grade crossings and drivers' response to installation of a centerline barrier that prevented them from driving around the gates. A comparison of unsafe driver actions in the pre- and post-barrier periods provided an idea about the effectiveness of the barrier in reducing unsafe driver actions. In this study, the railroad grade crossing at "M" Street in the city of Fremont, Nebraska, was studied continuously for approximately four months with the help of a night vision camera and a digital video recorder. Motor vehicle driver actions were observed whenever the gates were down, and instances of unsafe actions (e.g., gate rushing, U-turns, etc.) noted. In an effort to reduce gate rushing and other unsafe behaviors, a rubber and plastic barrier along roadway centerline was installed in two phases. The first installation phase limited access to crossing the centerline, starting several feet from the crossing, while the second installation phase closed all access to crossing the centerline at the gates. The actions of motor vehicle drivers were monitored during all three periods, and comparisons were made using appropriate statistical tools. The results provide insights into the effectiveness of such barriers. The conclusion reached from this research is that installation of the barrier was successful in reducing unsafe driver actions at the studied railroad-highway grade crossing.

Key words: driver behavior—railroad crossing—safety—video surveillance

PROBLEM STATEMENT

The safety of a railroad-highway grade crossing to an extent depends on the actions of motor vehicle drivers using the crossing. Unsafe maneuvers by drivers immediately before, during, and immediately after train crossings can lead to crashes with trains, as well as non-train-related crashes. Examples of unsafe actions include rushing closing or closed gates to beat an oncoming train and backing up and making U-turns that can potentially result in non-train-related crashes. This research was sponsored by the Nebraska Department of Roads (NDOR) to look into unsafe driver actions at railroad-highway grade crossings and determine how to eliminate or reduce them by using centerline barriers (also known as median dividers and centerline curbing).

RESEARCH OBJECTIVES

The objectives of this research were to study unsafe motor vehicle driver actions in close proximity to railroad-highway grade crossings and to evaluate the effectiveness of centerline barriers in reducing those unsafe driver actions. The effectiveness was judged by comparing frequencies of observed unsafe driver actions in the pre- and post-barrier periods. This research utilized a rubber and plastic barrier, installed at the study site, from Qwick Kurb, Inc. (Ruskin, Florida), as shown in Figure 1.

Figure 1. Flexible rubber and plastic barrier from Qwick Kurb, Inc.

RESEARCH METHODOLOGY

The railroad grade crossing at “M” Street in Fremont, Nebraska, (Figure 2) was selected due to accessibility from Lincoln, Nebraska (researchers’ base), adequacy of train and roadway traffic, suitability for video data collection, and cooperation from the local administration.

Figure 2. Grade crossing at M Street in Fremont, NE

The subject crossing has two sets of railroad tracks and is equipped with dual quadrant gates. The researchers collected data on driver actions at the grade crossing by mounting a day- and night-vision camera and a digital video recorder (DVR) with the cooperation of the City of Fremont administration. The camera was mounted on a utility pole on the north side of the crossing (Figure 3). A DVR storage box was mounted on the utility pole that could be accessed at the site, and power for these devices was provided by the City of Fremont. Although motion-activated recording was possible with the system, the DVR was set to record continuously due to frequent false triggering of the motion-activated recording.

Unsafe actions of motor vehicle drivers were noted in the research office by viewing the recorded video. These actions included gate rushing to beat an oncoming train, driving between train occurrences or after a train passed while the gates were still down, U-turns, wrong side entry (only in the post-barrier install period), vehicles backing up, and taking an alternate route. Additional data items collected included date and time of observation, weather (snow, rain, fog, etc.), number of crossing trains and whether trains stopped on the tracks, roadway vehicular traffic during the train crossing and vehicular queue at gate opening, and duration of gate closure (measured in minutes and obtained from the gate’s down and up times from time-stamped video).

Figure 3. Night vision camera (top) and DVR storage box mounted on a utility pole

An observation in this study was defined as an event when the gates came down with roadway vehicular traffic present at the crossing during the gate closure period. Motor vehicle driver actions were observed whenever the gates came down, and instances of unsafe actions were noted. During the course of this study, there were a few instances when the gates came down while no train was approaching but roadway vehicular traffic was present. Such instances qualified for the observation definition and were recorded in the data as gate malfunctions. Gate closures with no motor vehicle traffic present were discarded because there was no potential conflict present between train and roadway vehicular traffic.

The study period included three time intervals, totaling 5,126 observations. The three time intervals were a pre-barrier time, a limited installed barrier time, and a fully installed barrier time. The pre-barrier time interval was defined as the period when no barrier was installed at the crossing. The limited installed barrier time interval was the period in which barriers were installed on the two sides of the crossing, but they did not extend fully to the gates. The reason for not fully extending the barriers to the gates was to ensure enough space for the railroad company's trucks that frequently ply along the railroad right-of-way. However, during this time period, the authors found that not fully extending the barriers to the gates provided roadway traffic enough space to rush the gates, thus limiting the barriers' effectiveness. Therefore, barriers on both sides of the railroad tracks were fully extended up to the gates. The open space between the gates was utilized by the railroad company's trucks when the gates were not closed for plying along the railroad right-of-way. Thus, the fully installed barrier interval refers to that period when the barriers were fully extended to the gates. Of the total 5,126 observations, 2,989 were made during the pre-barrier period, 892 were made during the limited installed barrier period, and 1,245 were made during the fully installed barrier period. Table 1 shows simple frequency counts of different types of driver actions in the three periods.

Table 1. Simple frequency counts of driver actions

Driver action	Pre-barrier period	% of total	Limited installed barrier period	% of total	Fully installed barrier period	% of total
Number of gate rushes to beat train	539	66.71	112	13.86	157	19.43
Number of drivers taking alternate route	1,665	69.61	257	10.74	470	19.65
Number of U-turns	1,235	85.06	85	5.85	132	9.09
Number of drivers that backed up	220	34.98	96	15.26	313	49.76
Number of between/after train	47	83.93	0	0	9	16.07
Number of wrong side entries	0	0	72	76.6	22	23.4

KEY FINDINGS

The limited installed barrier period and fully installed barrier period were combined to form one post-barrier install period for this analysis. Using a combined post-barrier install period made it possible to compare the pre- and post-install periods to assess the effectiveness of the barrier in reducing unsafe driver actions. Table 2 presents t-tests comparing the means of different types of driver actions in the pre- and post-barrier periods. The equation used is as follows (Devore 1991):

$$t' = \frac{(\bar{x} - \bar{y})}{\sqrt{\left(s_1^2 / m\right) + \left(s_2^2 / n\right)}} \tag{1}$$

where

$\bar{x}$ *and* $\bar{y}$ are the mean number of unsafe driver actions in the pre- and post-barrier install periods

s_1^2 *and* s_2^2 represent the variances

m and n represent the sample sizes pertaining to the two time periods

Table 2. Comparison of pre- and post-barrier unsafe driver actions

	Pre-barrier period		Post-barrier period		
Driver action	**Mean**	**Std. dev.**	**Mean**	**Std. dev.**	**t'-value**
Number of gate rushes to beat train	0.180	0.684	0.126	0.369	-3.341*
Number of drivers taking alternate route	0.557	1.303	0.340	0.732	-6.958*
Number of U-turns	0.413	0.891	0.102	0.332	-15.408*
Number of drivers that backed up	0.074	0.290	0.191	0.521	10.333*
Number of wrong side entries	0.000	0.000	0.044	0.260	9.268*
Number of times crossed closed gates between successive trains or immediately after train passage while gates still closed	0.016	0.407	0.004	0.084	-1.288

* Statistically significant at 95% confidence level; a t'-value of >= 1.96 or <= -1.96 indicates statistical significance at the 95% confidence level; positive t'-values show that mean occurrences were greater during the pre-barrier period.

The null hypothesis is that the mean numbers of unsafe driver actions in the two time periods are the same, while the alternative hypothesis is that the means are different. The rejection region for the null hypothesis is $t' \geq t_{\alpha/2, \nu}$ or $t' \leq -t_{\alpha/2, \nu}$, where α represents the probability of making a Type I error (rejecting the null hypothesis when it is actually true) and ν represents the degrees of freedom. A t'-value of >= 1.96 or <= -1.96 in Table 2 is indicative of statistical significance at the 95% confidence level (i.e., $\alpha = 0.05$). Positive t'-values that are statistically significant show that mean occurrences of unsafe driver actions were greater in the before period. That is, the t-tests show that the number of U-turns, gate rushes, and alternate routes reduced after installation of the barrier, while the number of drivers backing up and driving on the wrong side of the barrier increased. Between the two time periods, the mean numbers of times that drivers crossed closed gates between successive trains or immediately after train passage while the gates were still closed were not found statistically different from each other.

These simple comparisons of means, while providing useful information, do not account for other factors, such as duration of gate closure, number of trains crossing, weather, motor vehicle traffic during train crossing, etc. One would expect these factors to affect driver actions at railroad crossings. To account for the effects of such factors, the study utilized a model suitable for count data: nonnegative integer values, e.g., number of undesirable driver actions. Analysis of count data usually requires application of Poisson or negative binomial models. For details of count data models, the reader is referred to Washington et al., (2003). Reported below are the results of a negative binomial model estimated for the total number of unsafe driver actions.

Negative Binomial Model Results for Total Number of Unsafe Driver Actions

A negative binomial model was estimated for the total number of unsafe driver actions, which was the sum of numbers of U-turns, wrong side entries, driving between/after trains, gate rushes, and vehicle backups. The model is reported in Table 3, and it provides information on elements that are prominent in unsafe driver actions at railroad grade crossings. A positive estimated coefficient in the model indicates that unsafe driver actions increase with increasing values of the dependent variable, and a negative estimated coefficient shows that unsafe driver actions decrease with increasing values of the dependent variable.

Table 3. Negative binomial model for total number of unsafe driver actions

Dependent variable	Est. coeff.	Std. error	t'-value
Duration of gate closure (in minutes)	0.034	0.0016	21.073
Gate up and down with no train (yes=1, no=0)	1.307	0.1324	9.869
Number of crossing trains at the same time	0.191	0.0423	4.513
Train stopped on crossing (yes=1, no=0)	0.642	0.0449	14.303
Gate malfunction (yes=1, no=0)	1.715	0.2898	5.917
Period dummy (pre-install=1, post-install=0)	0.182	0.0459	3.972
Constant	-2.221	0.0792	-28.054
Alpha	0.431	0.0316	12.626

Summary of statistical measures

Measure	Value
Number of observations	5126
Log likelihood	-4885.321
Restricted log likelihood	-5096.178
Chi-squared	421.71
Rho-squared	0.041

Modeling results in Table 3 show that the total number of unsafe driver actions increase with longer duration of gate closure, increase if the gates came down and went up without train presence, increase if multiple trains are crossing at the same time, increase when a train stops on the tracks, and increase if the gate malfunctions. The estimated coefficient for the pre-barrier time period dummy variable is negative and statistically significant, showing that the total number of unsafe driver actions decreased in the post-barrier period (coding: pre-install =1, post-install =0). Statistical significance of the alpha value in the model shows that data are overdispersed, and use of the negative binomial model in place of the Poisson is appropriate. No statistical evidence was found of the effect of weather (rain, snow, fog, etc.), amount of vehicular traffic present, and day of week on the total number of unsafe driver actions. For parsimony, these variables were excluded from the model specification.

Summary of Data Analysis Results

Unsafe driver actions showed a decrease in the two post-barrier time periods, except drivers backing up. This can be attributed to the installation of the barrier, which prevented drivers from performing unsafe actions. The study site had a number of drivers using the wrong side of the road after installation of the barrier, either for gate rush or to take an alternate route. The most significant finding from this study is that the total number of undesirable driver actions significantly decreased after installation of the barrier.

CONCLUSIONS

This study investigated unsafe actions of drivers at railroad-highway grade crossings and compared the effects of a rubber and plastic barrier that was installed to study changes in driver actions. Analysis of the collected data revealed the total number of unsafe driver actions decreased when a barrier that prevented drivers from going around the gate was installed. The conclusion reached is that installation of the centerline barrier was successful in reducing undesirable driver actions in the vicinity of at-grade railroad crossings.

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Using Driving Simulators to Train Snowplow Operators: The Arizona Experience

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ABSTRACT

Snowplow driver training is an attractive focus for simulator-based training, since these drivers must operate equipment valued up to $200,000 in stressful shifts in blinding snowstorms. In Arizona, the infrequency of snowstorms and the heavy turnover of drivers make simulator-based training particularly attractive.

In the 2005–2006 snow season, the Arizona Department of Transportation (ADOT) used a fixed-base L-3 Communications TranSim VSIII simulator in the Globe, Arizona, maintenance district to offer two training programs: one on enhancing drivers' sensitivity to potential hazards, and the other on fuel management and shifting techniques. In response to a request from ADOT's Transportation Research Center, a research team from Arizona State University assessed the effectiveness of the simulator-based training in addressing these objectives.

The assessment included surveys of driver participants, focus groups of driver participants and district field supervisors, and a review of reports generated by the simulator, as well as an assessment of relevant crash and claims records. Researchers noted the enthusiasm for the driver awareness program, particularly among recent hires. The fuel management program interested experienced as well as less experienced drivers who could see immediate application, and it also appeared promising in terms of improving fleet fuel economy. The experience with simulator training in Arizona left researchers and ADOT personnel optimistic about the potential for driving simulators as an integral part of a comprehensive driver training program.

Key words: Arizona—driving simulators—snowplow driver training

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INTRODUCTION

Driving simulators have been widely used for human factors research and automobile driver training and retraining for more than 30 years. Research simulators are often used for human factors and cognitive psychology experiments to study driving behavior (for examples, see Kemeny and Panerai 2003; Reed and Green 1999; Sidaway and Fairweather 1996), while training simulators are frequently used by public and private agencies to teach and evaluate driving skills (for examples, see Emery et al. 1999; Kihl et al. 2006; Strayer and Drews 2003; Strayer, Drews, and Burns 2004; Vance et al. 2002). Driving simulators offer several advantages over real-world driving. Safety is a primary advantage, as simulators can be used to expose drivers to driving conditions too dangerous to consider for real-world driving (Liu, Miyazaki, and Watson 1999; Reed and Green 1999). As a training tool, simulators allow trainees to practice driving and to develop confidence before taking a road test (Liu et al. 1999). Most simulators also have the ability to record and play back training sessions, allowing evaluations to be objectively assessed.

Driving simulators may be categorized as either fixed-base or motion-based. Fixed-base models range from simple, desktop computer models (for example, see Pierowicz et al. 2002), to those utilizing a head-mounted display with head tracking technology (Liu et al. 1999). Some units include partial (Ross-Flanigan 2002) or full vehicle cockpits (for examples, see Pierowicz et al. 2002; Roenker, Cissell, and Ball 2003). Motion-based simulators are generally more sophisticated and feature motion cues that mimic the roll, pitch, and yaw of actual vehicle dynamics. The National Advanced Driving Simulator, located at the University of Iowa, is one of the most sophisticated motion-based driving simulators (Kuhl et al. 1995) and has been used for both research and training purposes.

As simulator technology has improved and prices have dropped, more private corporations and public agencies, including departments of transportation (DOTs), are starting to use driving simulators as training tools. Snowplow training can be attractive for DOTs that are weighing the benefits of incorporating a driving simulator into their existing training programs. Snowplow drivers must operate equipment valued at up to $200,000 in long, stressful shifts in blinding snowstorms and demanding traffic conditions. Recognizing these concerns, DOTs in Pennsylvania (Vance et al. 2002), Utah (Strayer et al. 2004), Minnesota have used simulators to train snowplow operators. The Iowa DOT also introduced a simulator training program for snowplow operators in 2005. All are optimistic about the potential of simulator training in enhancing their driver training programs.

SIMULATOR-BASED TRAINING IN ARIZONA

The Arizona Department of Transportation (ADOT) began using driving simulators to supplement its snowplow operator training program in the 2004–2005 snow season. The primary objective was to increase safety both for drivers and the motorists with whom they share the road. It was thought that the simulator could be used to improve driver confidence and thereby driver retention, both of which would potentially improve overall safety.

A relatively short simulator-based training program was offered to drivers in five districts by an outside vendor in a traveling trailer during the 2004–2005 snow season. Then, at the start of the snow season in 2005–2006, ADOT purchased an L-3 Communications TranSim VSIII simulator and housed it at the Globe maintenance district, where four-hour training programs were offered to all drivers in the district by local trainers. In the 2006–2007 snow season, two more L-3 simulators were acquired and based at the Flagstaff and Holbrook maintenance districts. In summer 2007, ADOT purchased an additional L-3 simulator to be based in the Safford maintenance district. Figure 1 shows the Arizona engineering districts.

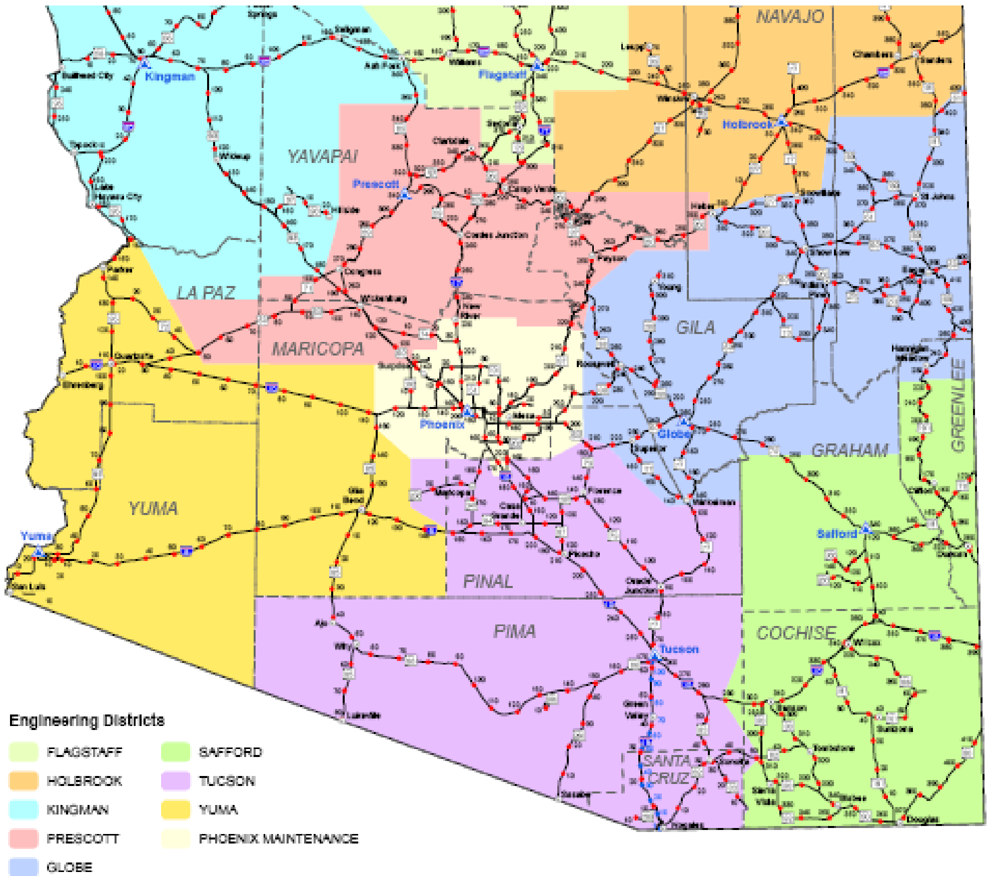

Figure 1. Arizona Department of Transportation maintenance districts

The TranSim VSIII Driving Simulator is a computer-controlled fixed-base simulator. The main computer system that operates the dashboard controls is located in the TranSim chassis. Image generators provide high-resolution graphics that are displayed on three plasma displays mounted on the chassis, giving the driver a 180-degree view of the simulated driving scenarios (Figure 2).

Although fixed-base simulators have the obvious advantage of relatively lower cost, their lack of motion cues may alter “the perceived motion variables that serve as inputs to [one’s driving] strategy” (Reymond et al. 2001). This becomes especially important during the low-friction conditions associated with snowplow operation. In this case, even small motion cues (e.g., 1 to 2 in., or 25 to 51 mm) make a significant difference in how realistic the simulation experience feels to users. A number of driver trainees have commented about what they call a lack of realism in the simulator training program.

The training programs used in Arizona include both lecture and simulator scenario components. They were developed initially by L-3 and were modified to reflect ADOT policies more directly. The curriculum is punctuated by real-word examples that are introduced by experienced Globe district snowplow drivers who served as trainers.

Figure 2. The MPRI L-3 TranSim VS III Driving Simulator

The primary snowplow driver simulator training course focuses on making drivers alert to unexpected hazards that can intrude into their plowing experience and adds safety challenges such as errant motorists, pedestrian crossings, falling rocks, animal crossings, and icy roadways. The course emphasizes a strategy developed by L-3, SIPDE (Search, Identify, Predict, Decide, and Execute), that is intended to increase driver awareness. Examples and modifications were added by the Globe district. This training program exposes driver trainees to three increasingly challenging 15-minute scenarios that illustrate key points emphasized in the accompanying classroom program. It was first offered in Globe in fall 2005.

A second driving techniques-based simulator training course, offered in March and April 2006, focused on teaching drivers how to reduce fuel consumption and minimize equipment repairs. Here, the classroom training is interspersed with simulator-based opportunities to demonstrate the application of the various driving techniques introduced in the course. In the 2006–2007 snow season, the Globe district trainers combine these two courses and offered it in one long session, thereby minimizing the time drivers are away from their regular duties. That course is still being refined.

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ADOT’s Arizona Transportation Research Center engaged an interdisciplinary research team from Arizona State University (ASU) to evaluate the effectiveness of the emerging simulator-based snowplow driver training. The evaluation included both a review of available data and an assessment of perspectives of driver participants and their field supervisors. The objective of the assessment was to determine whether the simulator could assist drivers in modifying their driving behavior. The focus in this paper is primarily on the 2005–2006 training program in the Globe maintenance district.

STUDY PARAMETERS

The training program, offered by experienced Globe snowplow drivers, was intended to alert drivers to the unexpected. Since snowplow drivers frequently have limited time to make decisions and act, it is most important that they remain alert, scanning the setting for potential hazards, identifying them, and trying to predict how a moving hazard will act. Drivers were taught to search their surroundings, identify potential hazards, and predict the path of moving hazards, such as wildlife crossing highways, speeding motorists, or pedestrians. They were then to decide what to do and, finally, execute that decision.

All 61 snowplow drivers within the Globe maintenance district participated in this driver awareness and safety program in October and November 2005. They were divided into groups of four. Each group was given extensive classroom training that was interspersed with simulator seat time. The simulator portion of the training involved driving three different simulator scenarios, each designed to allow the trainees to apply specific aspects of what they had learned in the classroom. The scenarios included a freeway, a two-lane rural mountain road, and a small city. The trainers had the capability to make each of these scenarios increasingly complex by adding whiteout conditions, iced-over windshields, and/or night conditions. Between the simulator drives, the participants discussed driving issues and reviewed the scenarios. A computer-generated report noted traffic violations associated with each driver's simulated drive.

The ASU research team observed several of the four-hour class sessions, including both the training lectures and the driver simulation sessions. At the end of the snow season in April 2006, the research team distributed a written survey soliciting drivers' opinions about the simulator training program after they had the opportunity to apply what they had learned in the real world.

In the spring of 2006, additional simulator training was conducted in the Globe district. The same drivers who were trained in the fall were given instruction on techniques that could improve fuel economy. In this course, initially developed by L-3 communications, the focus was specifically on shifting gears (using the gear shift, clutch, and accelerator), rather than on the overall driving experience (as was the case with all previous simulator training sessions). Trainees received a combination of stand-up lecture training, computer-based training (CBT), and simulator "seat time." The stand-up training covered the basic principles of shifting for fuel economy, while the CBT reinforced these points with one-on-one modules in which the trainees received instruction (via headphones) and responded to questions posted on a desktop computer screen. In the simulator, drivers were given approximately 15–20 minutes of practice time, during which they were coached by trainers. In a timed pre-test, each driver drove the simulator along the same simulated road scenario. At the end of the program, they all drove a timed post-test over the same simulated roadway. Fuel efficiency reports for both tests were compared by the research team.

In early June, the ASU team held four focus groups with a total of 24 drivers (with a range of experience levels) who had completed both the snowplow driver awareness program and the fuel management simulator program. One additional focus group involved the 14 supervisors representing all the maintenance organizational subsections in the Globe district.

In addition to these qualitative sessions, the research team also reviewed computer-generated reports related to individual driver performances in both simulator-based training programs. The reports generated by the snowplow program noted violations incurred by each driver while driving the scenarios in the simulator. Violations included such actions as sliding while trying to accelerate and encroaching on another lane, as well as actual collisions. The reports generated for the fuel management program listed fuel consumption records for each driver and driving-related issues, such as riding the brake or clutch and shifting improperly. The research team reviewed these reports to note trends and to suggest areas for additional simulator training. In addition, the team reviewed accident and claims records for the 2005–2006 snow season in comparison with similar records for the five years prior to simulator-based training.

Michon's Driving Model

To better understand the driving skills important to snowplow operators, the ASU research team rode in plow trucks and interviewed operators. From this information, the various operator activities were sorted into five major categories: Inspecting, Communicating, Driving, Plowing, and Spreading (some of these activities would be different for different types of heavy equipment). Michon's (1985) driving model was used as a framework onto which this activity model could be overlaid (see Table 1). The description of

Michon's driving model provided by Wickens, Gordon, and Liu (1998) is especially useful and is worth quoting fully:

> Three levels of activity describe the complex set of tasks that comprise driving—strategic, tactical, and control... Strategic tasks focus on the purpose of the trip and the driver's overall goals; many of these tasks occur before we even get into the car. Strategic tasks include the general process of deciding where to go, when to go, and how to get there... Tactical tasks focus on the choice of maneuvers and immediate goals in getting to a destination. They include speed selection, the decision to pass another vehicle, and the choice of lanes... Control tasks focus on the moment-to-moment operation of the vehicle. These tasks include maintaining a desired speed, keeping the desired distance from the car ahead, and keeping the car in the lane. (p. 438)

The awareness curriculum addresses primarily tactical driving skills related to driving and communicating. The tactical skills are associated with driving activities that include avoiding other drivers and objects and monitoring vehicle speed, all issues that are emphasized strongly in the SIPDE curriculum. While the awareness program was broadly based, the fuel management training program was narrowly focused, emphasizing proper gear shifting (and related clutch usage). As shown in Table 1, gear shifting is comprised of control-level driving skills corresponding to the driving activity of snowplow operation. The TranSim III simulator is capable of supporting either type of training program, but within limits. In particular, the training of control-level skills requires very specific real-world controls (and, other than shifting, few snowplow-related controls are incorporated into the current TranSim III simulator).

The ADOT Snowplow Operator Training Program

Snowplow operators must focus not only on driving safely in severe winter conditions; they must also operate a plow, monitor instruments and radios, and look through icy windshields for road obstructions, traffic, and the road's edge. The challenges faced by all snowplow operators are more obvious in parts of Arizona, where major snow events are infrequent and drivers have limited opportunity to gain or maintain proficiency between these events. In the last five years, snowfall in the Globe district, for example, varied between 38 in. (965 mm) and 80 in. (2,032 mm) per snow season. In 2005–2006, almost all of the snowfall (53.3 in./1,354 mm) occurred during one week in March. Turnover rates of 25% (and higher) among drivers in the district mean that some drivers experience only limited on-the-job training before they drive a plow in a heavy snowstorm.

In addition, the geography of the Globe maintenance district offers numerous challenges to snowplow operators. The northern portion is a high plateau and the southern portion includes two small mining communities wedged up against the mountains and a rural area around the lake created by Roosevelt Dam. In between is a scenic highway characterized by a narrow roadway and many switchbacks with limited shoulders. Guardrails offer drivers the only protection from driving off a cliff in whiteout conditions. Over the 90 miles (145 km) from Globe in the south to the high plateau in the north, the road ascends about 3,000 ft. (915 m). See Figure 3.

Table 1. Snowplow operator activities and Michon's driver behavior model

Activities of snowplow operators	Levels of driving skills		
	Strategic (Planning)	**Tactical (Maneuvering)**	**Control (Operational)**
Inspecting (pre- and post-trip and while plowing)	N/A	N/A	• Vehicle (hydraulic lines, tires, lights) • Snow removal equipment (wear bits, de-icing material, etc.)
Communicating	• Broad ADOT policies (e.g., public safety) • District policies • Receive orders from Snow Desk	• Contact other ADOT drivers • Assist other drivers (ADOT, DPS, public)	• Adjust radio volume • Locate and key radio microphone
Driving	N/A	• Navigation-Avoidance (other drivers, objects) • Monitor speed (by ear)	• Navigation-Aim (apply brake and steering inputs, etc.) • Shift gears; use clutch. • Visibility (heater, defroster, mirrors)
Plowing	N/A	• Aiming (height, angle—function of vehicle speed) • Avoidance (expansion joints, railroad tracks)	• Adjust height and angle of plows (main and wing)
Spreading	N/A	• Monitor road temperature (gages, weather stations, skies) • Monitor salt	• Adjust spreader controls

Figure 3. Snowplow heads up mountainous highway in Globe district

FINDINGS

Driver Awareness Training

Surveys, designed to measure how well the training was received by snowplow operators, were distributed by the maintenance supervisors in each organizational subsection and returned to the ASU researchers. More than 80% of the drivers returned surveys. Among the respondents, 45% had less than two years experience driving snowplows for ADOT, while 10% had over 16 years of experience. Nevertheless, 75% of all respondents had more than five years experience in driving various other types of heavy equipment.

Most drivers were satisfied with their simulator training. The majority felt that the four-hour classroom/simulator training was adequate. (That was reassuring, since the overwhelming proportion of driver respondents the year before felt that the two-and-a-half-hour program offered by L-3 the year before did not offer enough simulator time.) Among the respondents, 98% felt that they were at least relatively successful in the simulator training. Only 20% of the respondents found the training somewhat demanding; none felt it was very demanding.

Drivers were asked to identify specific challenges that they faced in snowplowing and whether the driver awareness training addressed these challenges. Drivers emphasized problems with limited visibility (53% of respondents) and dealing with traffic (59%) as the two most serious challenges they faced. Twenty-two percent of all respondents felt that the simulator fully addressed these concerns. Among the respondents, 43% felt that they experienced major challenges relating to roadway issues (such as guard rails, narrow mountain roads, and limited roadway shoulders), and only 22% felt these issues were adequately addressed by the simulator scenarios that did not include the roadways as challenging as those in the Globe district. When asked what aspects of the training they were able to use on the job, the largest proportion of driver respondents (29%) noted (via write-in responses) that they were able to use concepts that related to driver alertness to hazards or potential hazards. Indeed, that was the primary focus of the driver awareness simulator training course.

For newer drivers, the simulator training met a real need for practice driving in the snow; 65% of them wanted more time in the simulator. The problem for more experienced drivers, according to survey results, was that the simulator did not include controls for the sander and de-/anti-icing chemicals or levers to lift and angle the plow. These were features that were not available on the L-3 simulator, but represented continuing challenges.

Computer reports were generated in response to each of the three simulator runs performed by individual drivers. They reported a full array of driving violations, including collisions, following distance, sudden deceleration, stalling the engine, riding the brake, and riding the clutch. Among the driver participants, 26.5% were involved in a collision in at least one of the scenarios during the driver awareness program, highlighting the importance of sharpening drivers' awareness of existing and potential hazards.

There seemed to be some correlation between the number of collisions and the type of scenario. For example, only 10% of drivers experienced a crash in a scenario involving mountain roads, the environment most familiar to Globe district drivers, while 33% of the crashes were associated with a scenario featuring a downtown in a small city. Globe drivers do not typically plow in towns, with signalized intersections, parked cars, and pedestrians. The importance of the awareness training was not lost on drivers who drove the challenging town scenario. An animated discussion followed in the classroom session after all drivers finished the town scenario.

Focus Groups on Driver Awareness

A series of focus groups in June provided an opportunity to follow up on observations shared by the drivers in the April mail-back survey and to probe further into issues related to transfer of training. Recently hired drivers commented that they felt that the driver awareness training offered them a "jump start" on the snowplow season. In Arizona, where the snowplow driver turnover rate is greater than 25% in most maintenance districts, this is a substantial benefit. Some inexperienced drivers recounted stories in which they were white-knuckled while driving in whiteout conditions. But when they stopped, took a deep breath, and reviewed in their minds their awareness training, they were able to persevere. Nearly all of the drivers who experienced the simulator training appreciated the driving awareness aspects, suggesting that it "opens your eyes" and "makes you think." More experienced drivers, however, again pointed out the need for a simulator with controls they could use to practice key operational skills. They agreed that the SIPDE training course served as a refresher course on overall driver awareness (which was precisely the focus of this training course), but they wanted a program that would sharpen the skills they felt were essential to plowing snow.

A focus group of maintenance supervisors was overwhelmingly positive about the snowplow simulator. They, like the inexperienced drivers, saw the potential of the simulator as a tool for teaching driving safety skills. They also pointed out that even the experienced drivers were not fully alert to potential hazards and were involved in crashes on the simulator. The supervisors felt that operational techniques had to be learned on the job.

Fuel Management Training

Overall, the driver response to the fuel management training was more positive than for the driver awareness training, and there was greater consensus among new drivers and veterans. (Indeed, some veteran drivers admitted that the fuel management training was the first training they had received on the subject of proper shifting techniques).

Although the drivers were virtuously unanimous in their opinion that saving fuel while plowing snow was nearly impossible, many (even some veterans) said they had learned something of value in the fuel management training that they could use year-round. Indeed, reports generated by the simulator suggest that most drivers' performances did improve over the course of the training, as shown in Figure 4. Figure 4 graphs the changes in miles per gallon used by individual drivers in a timed scenario before and after the training course.

Many drivers, who scored low on their pre-test, made substantial improvements on their post-test at the end of the fuel management course. There were, however, drivers who performed better on their pre-test than on their post-test during the fuel management training. This may be a reflection of the lack of adequate practice time during the training program or of the fact that a number of the drivers regularly use automatic trucks and are not accustomed to shifting gears.

The focus groups revealed that drivers are very much aware of the fuel economy they get with their trucks. Many reported that they have applied the shifting techniques learned in class and have demonstrated improved fuel economy, which is apparent evidence of transfer of training.

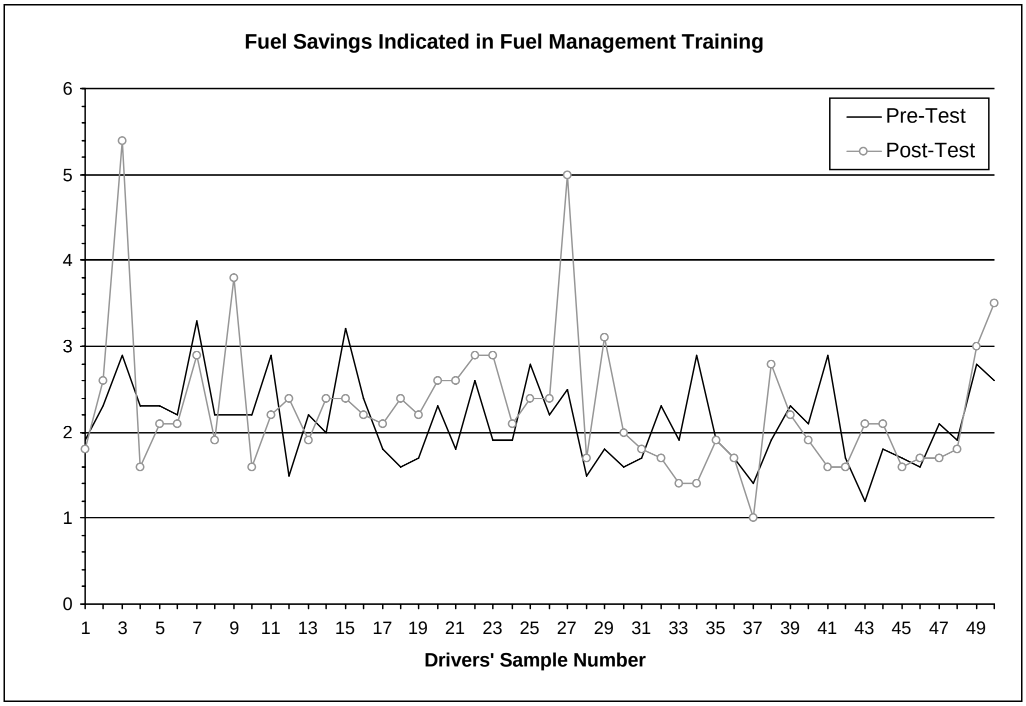

Figure 4. Pre- and post-test results of fuel management training program

ASSESSMENT

How Real is Real Enough?

Many trainees, especially those with many years of driving experience, criticized the driver awareness simulator program for its lack of realism. However, from what the surveys and focus groups revealed, even these skeptical trainees seemed to have learned something from both the driver awareness and fuel management courses. One might reasonably expect there to be a strong relationship between simulator realism and knowledge transfer, but this is not necessarily so. According to Vance et al. (2002), the realism (or fidelity) required of a particular simulator depends upon the training to be conducted, and "certain tasks and skills can be learned even in very crude simulators." In fact, according to these researchers:

> Reasoning or cognitive ability tasks do not require high physical fidelity levels. The skills in these settings are generalizable to many different areas, not only truck driving, and the physical layout need not be exact. High physical fidelity is necessary when the training involves learning perceptual-motor skills, or the interaction of the trainee with the layout of the equipment. An example of where high fidelity is needed is when the goal is to practice tasks that cannot be practiced in the field because they are too dangerous, such as simulated spinouts on ice. (p. 13)

Transfer of Training

In order to evaluate the effectiveness of the ADOT simulator training program, therefore, the research team focused attention not on the fidelity of the simulator, but on the potential for transfer of training, the ability to apply what is learned in one context to another context (Goldstein 1986). As Emery et al. (1999) note, "The validation of simulation... for the training of a particular skill is most appropriately addressed through an assessment of whether that training actually transfers to the environment in such a way as to encourage skill proficiency and safe operating practices."

In the current study, this refers to the ability of snowplow operators to apply what they have learned in the simulator training course to on-the-road driving practice. If drivers trained in the simulator perform better on the road than those drivers not trained in the simulator, then it could be concluded that positive transfer has occurred.

To date, the indicators are the self-reports included in the surveys, positive discussions in focus groups, and individual driver comments about having greater confidence when driving in low visibility. An assessment of crash and claims data proved to be inconclusive because of the limited number of incidents; one crash involving a snowplow can cause a spike in the report. Nevertheless, there are some encouraging initial findings.

Quantitative Assessment

Table 2 provides an indication of the level of operational losses associated with snowplowing in the Globe district that were incurred by ADOT over time. These operational loss figures include all ADOT equipment repair costs and claims associated with snowplow-related injuries or accidents. Operational losses included auto liability claims (claims involving a plow's contact with another motor vehicle), general liability claims (claims by an outside party for damage caused by a plow or plowing materials), injury to private property (inadvertent damage to private property caused by a plow), and ADOT equipment repairs (repairs of plow-related equipment other than normal wear and tear). Workers' compensation claims are generally not included in operating costs and have not been included in the table. (The liability claim data was obtained through ADOT Risk Management from the Arizona Department of Administration, and ADOT Equipment Services provided the truck repair data.)

Logically, the potential for these losses would increase with the level of exposure, so an effort was made to normalize losses by relating them to levels of exposure. Exposure for snowplowing operations, however, can be measured in different ways. Table 2 divides the total annual operational costs associated with snowplowing in the Globe district first by miles plowed, second by hours of operation, and finally by number of snowfall inches. The table shows considerable variability in terms of level of losses associated with the Globe district over six snow seasons and offers a comparison in terms of several measures of exposure. It is worth noting is that the Globe district fared better in the 2005–2006 snow season after all its drivers had taken the awareness simulator training. The loss costs were lower in terms of all measures of exposure after the simulator training program. (The figures in the table do not reflect any annual compounding.)

As indicated above, a single major accident, even one where the ADOT driver is not at fault, can cause the operational losses to spike in a single season, despite all efforts to train drivers. Nevertheless, the decline in total operational losses and in all measures of exposure is encouraging. Future years of data will indicate whether this trend can be sustained.

Table 2. Measures of exposure related to operational loss costs in Globe, AZ

Snow season	Total operational loss costs associated with snowplowing ($)	Cost/mile of snowplow operation ($/mile)	Cost/hour of snowplow operation ($/hour)	Cost/inch of snowfall ($/inch)
1999–2000	14,332	0.14	3.35	480.94
2000–2001	7,640	0.03	0.66	78.68
2001–2002	6,916	0.04	1.09	181.52
2002–2003	19,911	0.10	2.59	247.65
2003–2004	5,450	0.02	0.64	105.62
2004–2005	42,574	0.25	5.00	588.03
2005–2006	18,631	0.19	3.05	355.17

CONCLUSIONS

The driver awareness program seemed to do a good job of training tactical skills, while the fuel management program seemed to do a good job of teaching control skills. However, all of the training took place on the same driving simulator. Transfer of training, therefore, seems to have as much to do with the skills being trained as the simulator's realism (or fidelity). While none of the control skills shown in Table 1 is addressed in the driver awareness program, it is important to note that the program was never intended to teach control skills. The L-3 TranSim VS III model simulator lacks the physical fidelity needed to facilitate training of many control skills (gear shifting being a notable exception).

The fuel management training program is much more narrowly focused, emphasizing proper gear shifting (and related clutch usage). As shown in Table 1, gear shifting is comprised of control-level driving skills corresponding to the driving activity of snowplow operation. Here, there are already indications that positive transfer of training is taking place. Drivers in the focus groups reported that they quickly applied what they had learned on the simulator to their everyday driving and saw positive results (not only in ADOT vehicles, but in their personal vehicles as well). Although not statistically significant, study results also suggest positive transfer of training. A more complete analysis is underway in a follow up study.

Findings Related to the Michon Model

The findings, when seen through the lens of Michon's driving model, lead to some useful insights. These insights may be valuable to others considering the use of simulators for training operators of snowplows or other heavy equipment.

1. New and experienced snowplow operators seem to want different things from simulator training. While the novices are content with learning tactical-level driving skills, veterans look to the simulator primarily for a "refresher course" focused on control-level skills (the skills they don't get during their daily off-season work). How well each group of drivers will respond to simulator training, therefore, may depend on the driving skills being taught.

   For states like Arizona, with high rates of driver turnover, even simulators with relatively low physical fidelity might be very useful for training tactical-level driving skills that are more closely related to issues of safety, the primary concern of DOTs. This, of course, is precisely the point emphasized in driver awareness courses.

   The strong relationship between safety and tactical skills may help explain the apparent contradiction between the supervisors' and the veteran drivers' views of the simulator training.

ADOT supervisors seem to focus primarily on tactical-level skills that further safety, rather than the control-level skills desired by experienced drivers.

2. It may be easier to quantify transfer of control-level skills than transfer of tactical-level skills. Because tactical skills are more "big picture" skills, they are also more complex to study and measure. It's relatively easy to determine if drivers are shifting gears more efficiently (e.g., fuel consumption, reduced clutch maintenance, etc.), but it's much more challenging to determine if drivers are using SIPDE techniques. It is important to note, however, that "overlearning" control-level skills frees up cognitive resources, and therefore may improve performance of tactical skills; these two skill types are clearly interrelated.

   For the purposes of a cost/benefit analysis, focusing on issues related to control-level skills may prove more fruitful than focusing on more elusive issues related to tactical-level skills. However, if it is true that safety issues are more strongly related to tactical skills (see (1) above), then issues related to tactical skills must not be ignored for the sake of a simpler cost/benefit analysis.

3. How a training program is presented to trainees is critical to its success. For example, some drivers resented the thought of being taught fuel management. Drivers were virtually unanimous in their opinion that saving fuel while at the same time plowing snow was nearly impossible. However, they were quite eager to learn about proper shifting techniques (which was the real focus of the course). Similarly, the driver awareness course was generally referred to as "Snowplow Simulator Training," which built up specific expectations (for control-level skills training) in the minds of trainees. Had the course been called something like "Driving Awareness Training," perhaps it would have been better received (especially by those who criticized its lack of realism).

   The first step in designing or purchasing a training program, then, ought to be asking what driving skills are needed. (This ought to be straightforward, since training is generally aimed at addressing some existing problem.) Are the skills to be taught control-level or tactical-level skills? How the course is marketed to trainees (and others in the organization) should be based on the skills being taught. It seems reasonable to think that training programs that are better received by trainees are more likely to be more effective learning tools.

4. Trainees were unanimous in their praise of the ADOT trainers, who are all veteran snowplow operators. In fact, the trainees reported that they learned as much from the low-tech storytelling aspects of the training sessions as from the high-tech simulator. These in-house trainers are valuable assets to the organization and could be leveraged further in a variety of training programs (especially important in organizations with high turnover rates). They have such a wealth of personal experience that they are able to teach both tactical- and control-level driving skills, which may translate to the simulator environment.

SUMMARY

The experience with simulator training for snowplow operators in Arizona leaves both the research team and ADOT optimistic about the potential for driving simulators to be an integral part of a comprehensive driver training program for state DOTs. Clearly, there are elements of these driver training programs for which the simulator is not well-suited; this is especially true of simulators with low physical fidelity. Nothing can replace real-world, behind-the-wheel training. Broadly speaking, simulators seem to be better at training for tactical-level driving skills than for control-level driving skills. However, when appropriately equipped, simulators can be effective tools for teaching control skills as well.

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Development of a Real-Time Productivity Measurement System for Bridge Replacement

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ABSTRACT

Increased attention has been paid to the highway bridges, one of the critical components of the nation's transportation network, since the terrorist attacks of September 11, 2001, Hurricane Katrina, and the tsunami in South Asia. To enhance the capability of rapid bridge replacement after extreme events, a real-time productivity measurement system has been developed. The developed system has the potential to enhance the capability of rapid bridge replacement by providing more accurate onsite productivity information. Using the information, a more reliable construction schedule could be developed to support the rapid bridge replacement operations. To validate the system, field experiments were conducted at U.S. Highway 36 near Washington, Kansas. This paper presents the major components of the developed system, the framework of the system, and the preliminary field experiment results.

Key words: bridge—construction—productivity—replacement

INTRODUCTION

The terrorist attacks on September 11, 2001, and subsequent potential threats to U.S. transportation systems, have presented an urgent need to develop emergency management plans to quickly react to the possible consequences of an extreme event. These events include terrorist attacks as well as man-made and natural disasters, such as explosions, fires, floods, and earthquakes. Highway bridges, as a critical component of the nation's transportation network, have received closer attention from government agencies. The reasons that bridges are the key element of the nation's transportation system are as follows (Barker and Puckett 1997):

1. A bridge controls the capacity of the system.
2. A bridge is the highest cost per mile of the system.
3. If a bridge fails, the system fails.

To respond to an extreme event, a developed emergency management plan must include four related components (Parsons Brinckerhoff 2002):

1. Mitigation: Steps taken in advance to reduce the potential loss from an extreme event.
2. Preparedness: Steps taken in advance to facilitate response and recovery after an extreme event.
3. Response: Steps taken during or immediately after an extreme event to save lives and property.
4. Recovery: Steps taken to restore the affected areas to their normal status.

Since September 11, 2001, several research projects have been conducted to identify the infrastructure's vulnerabilities and to help government agencies develop or update the emergency management plans with a focus on mitigation, preparedness, response, and recovery. The American Association of State Highway and Transportation Officials (AASHTO) recognized the need to address the nation's vulnerability assessment requirements for highway transportation and sponsored the development of a guide for critical asset identification and protection (SAIC 2002). The guideline's authors divided vulnerabilities in highway transportation into the following three general categories:

1. The physical facilities themselves (e.g., bridges, tunnels, roadways, and interchanges).
2. The vehicles operating on the system.
3. The information infrastructure that monitors and manages the flow of goods, vehicles, and people on the highway system.

This guide provides a starting point to identify and mitigate the vulnerability of and consequences to highway transportation assets from terrorist threats or attacks. A companion document, *A Guide to Updating Highway Emergency Response Plans for Terrorist Incidents*, also funded by AASHTO and developed in parallel with the previous guide, assists government agencies in preparing and executing a coordinated emergency response to terrorist threats or attacks to the highway transportation system (Parsons Brinckerhoff 2002). Besides these two guides, AASHTO and the Federal Highway Administration sponsored other research projects on bridge and transportation security. One project was titled *Design of Highway Bridges for Extreme Events*, which was supervised by the National Cooperative Highway Research Program (NCHRP). The objective of this research was to develop a design procedure for application of extreme event loads and combination loading to highway bridges (Ghosn et al. 2003). Another project was titled *Surface Transportation Security*, which was also supervised by the NCHRP. Results of this project were released in NCHRP Report 525 (2004).

State departments of transportation (DOTs) also initiated efforts to investigate and develop methods to lessen the impact of terrorist attacks and other extreme events on their transportation infrastructure. Two recent research projects concentrated on bridges. One project was entitled *Design of Bridges for Security* and was intended to determine how bridges may be economically designed for security (Burkett et al. 2004). The other project was entitled *Rapid Bridge Replacement Techniques* and was intended to identify optimal bridge replacement and repair techniques (Bai and Burkett 2006). Bridge replacement techniques were to include both temporary and permanent replacement.

Results of previous research indicate that there is an urgent need to address the recovery component in the bridge emergency management plans. Specifically, one of the areas that must be improved in the recovery is to develop innovative technologies that could be used to produce an accurate and reliable construction schedule to support rapid replacement operations. For example, the estimated time for the replacement of the I-40 Webbers Falls Bridge started at 12 months, then went down to 6 months, and finished in a little over 2 months. Although the replacement was finished ahead of the original schedule, the process clearly indicated that an accurate and reliable schedule was unable to be produced and provided to the general public based on the existing construction technologies.

PROBLEM STATEMENT

Currently, most of the construction schedules are developed using the critical path method (CPM). A scheduler builds a CPM network based on durations of construction activities and relationships between activities with the consideration of resource constraints. Durations of activities are determined based on historical data (similar work done in the past) or estimation done by someone in the company (e.g., project manager, project engineer, or superintendent). Construction duration can be estimated using the following formula:

$$\text{Duration} = (\text{Quantity of Work}) / (\text{Construction Productivity}) \quad (1)$$

Because the quantity of work is relatively easy to determine accurately using printed drawings or a CAD system and specifications, the accuracy of the duration largely depends on the accuracy of construction productivity. There are many factors that will impact the construction productivity, such as weather, site condition, quality of supervision, complexity of each task, and labor skill level and age. To quantify these factors and to determine exactly how these factors impact the construction productivity are beyond the capability of current technology. Without accurate productivity data, it is not difficult to understand why a scheduler is unable to produce a reliable CPM schedule. In summary, poor productivity data impacts the accuracy of activity durations, and inaccurate activity durations make it impossible to produce a reliable construction schedule.

Productivity has been widely used as a performance indicator to evaluate construction operations through the entire construction phase. There are many methods that can be used to determine onsite construction productivity, such as questionnaires, activity sampling, still photography, time study, time-lapse filming, and full-time videotape recording (Adrian 2004; Oglesby et al. 1989). Among these methods, time study, also called stopwatch study, is the classic productivity measurement method developed by Frederick W. Taylor in 1880 (Meyers 1992). Since 1980, more and more construction companies have utilized time-lapse filming and full-time videotape recording methods due to the advancement of technologies and cost reductions for required equipment. However, these methods are conducted by employing additional people to manually collect data on the construction sites. As a result, using these methods increase costs, delay the analyses, and interfere with crew activities that may produce inaccurate data.

OBJECTIVE

The objective of this research project is to develop a prototype wireless real-time productivity measurement (WRITE) system that could be used to measure the onsite construction productivity for rapid bridge replacement. Using the real-time productivity data, engineers and project managers may be able to accurately determine bridge replacement progress and easily share the information with all parties involved in the bridge replacement project. Thus, the wireless real-time productivity measurement technology has great promise for improving construction schedule forecasts and increasing emergency response capability after extreme events.

DEVELOPMENT OF WRITE SYSTEM

The developed WRITE system includes a video camera, a digital camera, a data processor, an AC transformer, two antennas, and a laptop computer, as shown in Figure 1.

Figure 1. Major components of the WRITE system

The video camera and digital camera are housed in a steel box that can be mounted on a pole or wall. An operator can rotate the steel box horizontally (360 degrees) and vertically. The data processor, also called minicomputer, contains a software program called VM95 that can control the camera movement, the number of shots, the duration of a shot, and the zooming. A monitor, a keyboard, and a mouse are necessary items for the data processor to display real-time pictures and live video. The AC transformer transfers electric energy to other components. It can be mounted indoors or outdoors. Two antennas are used to transfer data wirelessly within a 12-mile range. Figure 2 shows the framework of the WRITE system.

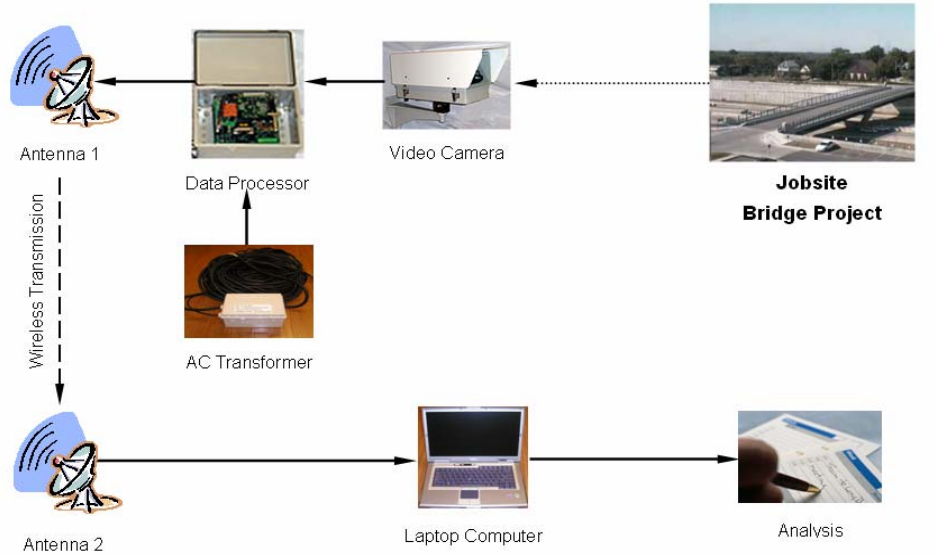

Figure 2. Framework of the WRITE system

In June 2007, researchers conducted field experiments on US-36 near Washington, Kansas. The main purpose of the field experiments was to determine if the developed WRITE system can be used to measure onsite productivity accurately. To accomplish this task, a comparison test was made between the productivity data collected by the WRITE system and the productivity data gathered using the stopwatch study, the classic productivity measurement method developed by Frederick W. Taylor. Figures 3 and 4 show the field experiment setup using the WRITE system and stopwatch, respectively.

Figure 3. Field experiment setup using the WRITE system on US-36

Figure 4. Data collection using stopwatch

These two time study methods were utilized to measure the onsite productivity for an asphalt paving construction project. The hypothesis for the analysis is as follows:

$$H_0: \mu_1 = \mu_2 \tag{2}$$

$$H_1: \mu_1 \quad \mu_2 \tag{3}$$

In the hypothesis test, μ_1 and μ_2 are means of cycle time for the asphalt paving construction measured by the WRITE system and stopwatch, respectively. The cycle time is the sum of working time (applying hot-mix asphalt to the desired width and thickness) and nonworking time (idle or waiting) of asphalt paving. For the preliminary experiments, a total of nine cycles were observed, and each cycles lasted for five minutes. Using the two-tail t test, the null hypothesis cannot be rejected at the significant level of 5% (t statistic = 0.022 < $t_{(0.025,\ 8)}$ = 2.306). This result indicates that statistically there is no difference between the productivity measurements taken by the WRITE system and the stopwatch study.

CONCLUSION

After the September 11, 2001, terrorist attacks, Hurricane Katrina, and the tsunami in South Asia, rapid replacement of damaged infrastructure such as bridges after extreme events has received close attention from government agencies, engineering and construction communities, and the general public. To enhance the capability of rapid replacement of damaged infrastructure after extreme events, there is an urgent need to develop innovative technologies that could be used to produce an accurate and reliable construction schedule to support rapid bridge replacement operations. To respond to this emergency need, a wireless real-time productivity measurement system was developed and tested in a construction project site. The preliminary test results indicated that the developed system can measure the onsite construction productivity accurately. Additional field experiments are scheduled in the near future to fully test the system. Results will be reported at a later time. If successful, the capability of rapid bridge replacement will be improved significantly.

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Early-Age Smoothness Variations of Jointed Plain Concrete Pavements

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ABSTRACT

Pavement smoothness is a major measurement related to the serviceability of a road for the traveling user. Many transportation agencies conduct pavement smoothness measurements for quality control and quality assurance purposes to judge the quality of new pavements and monitor the condition of their pavement network. Initial smoothness of pavement especially has been one of the major concerns because poor initial smoothness leads to higher rehabilitation costs, shorter services life, and significant reduction of ride quality (Lee 2005). Previous studies (Khazanovich et al. 1998; Janoff 1988 and 1990; Akhter et al. 2002) also show that initial smoothness can significantly affect the progression of roughness in a pavement. Many agencies have established and implemented smoothness specifications for newly constructed pavements. Using these specifications, the agencies determine the bonuses or penalties to the contractor, thereby encouraging the contractor to construct pavements with smoothness levels higher than a specified value (Chou et al. 2005).

The temperature and moisture variation in a climate could result in changes in the curvature of the portland cement concrete (PCC) slab, known as curling and warping. Previous studies (Hveem 1951; Karamihas et al. 1999 and 2001) shows that curling and warping can influence long-term PCC pavement smoothness measurements. However, Perera et al. (2005) recently observed that there was no noticeable effect of slab curvature changes on initial smoothness in five newly constructed PCC pavements.

The primary objective of this study is to investigate the variations of early-age concrete pavement smoothness at different measurement times and locations in different jointed plain concrete pavements (JPCPs) representing different ranges of construction procedure times. The case studies involve three newly constructed JPCPs: US-151 (Platteville, Wisconsin), US-34 (Burlington, Iowa), US-30

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(Marshalltown, Iowa). Each of the JPCPs studied experienced a variety of climate conditions. US-151 (Platteville, Wisconsin) was constructed late in the year (October) and experienced only modest daily diurnal cycles. US-34 (Burlington, Iowa) was constructed early in the paving season (June) and experienced multiple rainfall events during the evaluation period. US-30 (Marshalltown, Iowa) was built towards the middle of the construction season (July) and experienced relatively higher ambient temperatures.

In order to capture the effect of changes in PCC slab curvature conditions due to varying environmental ambient conditions throughout the day, field monitoring activities were performed in a diurnal cycle (morning and afternoon). JPCP shows the unique bending curvature behavior associated with temperature and moisture variations through the depth of PCC slab. In addition, this curvature behavior of early-age JPCP is more complicated because several other environmental factors, such as shrinkage, pavement temperature condition during setting, and creep of the slab, could be also involved. However, in general, the maximum or minimum slab curvature conditions are the timeframe for the maximum (afternoon) and minimum (morning) slab temperature gradient. Variations in pavement temperature during the evaluation periods were monitored using the temperature sensors installed within the test sections.

To accommodate the additional effects of paving procedure times and profile measurement locations, different locations on two test sections corresponding to morning and afternoon construction conditions for each case study were evaluated. This diurnal testing of multiple sections provided a better understanding of the changes in smoothness measurements due to environmental ambient conditions for early-age JPCPs.

The travel lanes in two test sections of each JPCP corresponded to morning and afternoon construction, selected for profile measurements. Several profile patterns, as shown in Figure 1, were used to accommodate the data collection. An inclinometer-based profiler such as a Dipstick or Rolling Profiler was used for surface profile measurements at different times (morning and the afternoon) along the different traces of longitudinal direction in the test sections. All measured longitudinal profiles were in the direction of future traffic.

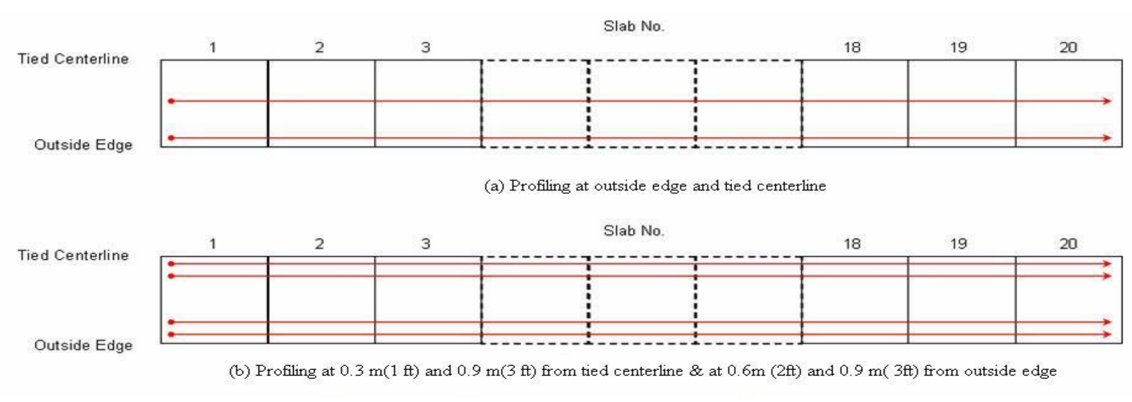

Figure 1. Typical longitudinal profile pattern

Since pavement smoothness is related to a lack of roughness, the severity of roughness in pavements has been used to characterize smoothness. Using the Federal Highway Administration's Pavement Profile Viewing and Analysis (ProVAL) software, the measured longitudinal surface profile data were transformed into smoothness indices, namely the international roughness index (IRI) and ride number (RN).

The results showed that measurable changes in early-age JPCP smoothness do occur at different measurement times and locations. Within the scope of this study, it can be concluded that the variations in early-age JPCP smoothness can be significant from the standpoint of smoothness specifications. The limited field data from this study showed that morning paving produces smoother JPCP (in terms of measured smoothness indices) than afternoon paving. It is also interestingly noted that the variations of IRI during early-age in afternoon paving produces can more significantly influence the smoothness specification grade of new concrete pavements. Therefore, in situations where nighttime paving may not be feasible, which is the preferred option to maintain minimal temperature gradient change during concrete hardening, it is recommended that paving operations be performed during morning times, where the effects of curling and warping on the PCC slab are considered to be minimal.

Key words: early-age—JPCP—pavement analysis and design—smoothness

ACKNOWLEDGMENTS

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I-95 Corridor Mile Marker and Ramp Designation Signing

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ABSTRACT

Mile marker signs on interstates are both a convenience and a safety measure in that they provide travel progress information to motorists and essential location information for 911 emergency procedures. Motorists who report accidents need to be able to identify the location of a crash so that first responders will be able to deploy help from the appropriate facility as fast as possible. Providing crash location information becomes more difficult on complex urban highway interchanges. Inadequate ramp designation signing may lead to incorrect locations being called into the 911 dispatcher. This may cause delays in providing aid. The goal of the research performed in this study was to determine mile marker and ramp designation sign effectiveness using TarVIP, a computer-based sign comprehension analysis, and a test road validation. TarVIP is a traffic sign legibility modeling program that simulates visibility from an automobile with specified headlamps driving toward a sign at a specified speed and for specified driver characteristics such as driver eye position and driver age. The program also incorporates the sign size, letter height, font, background, and legend sheeting materials. TarVIP provided the letter height and the legibility distance of the mile marker and ramp signs, while the comprehension analysis provided the design, content, and layout of the signs. The test track validated the results from TarVIP and the comprehension analysis.

The purpose of this research project was to find the best design/deployment of mile markers and to find the best design for ramp designation signs so that motorists can pinpoint their location and relay it to first responders.

A number of parameters that possibly affect mile marker and ramp sign legibility during nighttime driving conditions were investigated in this study. Among these parameters are available/required

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preview time, recognition distance, and legibility distance. The first and second phases of this study were conducted inside a laboratory at the University of Iowa. The third phase was conducted on a public county road.

In the first phase, a TarVIP model was built to compare the nighttime legibility performance of American Society for Testing and Materials Type III and full cube corner sheeting materials under automobile low-beam headlamp illumination. Parameters for the driver eye position in relation to the sign and headlamp locations were obtained from a previous study performed by Zwahlen and Schnell. The benchmark for the minimum required legibility distance (MRLD) was obtained from a study done by Zwahlen. The results generated with TarVIP were legibility distance estimates for the different sign designs. By comparing the MRLD with the computed legibility distance, it was possible to determine the required sign legend font, letter height, and overall sign size. TarVIP results demonstrated that a traffic sign legend with Series D font has a longer legibility distance than Series B font. However, using the larger Series D font is not always possible due to sign size trade-offs. The Federal Highway Administration has seven different fonts, "A" (the narrowest), "B," "C," "D," "E(M)," and "F" (the widest). TarVIP results also demonstrated that if a full cube corner sheeting material was used on the legend and background, the maximum legibility distance was increased. Thus, TarVIP was helpful in evaluating the extent to which one may be able to overcome smaller fonts by using sheeting materials with higher retroreflectivity.

The second phase of this study was a traffic sign comprehension analysis to evaluate the information motorists were able to recall about a simulated sign shown for three seconds. The conceptual idea is that a motorist might have to recall the last mile marker or ramp designation sign when making a call to first responders. After viewing each sign on a computer screen, the subjects were asked a series of questions probing sign comprehension. The three-second presentation time for the sign stimuli was selected using a pilot run based on an 85th percentile correct response level. Participants were seated in front of a computer while the program ran through the presentation and answer sequence.

The third phase, the test road study, was used to validate what was found from the TarVIP model and comprehension analysis. This phase of the study was only run during nighttime with low-beam headlamp illumination. Participants drove at 15 mph throughout the study. A Ford Taurus SE (2000) instrumented car was equipped with an audio and video recorder and a distance measurement instrument system to measure the legibility distance to the signs. Participants were asked to identify the color of the sign and to read out loud whatever they could read when the sign's legend would come into view. After the participants were done driving through the test section, a post-survey was administered to evaluate the comprehension of the signs and to rank the quality of the information presented on the signs.

Key words: legibility distance—mile markers—ramp signs—TarVIP—traffic signs

Determination of Load Ratings for Non-Composite Steel Girder Bridges through Load Testing

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ABSTRACT

This project was Phase II of the recently completed Iowa Highway Research Board project, “Alternative Solutions to Meet the Service of Low-Volume Bridges in Iowa” (TR-452). In Phase I, the overall objective was to develop a state of the practice in the area of bridge maintenance/rehabilitation/ strengthening. Information was obtained from extensive literature reviews and from two questionnaires, a national questionnaire and a questionnaire sent only to Iowa county engineers (ICEs). The questionnaire to ICEs obtained information on unique solutions to various bridge problems they have encountered, including problems specifically associated with low-volume road (LVR) bridges. Based on the evaluation of the information obtained from Phase I and input from several county engineers, common problems with substructures and posted steel stringer bridges were identified, and Phase II was initiated. This paper presents a brief summary of the scope of the research and results for both the superstructure and substructure portions of the study.

Bridge testing (diagnostic or proof load) is becoming an increasingly viable tool for evaluating the response of a bridge to live load and thus determining performance-based ratings. Bridge testing is

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relatively costly compared to the conventional rating process and thus can not be used on all bridges. Recent research has focused on determining if the physical structural performance of bridges within a family or fleet that have similar physical characteristics can be predicted from testing a representative sample of similar bridges in the fleet and using that information to better evaluate bridges in the family that are not load tested. This would eliminate the need to test all bridges but would still retain some of the benefits gained through physical testing for providing more accurate ratings.

In this study, a family of single-span non-composite steel girder bridges with cast-in-place concrete decks was chosen to determine if predictable superstructures characteristics could be identified. These bridges lack a shear connection between the concrete deck and steel girders and were commonly built prior to 1950. Past testing has shown that these bridges will often produce a live load response lower than analytical models due to various factors, including partial composite action between the deck and girders, edge stiffening of the exterior girders due to the presence of curbs and railings, bearing restraint due to connection details partially restraining the end rotation of the girders, and more precise live load distribution factors. The study results were based on the testing and evaluation of six bridges using diagnostic load testing. Conventional load ratings of the six bridges were performed by three different rating agencies, and the results were compared to performance-based load ratings determined through diagnostic load testing. The test results showed that the diagnostic load test ratings were larger for all six bridges compared to the conventional ratings. There were some behavior characteristics that were predictable, and thus extrapolation of these performance characteristics could possibly be extrapolated to other bridges in the fleet. It is recommended that a more statistically significant sample group of fleet bridges be tested and evaluated before widespread application of fleet results can be reliably extrapolated.

Current conventional bridge rating systems rely primarily on superstructure information, and the rating process of the substructure is typically not as detailed or quantifiably rigorous as that for the superstructure. The substructure condition, however, can be a governing factor in bridge integrity, particularly in cases of bridges with unknown foundations or bridges supported by timber piling. With most of these bridges, there are no design or as-built bridge plans and no documentation of the type, depth, geometry, or materials incorporated in the foundation. In addition to the lack of design information, timber piles exhibit deterioration with time due to different biological and physical factors (Figure 1). If not detected and mitigated, pile deterioration can considerably reduce the pile bearing capacity.

Figure 1. Timber pile deterioration

Currently, there are no reliable means to evaluate timber substructures. The lack of reliable evaluation methods often leads to conservative and costly maintenance practices. Therefore, this study was completed to develop procedures for assessing bridge substructures and propose various procedures for rehabilitating/strengthening/replacing inadequate substructure components. In this study, problems with timber substructures in Iowa were identified by inspecting 49 low-volume bridges with poor performing substructures. Furthermore, the six bridges previously noted that were load tested to evaluate the superstructure performance were also tested and evaluated to determine the live load distribution and performance of deteriorated bridge foundations. During load testing, the timber piles and backwall were instrumented with strain gages (Figure 2). Pile and backwall strain responses were used to make inferences on the substructure performance. By using nondestructive and destructive static load testing, it was possible to identify deteriorated pile sections and evaluate the overall performance of the bridge substructures. In future static load testing, it is recommended that pile movement parallel and perpendicular to the backwall be measured so that it is possible to separate pile strains induced by axial and bending loads. In addition to evaluating the structural performance of substructures for rating purposes, additional work was performed at one of the load tested bridge abutments, where several pile integrity tests were performed in an attempt to estimate the unknown pile length below ground (Figure 3).

Figure 2. Timber substructure instrumented with strain gauges (south abutment, Bridge No. 237350, Mahaska County)

Figure 3. Pile integrity testing (Bridge No. 237350, Mahaska County)

To estimate the residual bearing capacity of deteriorated pile sections, a laboratory study was carried out correlating the pile elastic modulus determined by axial compression tests and the dynamic elastic modulus determined using the nondestructive ultrasonic stress wave technique. The ultrasonic stress wave technique was also used to produce two-dimensional tomography images of internal pile deterioration (Figure 4). Test results indicated that the ultrasonic stress wave test is capable of predicting the residual capacity of deteriorated timber piles and detecting internal pile damage; however, the method has a tendency to overpredict the area of the internal defect. An additional laboratory study was conducted where selected pile repair methods were evaluated to determine the effectiveness of the method in restoring axial and bending capacities of timber piles (Figure 5). The repair methods investigated were capable of partially restoring the axial and bending capacity of damaged piles.

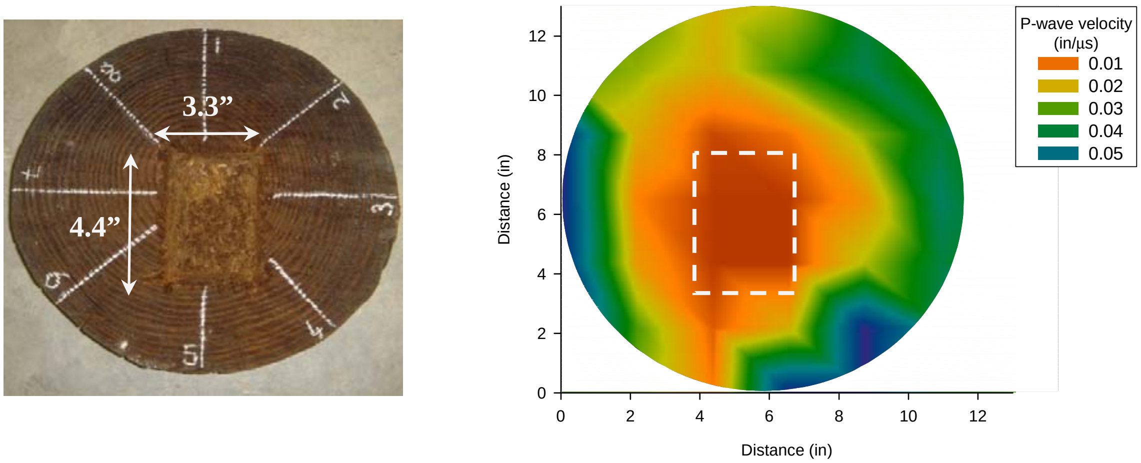

Figure 4. Tomography image showing the pile internal condition

Figure 5. Timber pile repaired using filler epoxy and fiber-reinforced polymer

Assessment of Channelizing Device Effectiveness on High-Speed/High-Volume Roadways

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ABSTRACT

Part 6 of the Manual on Uniform Traffic Control Devices (MUTCD) describes several types of channelizing devices to warn and guide road users through work zones, including cones, tubular markers, vertical panels, drums, barricades, and temporary raised islands. On higher speed/volume roadways, drums and/or vertical panels have been popular choices in many states due to enhanced visibility and a more formidable appearance compared to standard cones. However, the larger sizes of these devices also require more effort and storage space to transport, deploy, and retrieve. Recent editions of the MUTCD have introduced new devices for channelizing; specifically of interest for this study is a taller (>36 in.) but thinner cone, sometimes referred to as a channelizer. While this new device does not offer a comparable target value to that of drums, the size is significantly larger than standard cones, and stability is improved as well. In addition, these devices are more easily deployed and stored than drums.

An investigation of the effectiveness of these devices provides a reference for states to use in selecting appropriate traffic control for high-speed, high-volume applications, especially for short-term or duration exposures. This study includes a synthesis of common practice by state departments of transportation (DOTs), daytime and nighttime field observations of driver reaction using video detection equipment, and opinions of workers using these devices. The preliminary results of this study are promising for evaluation of these devices. The study should provide a valuable resource for state DOTs to use in selecting the most effective channelizing device for use on high-speed/high-volume roadways where timely merging by drivers is critical to safety and mobility.

Key words: channelizers—traffic control—work zone safety

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INTRODUCTION

The primary function of temporary traffic control in work zones is to provide for the safe and efficient movement of vehicles through and around the control zone while protecting workers and equipment. A concurrent objective of temporary traffic control is the efficient construction and maintenance of the highway. This study tested various types of temporary traffic control channelizing devices to determine if drivers reacted differently to the various devices. The study team established a temporary work zone on US-30 and studied the merge activities of the drivers over four consecutive days. The observations showed little difference in merge activities among the channelizing devices.

Part 6, Temporary Traffic Control, in the Manual on Uniform Traffic Control Devices (MUTCD) describes several types of channelizing devices to warn and guide road users through work zones, including cones, tubular markers, vertical panels, drums, barricades, and temporary raised islands (Figure 1). On higher speed and volume roadways, drums and/or vertical panels have been popular choices in many states due to enhanced visibility and a more formidable appearance compared to standard cones. However the larger sizes of these devices also require more effort and storage space to transport, deploy, and retrieve.

The 2003 edition of the MUTCD introduced more options for channelizing devices, including a larger cone, greater than 36 in. in height. Description of these devices in different states might include grabber cones, 42 in. channelizers, large cones, etc. While this new device does not offer a comparable target value to that of drums, the size is significantly larger than standard cones, and stability is improved as well. In addition, these devices are more easily deployed and stored than drums.

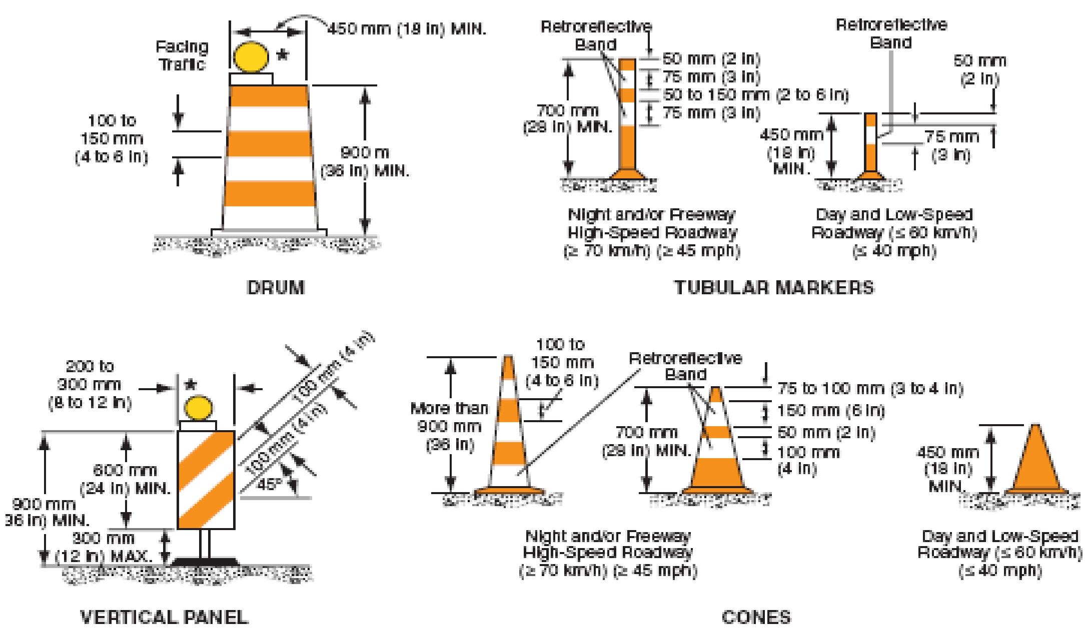


Figure 1. Various types of channelizing devices used

Another new device, introduced in the millennium edition of the MUTCD, was the direction indicator barricade, with positive guidance for drivers provided with an arrow sign (Figure 2).

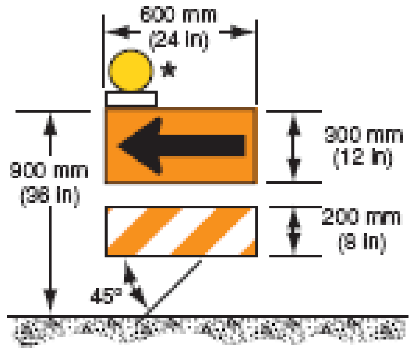


Figure 2. Direction indicator barricade

With many choices available for traffic guidance, transportation agencies could benefit from an effectiveness evaluation of various channelizing devices to aid in selecting the most beneficial and efficient devices, especially for short-term stationary use on high-volume/high-speed roadway applications.

The present study was composed of several tasks, including establishment of an advisory group, a thorough review of current literature, a synthesis of practice survey of selected Midwestern state departments of transportation (DOTs) in the use of channelizing devices, a comparison of the effectiveness of several devices under field conditions, analysis of results, and preparation of a final report.

The advisory team for the evaluation consisted of experienced staff from the Iowa Department of Transportation Offices of Construction, Maintenance, and Traffic and Safety, as well as field maintenance. In addition, representatives from the Federal Highway Administration and contractor representatives were consulted.

LITERATURE REVIEW

A limited number of studies have been conducted on the effectiveness of the channelizer devices that were used in this experiment. A brief discussion of these studies is provided here.

A study conducted by the University of Wisconsin examined safety in reconstruction and maintenance work zones (Chen, Qin, and Noyce 2006). The study concentrated on speeding as one of the major contributors to work zone crashes. Speeding reduces the driver’s ability to safely control, guide, and navigate a vehicle, which increases the possibility of crash occurrence. Many speed control technologies and traffic management strategies are currently being used throughout the country. To decrease the occurrence of potential speeding-related crashes, the Wisconsin Department of Transportation is seeking effective methods of controlling speed at Wisconsin work zones to improve the safety and mobility. To better understand how the speed management strategies and technologies impact work zone speed profiles, there is a need to evaluate their effectiveness in reducing speeds and the number of speeders and improving speed uniformity. There were two primary objectives in this study:

1. Record and compare the speed characteristics with and without speed management strategies at work zones.
2. Measure the effectiveness of speed management strategies at work zones.

Three strategies, including dynamic speed display board, dynamic late merge system, and various enforcement methods, were evaluated in three Wisconsin long-term highway work zones located on interstate highway I-94, STH 29, and STH 164, respectively. The three work zones, located in the northwest, central, and southeast of Wisconsin, respectively, represented a typical geographic sample of Wisconsin drivers. Compared with previous research, the study provided more insight into the long-term impact of some speed control strategies and the effectiveness of the combination of various approaches. The results showed a promising outcome of using these speed management strategies.

A study conducted by the University of Kansas investigated whether observers attend more closely to moving work zone signs if those signs are surrounded by a fluorescent yellow-green (FYG) border. The logic of this signage change is that there is insufficient color contrast between the warning signs and the vehicles on which they are mounted. Two laboratory studies were conducted using very sensitive and robust techniques to measure the attention to signs with and without the FYG border. In each study, a different method for assessing observers' attention was used. In the first study, a perceptual change detection method was used, in which observers were required to detect a change to an object in a traffic scene. It was concluded that changes to more frequently attended objects are noticed more rapidly. A comparison of change detection times for signs with and without the FYG border revealed no difference in the amount of attention allocated to the sign when the FYG border was added. In the second study, eye tracking data was collected for a set of observers. An increase in fixation time on an object indicates that more attention is being paid to that object. In this study, there was again no difference between the two sign types. The researchers concluded that there is no evidence that the addition of a FYG border increases driver attention to vehicle-mounted warning signs (Atchley and Dressel 2006).

In an earlier study of a similar subject, Kamyab and Storm (2001) examined the effect of the FYG background on lane changing behavior in Iowa. Undoubtedly, the FYG background creates a clear contrast between the orange sign and an orange Iowa Department of Transportation (Iowa DOT) truck that follows a moving work area. This study examined the impact of the sign's improved visibility in encouraging drivers to make an early merge to the open lane prior to a lane closure.

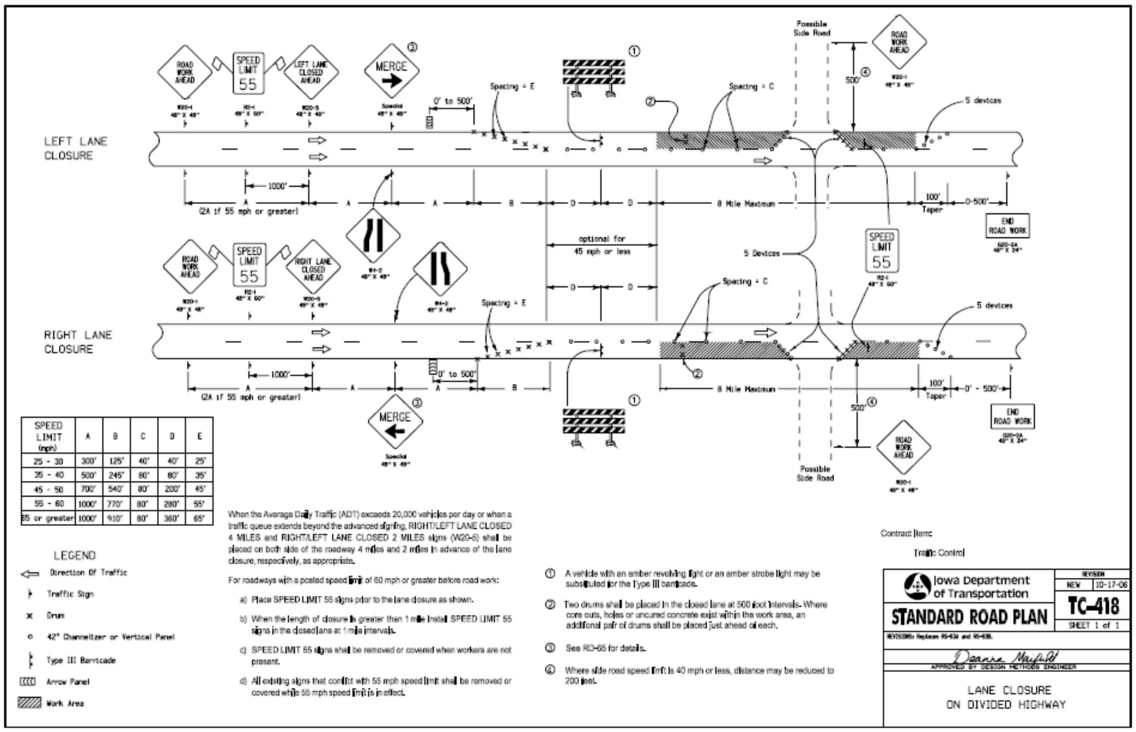
The analysis of data indicates that overall right-lane traffic volumes, recorded during the seven days of data collection after the background placement, were 2% less than the traffic observed in the after condition. The study concludes that the difference between the right-lane traffic observed in the before and after conditions is indeed statistically significant at the 95% confidence level. The resulting right-lane traffic counts are representative of lane distribution changes within 100 ft. upstream of the truck. It is suggested that collecting data at locations, for example, at a distance 500 ft. from the truck, or where most approaching vehicles move to the open lane, would be good information to collect if additional research is conducted. However, using the data collection trailer or individuals to count traffic at a different location may influence drivers' lane changing behavior.

Another factor that could lead to different results is having a real lane closure. Due to the difficulties in developing an experimental design to collect traffic data in advance of an actual moving work zone, data were collected at an "imaginary" work zone. In a more realistic setup, where drivers actually face a real lane closure, a lower right-lane traffic volume is expected to be observed in the after condition.

Furthermore, a survey conducted at a rest area during the after condition indicates that more than 50% of drivers identified the enhanced orange sign as a sign seen on the back of the Iowa DOT truck before reaching the work zone (Kamyab and Storm 2001).

COMMON PRACTICES OF STATES

To assess current practices for temporary traffic control when a single lane is closed on a four-lane divided roadway, several state DOTs were requested to provide typical applications. Information was furnished by Missouri, Kansas, Nebraska, Pennsylvania, and Iowa. The illustrated layout in Figure 3 is used by the Iowa DOT, and similar schemes are employed by the other surveyed states as well, all modeled after Typical Application TA-33 in the MUTCD.



SPEED LIMIT (mph)	A	B	C	D	E
25 - 30	300'	125'	40'	40'	25'
35 - 40	500'	245'	80'	80'	35'
45 - 50	700'	540'	80'	200'	45'
55 - 60	1000'	770'	80'	280'	55'
65 or greater	1000'	910'	80'	360'	65'

Figure 3. Work zone layout used

The main point of difference in temporary traffic control among the contacted states for this situation concerns the selection of channelizing device type used in the taper and lane delineation. As can be seen in Figure 3, Iowa specifies the use of drums in the taper and 42 in. channelizers (tall cones) for lane delineation. Other states are somewhat less prescriptive. For example, Kansas uses conical delineators (trim-line) for channelizing devices. Missouri also selects trim-line channelizers as the device of choice, but may allow contractors to use other devices such as drums, vertical panels, cones, or direction indicator barricades in certain situations. Nebraska uses drums for high-speed roadways. In Pennsylvania, contractors are allowed to use a wide array of channelizing devices for both tapers and lane delineation. Spacing of devices in most states is similar to guidance in the MUTCD, approximately equal to the roadway speed limit for tapers and twice the speed limit for lane delineation. Nebraska, however, uses a much reduced spacing for these devices.

STUDY DESIGN

Data Collection

Data were collected on US-30 near Ames, Iowa, on October 30, 31, November 1, and 2, 2006, using Autoscope video cameras. The cameras were erected on the Y Avenue Bridge overlooking US-30. Data were collected during the afternoon and evening hours on these days.

The study examined the differences in merge behavior using different channelizing devices during the four days of data collection. On the first day of data collection, the drum-like channelizer (Figure 4) was used. These drums were set up in a standard taper at 60 ft. apart to close the right lane and shift traffic to the left lane. Drum-like channelizers are generally used in work zone areas to provide longitudinal channelization within the activity area if their larger size and additional retroreflective area are deemed appropriate. Drum-like channelizers are not generally used in ramp areas, intersections, or areas with limited lateral clearance. When specified, quantities are calculated and shown on the plans.

Figure 4. Drum-like channelizer

On the second and third days of data collection, the 42 in. high trim-line channelizers (Figure 5) were erected in the same area to simulate a work zone. On the second day, the trim-line channelizers were erected at 60 ft. apart. On the third day, the trim-line channelizers were erected at 40 ft. apart. Trim-line channelizers are channelizing devices used in work zones and can be used in ramp areas, intersections, and areas with limited lateral clearances. Trim-line channelizers may be used in daytime or nighttime operations. When specified, quantities are calculated and shown on the plans.

Figure 5. Trim-line channelizer

On the fourth day of data collection, a direction indicator barricade (DIB) was used in conjunction with the trim-line channelizers (Figure 6). Weather conditions, however, were such that the DIB blew over several times during the data collection period, so its effectiveness was not fully measured.

Figure 6. Direction indicator barricade

DIBs can be used instead of trim-lime channelizers in merging tapers, as they provide direction and have a larger visual target area for the motorists. DIBs are not specified in shifting tapers. When specified, quantities are calculated and shown on the plans.

Figure 3 shows the work zone configuration used in the study. This is a standard work zone configuration used by the Iowa DOT for a lane closure. On the first day, the drums were set up at 60 ft. apart. On the second day, the drums were replaced by the trim-line channelizers, set up at 60 ft. apart. The third day, the channelizers were used again, this time set up at 40 ft. apart. On the final day, the channelizers were set up at 60 ft. apart, and a direction indicator barricade was included the work zone.

Table 1 describes the data collected from the video observations. The traffic movements through the work zone were observed and the cars were counted as the drivers began to change lanes and merge to the left lane. The number in the Avg. Begin Merge column is the number given where the driver began the merging activity. The Avg. End Merge column is the number given when the merge movement was deemed to be completed. The Avg. Length Merge is the number from the average length of merge for that particular configuration. The data are divided into day and night categories to determine the extent to which daylight influences merging behavior.

Table 1. Average merge distance by device (ft.)

Overall	Avg. Begin Merge	Avg. End Merge	Avg. Length Merge
Drums Day	2,376	2,107	269
Drums Night	2,214	1,904	310
Channel 60' Day	2,389	2,108	280
Channel 60' Night	2,293	1,882	411
Channel 40' Day	2,403	2,121	281
Channel 40' Night	2,327	1,929	398
Dir. Ind. Bar. Day	2,281	2,035	246
Dir. Ind. Bar Night	2,284	1,954	330

The data in Table 1 show the weighted average distance that the vehicles traveled past the observation point. The merge point is the estimated distance from the top of the work zone taper. Thus, when conventional drums are used during the day, the average begining merge point for the observed traffic is 2,376 ft. from the top of the taper. The average ending merge point is 2,107 ft. from the top of the taper. The average length of length of merge in this scenario is 269 ft. This can be expressed graphically as well, as shown in Figure 7.

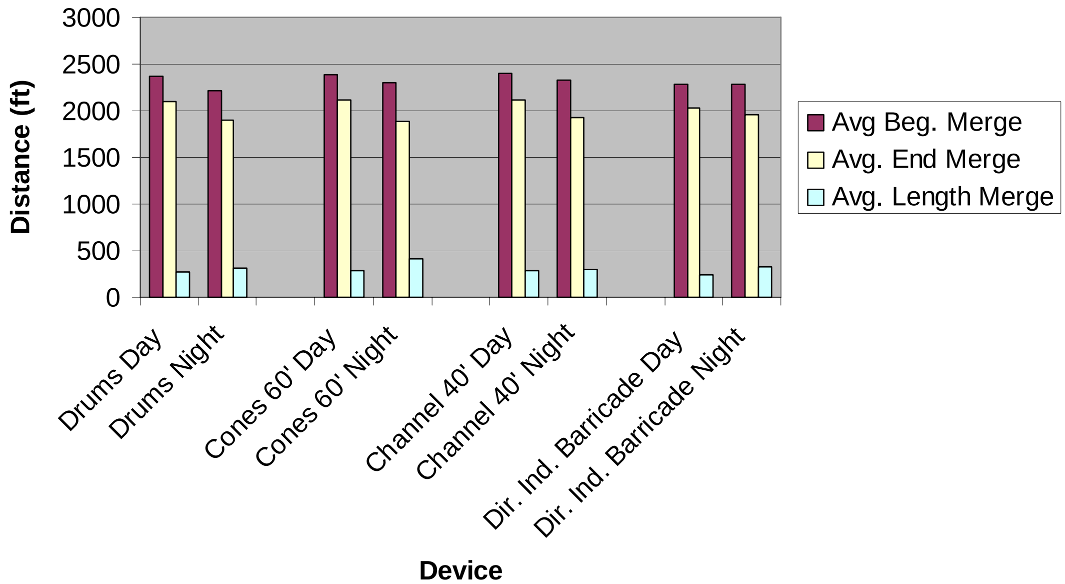


Figure 7. Average merge distance by device

Shy Distance

Shy distance is defined as the distance the cars are away from the channelizers at a designated point. Shy distance in this instance was estimated at the top of the taper by examining the video tape at reduced speed and calculating the distance from the designated spot, from 0 to 12 ft. (12 ft. representing an entire traffic lane). Shy distances were estimated for each treatment. An examination of the data showed little significant differences between each device. Most of the shy distances were between 6 ft. to 8 ft. from the center line at the top of the taper. Distances did increase at night, however. The graph in Figure 8 illustrates the shy distances.

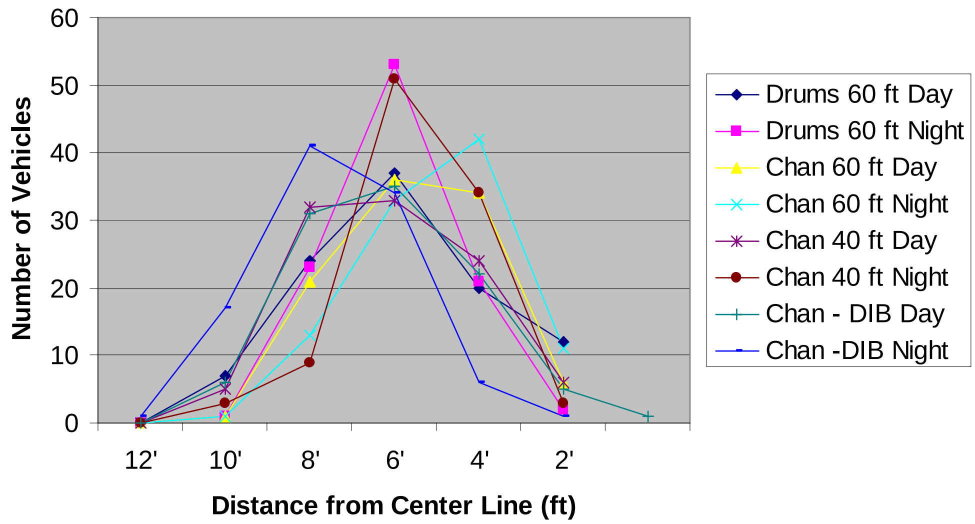


Figure 8. Shy distance by device

Table 2 is a sample of the traffic and describes the shy distance that cars moved away from the top of the taper. The table shows the number of cars by the distance they moved away from the center line. For example, when the drums were set up at 60 ft. during the day, at the peak, approximately 37 cars moved approximately 6 ft. away from the center line.

Table 2. Speed data, MM 142.71

65 mph posted	All vehicles	Pass. Cars	Trucks
EASTBOUND, NO TREATMENT			
Avg. Speed	68.7	69	67
85th % Speed	74	74	72
Standard Deviation	5.08	5.07	4.81
Minimum	24	31	24
Maximum	99	99	82
% >limit	79	80.6	69.2
% >5 over limit	43.3	46	26.4
% >10 over limit	9.7	10.7	3.8
% >15 over limit	1.8	2.1	0.3
% >20 over limit	0.3	0.4	0.0
EASTBOUND, 3:00–8:30 TREATEMENT			
Avg. Speed	68.7	68.9	67.1
85th % Speed	74	74	72
Standard Deviation	4.74	4.68	4.8
Minimum	26	26	26
Maximum	96	96	81
% >limit	79.1	80.6	68.4
% >5 over limit	42.8	44.6	29.9
% >10 over limit	8.7	9.4	3.3
% >15 over limit	1.5	1.7	0.1
% >20 over limit	0.4	0.4	0.0

Speed Data

One aspect of the data collection effort was to gather traffic speed data as well. Three sets of tube counters were placed on US-30 to gather the speed data (Figure 9). One counter was placed upstream, ahead of the work zone. The second counter was placed just under the Y Ave Bridge, approximately 1,000 ft. from the beginning of the taper. The third counter was placed approximately 1/4 mile from the end of the work zone. The data show a slight decrease in traffic speed through the work area.

Figure 9. Placement of tube counters

Table 2 shows the summary data from the first tube counter placed at mile marker (MM) 142.71 upstream of the work zone. The posted speed limit on this section of US-30 is 65 mph. For all vehicles, the average speed was 68.7 mph during the entire four days of data collection, with 79% of traffic traveling over the posted limit. During the treatment period of the data collection period, when the work zone set up, the average speed for all vehicles at this point was 68.7 mph, with 79% of traffic traveling over the posted speed limit.

Table 3 shows the data collected from under the bridge just prior to the work zone area, approximately 1,000 ft. from the beginning of the taper. The nontreatment periods show a slight increase in speed, an average 69.4 mph for all vehicles. The treatment periods show an average speed of 67.3 mph, with 68.2% of traffic traveling over the posted speed limit.

Table 3. Speed data, under bridge

65 mph posted	All vehicles	Pass. Cars	Trucks
EASTBOUND - NO TREATMENT			
Avg. Speed	69.4	69.7	67.8
85th % Speed	75	75	73
Standard Deviation	5.2	5.22	4.74
Minimum	23	23	25
Maximum	80	103	94
% >limit	82.2	83.7	73.9
% >5 over limit	50.6	53.6	33.9
% >10 over limit	13.3	14.5	6.4
% >15 over limit	2.5	2.9	0.69
% >20 over limit	0.6	0.7	0.1
EASTBOUND - 3:00-8:30 TREATEMENT			
Avg. Speed	67.3	67.6	65.4
85th % Speed	73	73	71
Standard Deviation	5.27	5.22	5.27
Minimum	25	37	25
Maximum	95	95	81
% >limit	68.2	70.1	54.6
% >5 over limit	34.4	36.5	19.7
% >10 over limit	6.1	6.7	1.7
% >15 over limit	1.1	1.2	0.3
% >20 over limit	0.2	0.2	0.0

Table 4 shows the speed data downstream from the work zone area. The data show an average speed of 69.2 mph, with 81.1 % of the traffic traveling over the posted speed limit. During the treatment periods, however, the data show an average speed of 61.7 mph, with only 32.2% of the traffic traveling over the posted limit. These data indicate that the traffic was slowing down within the work zone area.

Table 4. Speed data, MM 144.10

65 mph posted	All vehicles	Pass. Cars	Trucks
EASTBOUND - NO TREATMENT			
Avg. Speed	69.2	69.5	67.8
85th % Speed	74	75	73
Standard Deviation	5.19	5.22	4.79
Minimum	14	14	28
Maximum	100	100	99
% >limit	81.1	75.8	73
% >5 over limit	48.2	50.6	33.8
% >10 over limit	12.5	13.6	6.2
% >15 over limit	2.5	2.9	0.9
% >20 over limit	0.5	0	0.1
EASTBOUND - 3:00-8:30 TREATEMENT (1 LANE)			
Avg. Speed	61.7	61.9	60.4
85th % Speed	69	69	67
Standard Deviation	6.92	6.99	6.21
Minimum	26	26	36
Maximum	85	85	76
% >limit	32.2	33.4	22.7
% >5 over limit	11.8	12.7	5
% >10 over limit	1.4	1.6	0.1
% >15 over limit	0.1	0.2	0
% >20 over limit	0	0	0

CONCLUSIONS

The results are encouraging for the safety impact of using the trim-line channelizers for work zone traffic control. The analysis shows little difference in merging behavior between the traditional drum devices and the trim-line channelizers. The analysis also shows that there is a slight decrease in speed through the work zone when either device is used. Upon looking at the results from the analysis, the trim-line channelizers do not significantly change merging behavior; thus, these devices should provide ample guidance to drivers through the work zone.

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Temporary Traffic Control and Enforcement of Traffic Laws in Closed Road Sections

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ABSTRACT

Public travel by motor vehicles is often necessary in road and street sections that have been officially closed for construction, repair, and/or other reasons. This authorization is allowed to provide access to homes and businesses located beyond the point of closure. While the Manual on Uniform Traffic Control Devices (MUTCD) does address appropriate use of specific regulatory signs at the entrance to closed sections, direct guidance for temporary traffic control measures within these areas is not included but may be needed. However, interpretation and enforcement of common practices may vary among transportation agencies. For example, some law enforcement officers in Iowa have indicated a concern regarding enforcement and jurisdiction of traffic laws in these areas because the Code of Iowa only appears to address violations on roadways open to “public travel.” Enforcement of traffic laws in closed road sections is desirable to maintain safety for workers and for specifically authorized road users. In addition, occasional unauthorized entry by motor vehicles is experienced in closed road areas, causing property damage. Citations beyond simple trespass may be advisable to provide better security for construction sites, reduce economic losses from damage to completed work, and create safer work zones.

Key words: closed road sections—law enforcement—temporary traffic control

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BACKGROUND

The Code of Iowa does not appear to definitively address the issues of temporary traffic control and law enforcement on roads and streets closed to general public access during construction, maintenance work, or other activities. Enforcement powers are granted to an agency for “public roads,” which may be defined as roads open to travel by the public at large. When a road or street is officially closed, a technicality may exist that such a facility is no longer “public,” and therefore may not be subject to the same level of enforcement by state and local agencies. In addition, the agency and the contractor are protected from liability within the boundaries of a closed road by the Code (except for gross negligence), possibly further confusing the responsibilities and authority of an agency. Pertinent sections of the Iowa Code relating to temporary road closure are discussed in this report, along with relevant sections from the codes of surrounding states.

Based on the issues described above, a research study was conducted to investigate temporary traffic control and enforcement practices and policies of various agencies in closed road sections. The three primary objectives of the research were as follows:

1. Provide a synthesis of current practices and policies regarding temporary traffic control and enforcement of traffic laws in closed road sections.
2. Evaluate the needs of state, county, and municipal transportation agencies relating to potential changes in enforcement policy for closed road sections.
3. Recommend changes in temporary traffic control and enforcement policies and procedures to improve safety in road closure areas.

This report presents the results of the second and third objectives. The summary of practices, policies, and state code reviews (objective 1) is not reported here due to space limitations, but can be found in the final report published by the Center for Transportation Research and Education at Iowa State University. This study presents findings from surveys of department of transportation (DOT) staff in Iowa and in other states, Iowa law enforcement officers, and local agency personnel, and findings from expert panel discussions. The preliminary findings suggest that enforcement of traffic control and safety in closed road sections could be improved through possible code revisions, better communication of best practices, implementation of surveillance and control technologies, and development of an expanded driver education program.

INTRODUCTION

Public travel by road users is often necessary in road and street sections that have been officially closed for construction, repair, or other reasons. This authorization is permitted in order to provide access to homes, farms, and businesses located beyond the point of closure. The Manual on Uniform Traffic Control Devices (MUTCD) does address appropriate use of specific regulatory signs at the entrance to closed sections; however, direct guidance for temporary traffic control (TTC) measures and enforcement of traffic laws within these areas is not included but may be needed. Interpretation and application of common practices may vary among transportation agencies. For example, some law enforcement officers in Iowa have indicated a reluctance to enforce traffic laws in these areas because the Code of Iowa appears to address only violations on roadways open to “public travel.” TTC and enforcement of traffic laws in closed road sections is desirable to maintain safety for workers and authorized road users. In addition, occasional unauthorized entry by motor vehicles is experienced in closed road areas, resulting in property damage and potential liability for agencies and contractors. Citations beyond simple trespass may be advisable to provide better security for construction sites, reduce economic losses from damage to completed work, and create safer work zones.

This study presents data collected directly from law enforcement officers, municipal and county engineers, and state DOT staff describing their opinions and personal experiences with TTC and law enforcement in closed road sections. The data was collected through surveys distributed both through the Internet and during a focus group interview session with county engineers and staff on December 5, 2006. A comprehensive breakdown of the survey results and interview session findings is included.

An advisory committee was invited to contribute to the study by sharing experiences and offering suggestions for possible Iowa Department of Transportation (Iowa DOT) specification revisions and Code modifications. The advisory committee included staff from the Iowa DOT, the Iowa Governor's Traffic Safety Bureau, contractors, law enforcement officers, and city and county engineers. The committee met twice during the execution of the study, once near the initiation of research to discuss potential sources of information and later to discuss and respond to survey results and make suggestions for possible Code enhancements and Iowa DOT specification revisions. The results from the committee meetings are summarized in this report.

LITERATURE REVIEW AND EXISTING RESEARCH STUDIES

As part of this study, a literature review was performed to identify existing research and/or other references for guidance in applying TTC and enforcement in closed road sections.

Very few studies addressing these issues were identified. For example, Elias and Herbsman (2000) published the results of a study suggesting that legislation and programs at state and federal levels are emphasizing a need for increased study of work zone issues. This need is especially acute as it relates to road closures, since many transportation agencies shift resources from new infrastructure development to rehabilitation. A review of the academic literature in the fields of civil engineering found no studies or existing research dealing with the topic specifically. However, some notable studies have been performed on the related topics of TTC, temporary road closures, and lane closures. A field evaluation of late merge traffic control in work zones was performed by Beacher (2005), which only dealt with two-to-one lane closure and not full closure. Zech (2005) performed an evaluation of rumble strips and police presence as speed control measures in highway work zones and concluded that police presence with rumble strips decreased vehicle speeds greatest; however the highway was under traffic and not closed to through traffic. Pre-announced temporary closures were modeled by Tong (1998) to generate optimal routes for trips in road networks operating at capacity; however, the issue of traffic control enforcement was not mentioned. A study of urban work zone traffic management by McGuinness (1997) in the City of Columbus, Ohio, explored practices such as closing freeway ramps and approach roads to work areas to siphon off excess demand, providing alternate routes, and using a traveler information program, but the study does not address the issue of enforcement.

Interestingly, some studies have relevance to TTC in closed road sections because they primarily examined detection and surveillance on open roads. The subject of online object tracking for color video analysis was described by Lannizzotto (2002) as a possible use in traffic control because video sequences can track shapes, positions, and the orientation of objects. Use of closed circuit television systems to monitor/detect urban traffic and as a control device was explored by Franklin (1999). Harrison and Lupton (1999) discussed the development of the Automatic Road Traffic Event Monitoring Information System (ARTEMIS), a computer monitoring system that can detect traffic events and dispatch patrol cars. However no studies were found that specifically addressed the issue of closed road sections and enforcement of traffic laws therein. Therefore, the current study will make an important first step in analyzing the issues of traffic control and enforcement in closed road sections.

SURVEY RESULTS AND INTERVIEW SESSION FINDINGS

To determine the extent and severity of problems and issues associated with road closures, the research team created and distributed a survey at the Iowa County Engineers Association annual meeting and via the Internet. Similar surveys were distributed to cities and selected Iowa DOT staff. The county engineers' survey consisted of 16 questions intended to help the team gain a more detailed understanding of the problems and issues that arise in closed road sections. A summary of the survey responses is provided below; the results have been divided into groups by responding agency (e.g., municipal, county, state).

The survey results from 34 responding municipalities found that 61% of respondents had experienced enforcement and/or traffic control problems in closed street sections, and 41% reported property damage resulting from unauthorized entry into these sections. The most commonly reported problem was damages to finished surfaces and slopes. Methods used by cities for addressing closed road sections included delegation or independent contracting and/or discussion at preconstruction meetings. Most respondents reported that any law enforcement used was reactive, after damage had already occurred. Solutions or suggestions for improvements offered by municipalities included improved signing and traffic control maintenance and improved contractor procedures and personnel training.

Survey results from 75 responding county engineers and staff showed that 90% of respondents had experienced enforcement and/or traffic control problems in closed road sections, with 54% reporting property damage resulting from unauthorized entry into those sections. The most commonly cited problems included damage to finished surfaces and slopes, theft of signs and barricades, low public awareness of important road work issues, and worker safety exposure. Mitigation strategies offered by the county engineers and staff were similar to those of municipalities, with an additional suggestion for a program to raise public awareness of issues.

The survey results from Iowa DOT staff indicated that 80% of respondents have experienced enforcement and/or traffic control problems in closed road sections, with 60% reporting damages resulting from unauthorized entry. The most commonly reported problems included damage to local property, risk management issues, and worker safety. To address concerns, preconstruction meetings were the preferred method of mitigation.

Law enforcement officers were surveyed with different questions than those given to municipalities and counties. The law enforcement survey asked officers to identify the Iowa Code sections they felt were most appropriate for enforcement in closed road sections, as well as those that should be clarified for better understanding. Officers were also asked whether they had answered a call in a closed road section and, if so, whether a serious accident had occurred. The officers were also asked if they were aware of any specific occurrences of court cases involving an interpretation of Iowa Code in closed road sections.

The results of 180 law enforcement officer surveys revealed that 87% had responded to a call in closed road sections one or more times, with 29% reporting that those calls involved a serious accident. Officers identified the most appropriate sections of the Iowa Code as 306.41, 321.1, and 321 (.228, .232, .252, .256, .260, .285, and .288). These sections primarily regard jurisdiction, signage, and driver conduct and compliance. The survey results indicate that most officers feel there are few problems with interpretation of the current Iowa Code; however, some modifications could better clarify the intent in the sections listed above.

A focus group interview session with county engineers and staff occurred on December 5, 2006, on the Iowa State University campus. In attendance were seven county engineers, one assistant county engineer, one county technician, and one representative from the Office of Local Systems at the Iowa DOT. The session lasted approximately 90 minutes and was facilitated by Dr. Kelly Strong and Tom McDonald of the Center for Transportation Research and Education. The facilitators provided four questions to stimulate discussion among the group. The questions covered the following topics:

- Perceived liability exposure
- TTC and law enforcement for construction, and their current levels of sufficiency
- Communication methods with and between contractors, enforcement agencies, emergency responders, local residents and businesses
- Opinions on modifying existing Iowa Code, Iowa DOT specifications, and MUTCD standards

The focus group consensus indicated that most concern for liability exposure or actual liability exposure involves signs and barricades. Barricades and protective fencing are often moved or vandalized, thus exposing finished work that may not be completely cured to vehicular traffic. Signs and warning lights that have been stolen, knocked down, or covered in dirt are sometimes not addressed in a timely manner, possibly exposing the agency to liability.

Project communication methods for road closures noted by the focus group included preconstruction conferences that involve local enforcement officers and the Iowa DOT. Advice for residents and businesses might be provided through letters, news media, and/or local radio segments. Coordination of postal delivery, school, or emergency routes must be planned ahead of time and implemented once the closure signs have been erected.

Suggested modifications to Iowa Code from the focus group included increasing fines and penalties for sign theft, removal, or damage. However, it was also stated that local magistrates and judges are often reluctant to impose the current penalties and fines. Another suggestion was to increase sign credibility by covering the road closure signs until the date of closure. Covering signs until needed improves message credibility. “Road closed” signs are sometimes installed before the route is actually impassible, which encourages drivers to ignore the signs, which can later result in damages to newly constructed work from unauthorized entry. It was also suggested that increased penalties for contractor noncompliance with temporary traffic control requirements may be necessary in the Iowa DOT specifications. A minimum response time for sign repair by contractors with penalties for delays may improve speed of response time for needed repairs.

ADVISORY COMMITTEE DISCUSSION AND SUGGESTIONS

Several experienced professionals from a variety of disciplines were invited to contribute to this research effort by sharing advice, opinions, and suggestions for needed improvement regarding the topic of closed road traffic control and enforcement. Committee members, who were listed previously in this paper, met twice during the progress of this study.

Advisory committee members were provided with an overview of the project, summary of literature (including Code provisions from Iowa and surrounding states not presented here), and survey summaries, and they were then asked to suggest ideas for possible Code, specification, and /or policy changes.

One member of the committee noted that road closure may not be a definitive issue, and there may be “degrees of road closures” and varying types of risk depending on the location, service level of the road, etc. For instance, low-volume roads may not need to be signed as extensively during construction as higher volume facilities. In addition to varying types of “road closures,” there are also instances where the situation may change during the project life cycle, and when drivers find that a road section is signed as closed but is usable (for example, paving complete but guardrail not yet installed), it becomes very difficult for the contractor and agency to restrict entry, which can increase potential liability exposure.

Technology solutions such as controlled access gates, surveillance cameras, and video logs can assist agencies and contractors in managing risk on closed road sections. Additionally, effective strategies used in some jurisdictions include specific assignment of a deputy to issue citations to unauthorized traffic; preconstruction conference planning; ongoing coordination and cooperation with law enforcement; working with contractors on best practices for preventing, repairing, and recovering any damages; and use of proper MUTCD TTC.

In addition to road closures for construction and maintenance, similar issues may exist for special event road closures, such as the popular Register’s Great Bicycle Ride Across Iowa (RAGBRAI), street festivals, parades, etc. Because those in charge of special event closures may not have access to TTC expertise, good practice may be unknown, thereby increasing exposure to safety concerns and liability.

Section 306.41 may be the most relevant section of the Iowa Code pertaining to road closures. The liability waivers described in that section provide risk mitigation for agencies and contractors, except in the case of gross negligence. Gross negligence might occur when an agency or contractor fails to follow good practice and TTC prescribed in the MUTCD and project specifications. An example of gross negligence would be if an agency installs a “Road (Street) Closed” sign and allows road users to pass that point. When entry is allowed for property owners and businesses, the proper signs are “Road Closed to Through Traffic” or “Local Traffic Only.” The definition of gross negligence is determined through the discretion of the court, but the requirement for agencies to follow the state manual is clearly stated in the Iowa Code.

Committee members surmised that, in the case of property damage (including damage to completed work), it can be difficult to determine and/or locate the responsible individual. In addition, since fines are minimal and the Code does not provide definitive guidelines, it is often deemed not worthwhile to prosecute violators.

The most common process for initiating road or street closures is for the city council or board of supervisors to adopt a resolution and proper temporary traffic control designed by county or municipal engineers. The MUTCD recommends that TTC be inspected regularly, but there is ambiguity on the exact frequency. Even so, frequent inspections are a common interpretation, especially for overnight closures. Contractors or subcontractors are generally required by the project documents to provide and maintain prescribed TTC, including timely inspections and repairs. This can be problematic at times, especially with frequent unauthorized entry and the need for weekend surveillance. Some contractors frequently hire agency workers or other local personnel to check signage everyday, typically after hours.

The committee noted that there is currently no standard form for documenting surveillance, and some individuals have expressed opinions that log requirements should be eliminated. It is difficult to establish the credibility or performance effectiveness of a person hired to check signs and file logs. Also, the line of authority for needed sign repair is not always clear. A project contract involves the agency and prime contractor; however, it is common for subcontractors to perform TTC. Some county engineers want a minimum response time for needed sign repair and replacement, similar to that described in the Iowa DOT Standard Specifications.

Remote sensing for damage to important signs may be another option for promoting more effective TTC. A sensor could send a signal to law enforcement dispatchers, who could contact the contractor's representative to replace or reset the device.

It was noted that a surveillance system may not be difficult to implement and may help solve some of the problems with TTC inspections, documentation, identification of trespassers and vandals, etc. Construction companies frequently make use of daily video logs to manage project risk for nonpublic projects, but privacy and other guidelines may be different for publicly funded improvements. Night vision cameras could be used to help identify unauthorized individuals or vehicles entering closed road sections or vandalizing signs and barricades. For continuous surveillance, a stationary camera might be helpful as a complement to a video log of all of a project's TTC. For small projects (i.e., structure replacement), it may be possible to use entire scene cameras; for larger projects, the cameras could be focused on specific points of entry to the project.

The feasibility of a surveillance system should be further investigated. In spite of some legal issues needing resolution, the use of video surveillance is generally increasing. In Alaska, law enforcement must record all interviews, and courts are increasingly accepting the use of video logs as evidence. Unless identification of violators can be ascertained with certainty, the use of surveillance camera images as evidence may be problematic. However, the known presence of video surveillance on a project may very well be a deterrent for unlawful activities.

Another area warranting further investigation is the penalty for citations in closed road sections. Stiffer penalties, enforcement, and consistent prosecution of perpetrators may help reduce the problems currently experienced in closed road sections.

Committee members noted several Iowa Code sections where clarification of application in closed road sections might be beneficial. Sections 321.260 and 321.285 were specifically mentioned. Language could be added to the Code clarifying that statutes "apply to public roads, unless otherwise noted, open or closed" in Section 321.228. This would provide a clearer definition of "highway" that would include closed road sections. Clarification of certain Code sections could help improve consistency between jurisdictions. However, it was noted that even if infractions are provable, penalties are applied by a local judge or magistrate, and thus penalties for similar offenses vary widely across the state. Trespassing is a common citation, and this can be applied to unauthorized entry, whether on foot, in a car, or in an off-road vehicle. The process used in Linn County, Iowa, appears to be effective and could serve as a model for enforcement of authorized entry to closed road sections.

One county engineer on the committee reported over $1 million in costs due to vandalism over the years (not just in closed roads). A recommendation from this study is to gather information about the estimated cost of damages due to unauthorized entry in closed road sections. These data could be used in conjunction with a technology feasibility study, mentioned earlier, to compare costs and benefits.

The committee also suggested adding language to the Iowa DOT Standard Specifications indicating that the contractor is responsible for communicating and coordinating access issues with the public. In addition, past performance in TTC compliance could be included in contractor evaluation criteria.

Another suggestion from the advisory committee was to include pertinent work zone traffic control issues, including closed road requirements, in driver education programs. Public awareness of these important issues could prove beneficial in reducing crashes and violations in work zones. More support and information for driver education may be effective.

The committee recognized that the ultimate responsibility for safety in closed road sections must rest with the agency in charge. However, contractors must fulfill their obligations to provide and maintain quality TTC for all work zones areas, including closed roads and streets. Communication and cooperation between agencies and contractors is mandatory in this effort.

CONCLUSIONS

To conduct this research, information was gathered from many sources using several different methods. A literature review identified the few existing studies or references with specific relevance to the subject of law enforcement and temporary traffic control for closed road/street sections. Personal interviews with county engineers/staff, as well as expert opinions and guidance from an advisory committee, were also a part of this study and proved very beneficial. From these sources, the following conclusions can be drawn:

- Allowing limited public traffic on road or street sections that have been officially closed for construction, maintenance, or other special events is not uncommon in most agencies. Most state codes prohibit denial of lawful access to property without just compensation.
- Requirements and guidance for application of TTC for public travel in closed road/street sections is minimal in state DOT specifications and the MUTCD.
- Many public agencies have experienced problems with unauthorized traffic in closed road/street sections, with the most serious commonly cited problems including damages to contractor work, theft, and vandalism. Significant and costly damages to finished work have occurred for many agencies.
- Most law enforcement officers do not feel the current Iowa Code reduces their authority to issue citations for traffic violations in closed road/street sections, but some modifications to provide clarification to certain sections would be beneficial.
- Many local agencies in particular would recommend strengthening current specifications to better clarify contractor responsibilities for TTC in closed road areas and raising penalties for nonperformance.
- Some counties receive good support from the sheriff’s office in monitoring the security of closed road sections and issuance of citations for unlawful entry.
- Public awareness of temporary traffic control requirements and procedures for work zones in general and closed road sections in particular could be improved.
- The current Iowa Code contains several provisions and penalties for violations of established traffic control, as well as theft/vandalism of traffic control devices, but enforcement and application of penalties vary widely across the state.

RECOMMENDATIONS

To address concerns and problems identified with law enforcement and temporary traffic control in closed road and street sections, the following recommendations are offered:

- Amend the MUTCD, Part 6, Temporary Traffic Control, to describe recommended TTC for closed road/street sections where authorized public travel is allowed.
- All agencies should ascertain that staff are familiar with and comply with MUTCD requirements for the use of “Road (Street) Closed” signs, especially Section 6F.08.
- Add language to state DOT specifications to require adequate TTC and protection for equipment in closed road/street sections where public travel is allowed. The TTC should closely replicate the expectations for open roadways (e.g., obstacles and hazards should be adequately delineated and/or protected, especially for nighttime hours).
- Agencies should strive to develop cooperative working relationships between transportation and law enforcement agencies to assure that TTC is properly designed, deployed, maintained, and enforced in work zones. This topic should be included in agenda issues for preconstruction conferences.
- Investigate the feasibility of technology solutions to control vandalism and unauthorized travel in closed road sections, including surveillance cameras.
- Provide information and data for driver education programs addressing safe travel through work zones, with a segment on authorized travel in closed road and street sections. This information could be furnished to driver education instructors or provided on a video for viewing at driver’s licensing stations.
- Revise appropriate sections of the Iowa Code to better clarify intent for enforcement in closed road/street sections, specifically Section 306.41 and 321.1 (78) to expand the definition of “Street” or “Highway” to include closed road/street sections open to authorized traffic.
- Develop best practice guidelines for temporary traffic control in closed road/street sections for distribution to state, county, and municipal transportation and traffic managers. The guidelines should include a description of the regulatory process for official establishment of closures, as well as suggestions for effective temporary control of authorized traffic during construction and maintenance activities or special events.
- For the purpose of future research, the Iowa DOT should consider characterizing crashes that occur in and tort claims arising from incidents that occur inside of closed road/street sections. Current databases do not include information with this specificity.
- Conduct a survey of state and local agencies to obtain an approximation of the actual cost of theft, vandalism, and damages from both authorized and unauthorized traffic in closed road/street sections. Publish the results for better agency and public appreciation for the scope of concern.

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Embankment Construction QC/QA using DCP and Moisture Control: Iowa Case History for Unsuitable Soils

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ABSTRACT

The Iowa Department of Transportation (Iowa DOT) has developed an end-result embankment construction specification, Quality Management Earthwork (QM-E), that utilizes the dynamic cone penetrometer (DCP) for quality control and quality assurance (QC/ QA) of fill compaction. The DCP measures the shear strength of a soil compaction layer by penetrating a conical tip attached to a shaft through the soil with successive drops of an eight kg mass. The QM-E specifies a maximum depth of penetration per blow of the DCP for acceptance of lift compaction for three different Iowa DOT soil grades: (1) select, (2) suitable, and (3) unsuitable. The current criteria for suitable and unsuitable soils is 70 mm/blow and was developed from a limited data set containing very few tests in unsuitable soils. In an attempt to better develop improved DCP acceptance criteria for unsuitable soils, a pilot project was selected. This paper presents the application of the QM-E to the construction of the US-34 bypass in Fairfield, IA. QC/QA testing was conducted throughout all phases of construction, including DCP, moisture, and density tests. The QM-E was successfully implemented at the pilot project. Modifications to the QM-E special provision based upon observations and experiences at this project are described in this paper.

Key words: compaction—dynamic cone penetrometer—embankment—moisture content—unsuitable soils

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INTRODUCTION

After a series of earthen embankment failures during the mid 1990s, the Iowa Department of Transportation (Iowa DOT) decided to investigate the possibility of improving their embankment construction practices. Research conducted by Iowa State University found that poor embankment quality was often linked to one or more of the following occurrences (Bergeson 1998):

- Inadequately trained personnel who, at times, misidentified the type of fill material, leading to fill misplacement
- Compaction of overly thick lifts
- Fill material placed and compacted at moisture contents well in excess of standard proctor optimum moisture content
- Field classification testing that had formerly relied too heavily on one-point proctor tests, while such tests are inadequate for certain soils for the determination of optimum moisture-density characteristics.

Based upon these findings and the results of two additional phases of research (White 1999; White 2000), the Quality Management Earthwork (QM-E) special provision was developed. The QM-E is unique in that it is an end-result specification that provides the contractor with greater flexibility in the methods and equipment used during construction, provided that the finished product meets the applicable acceptance criteria. This type of specification has the potential to reduce construction costs and time through contractor innovation.

PROBLEM STATEMENT

The Iowa DOT was interested in implementing a new embankment construction specification that would result in improved embankment quality. Though the QM-E special provision was seen as having great potential, it was largely untested, especially on projects in predominately “unsuitable” soil. Therefore, a full-scale pilot project was conducted to gain insight into the practicality of applying the special provision for these types of soils. The quality control and quality assurance (QC/QA) test data from the project were then used to refine the QC/QA testing requirements of the special provision.

QUALITY EARTHWORK MANAGEMENT PROGRAM

The QM-E aims at improving embankment quality by reducing or eliminating the problems, mentioned above, that are linked to poor performance. This is accomplished through three main components of the specification: inspector training/certification, QC/QA testing requirements, and test strip construction.

Inspector Training/Certification

The QM-E special provision required that all QC/QA personnel, both contractor and Iowa DOT, must first complete a five-day Certified Grading Technician Level I training course and examination. This training course sought to impart basic skills that aid in the field identification of soil and in properly conducting the required field and lab testing.

QC/QA Testing

The QC/QA requirements of the specification constituted the most significant changes in comparison to the former specification. Moisture content, lift thickness, density, stability, and uniformity are the key criteria used to evaluate embankment quality during the construction process. The QM-E requires that these tests be conducted at a minimum frequency and that necessary control limits are met, varying with the classification of the fill material.

Moisture Content

The moisture control limits specified by the QM-E for the pilot project were +/- 2.0% of standard Proctor optimum moisture content for all material, unless otherwise specified. These control limits could potentially be changed from project to project depending on the circumstance.

Density

The density control limits for fill material must not be less than 95% standard Proctor maximum dry density. Future versions of the QM-E specification may rely solely on moisture and in situ strength testing with the dynamic cone penetrometer (DCP).

Stability and Uniformity

The stability and uniformity of an embankment lift is measured by testing with the DCP (Figure 1). DCP tests are conducted by driving a 20 mm diameter, 60° cone into the ground under the force of an 8 kg hammer being dropped 575 mm (ASTM D 6951-03).

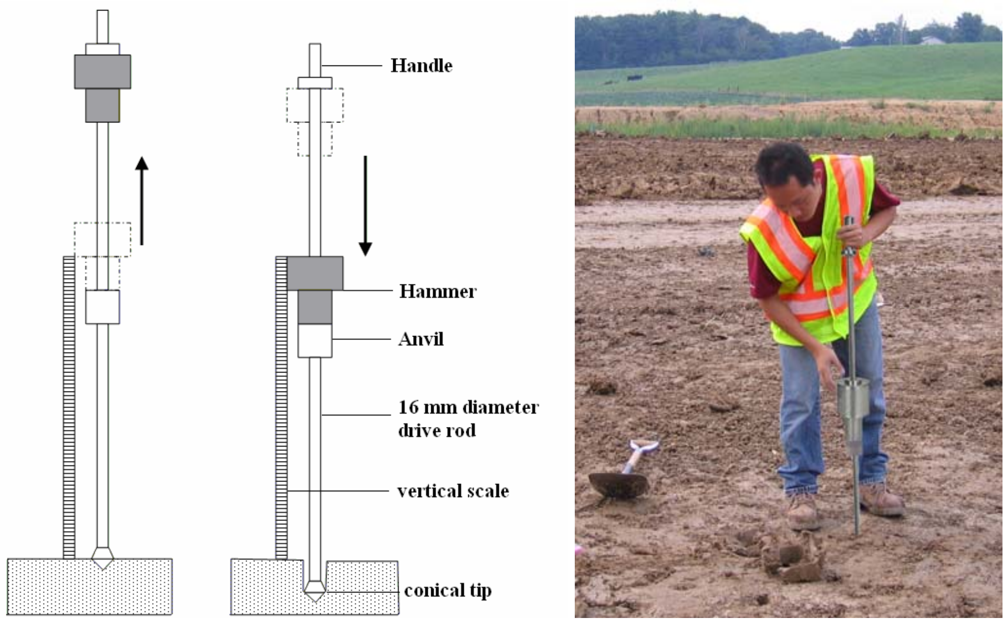

Figure 1. Dynamic cone penetrometer

DCP measurements are reported in millimeters of penetration divided by the number of hammer blows, and are referred to as DCP indices, recorded over a desired test depth. Figure 2 shows two plots of DCP index vs. depth for sets of hypothetical DCP readings. These plots give a lot of information about the soil profile. However, for convenience it is helpful to reduce this data to a single average DCP index value. There are various ways to attain an average DCP index for a given profile. The QM-E uses a weighted average method, calculated in accordance with Equation 1:

$$\text{Average DCP Index} = \frac{1}{H}\sum_{i=1}^{n} d_i{}^2 \tag{1}$$

where n is the total number of blows, d_i is the penetration distance for the i^{th} blow, and H is the depth of the desired test layer. Graphically, this is represented as the gray shaded areas in Figure 2B. A low DCP index, 25 mm/blow, is typical for stiff soils, and a high DCP index, 100 mm/blow, is typical for soft soils.

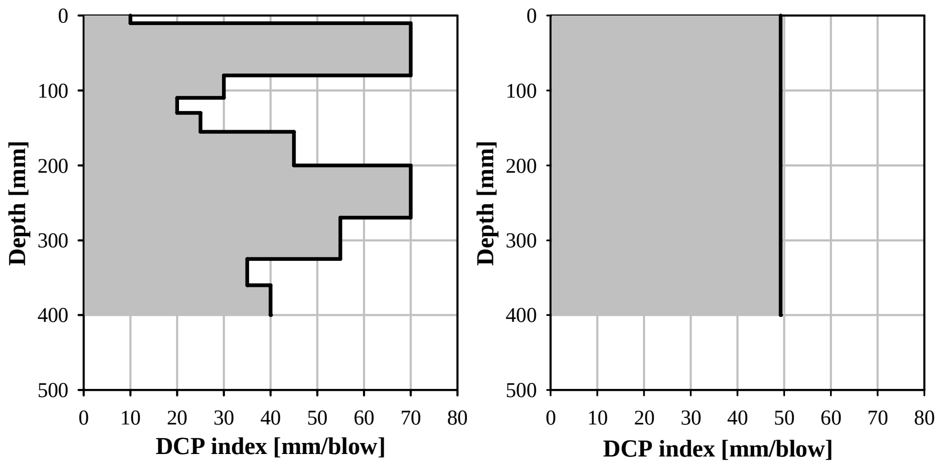

Figure 2 . DCP depth profile A (right) and B (left)

By reducing the test data to a single value, there is some degree of information lost. Figure 2 shows two different DCP profiles with the same average DCP index of 49.3 mm/blow for a test layer of 400 mm. However, both profiles are clearly very different: the first profile is more variable than the second profile. The uniformity value or variation in DCP index is used to represent this variability. The uniformity of a lift is calculated in accordance with Equation 2:

$$\text{Variation in DCP Index} = \frac{1}{H}\sum_{i=2}^{n} \left| d_i - d_{i-1} \right| \cdot d_{i-1} \tag{2}$$

where, once again, n is the total number of blows, d_i is the penetration distance for the i^{th} blow, and H is the depth of the desired test layer. Figure 3 shows the plot of variation in DCP index for the DCP profile shown in Figure 2 with depth for the DCP index in Figure 2A. Thus, over a depth of 400 mm, the variation in DCP index or uniformity value is 26.3mm/blow. The uniformity value is most convenient for detecting the “Oreo cookie” effect, whereby lifts of soil alternate between hard and soft due to overly thick lift compaction. Ideally, the uniformity value for each lift would be as close to 0 mm/blow as possible; however, this is almost never the case, and it is fairly common to have uniformity values ranging from around 5–20 mm/blow for well-compacted fill.

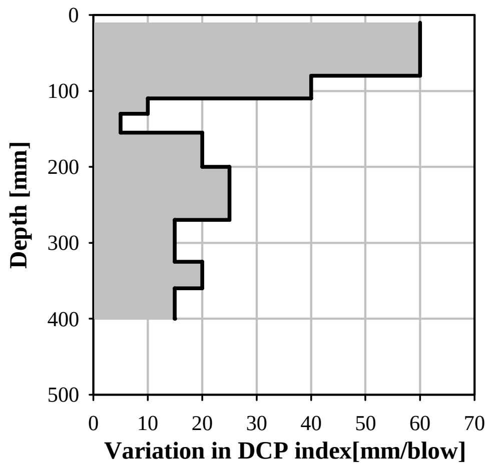

Figure 3. Uniformity or variation in DCP index for depth profile

The QM-E special provision sets the maximum stability and uniformity values acceptable for adequate lift compaction based upon the Iowa DOT borrow material soil type and grade classification (Iowa DOT Standard Specification 2102.06). See Table 1.

Table 1. QM-E stability and uniformity control limits for each soil type and grade

Soil Classification		Average DCP Index (mm/blow)	Variation in DCP Index (mm/blow)
Cohesive	Select	65	35
	Suitable	70	40
	Unsuitable	70	40
Cohesionless	Select	35	35
	Suitable	45	45

It is important to note that these control limits only apply explicitly for each test in test strip construction, as explained below, and to the four-point running average of all other QC/QA tests. This method is used to make the special provision more practical in the field to account for natural soil variability.

Test Frequency

The QC/QA testing for lift thickness, moisture content, and DCP tests are conducted at the same location and measured for each lift of embankment being placed. In addition, occasional material classification testing is conducted. The QM-E requires that all of the QC/QA testing maintain a minimum testing frequency, as shown in Table 2.

Table 2. QM-E required testing frequency

Test	Minimum Test Frequency
Lift Thickness Moisture Content DCP	Concurrently every 500 m^3
Determination of soil classification standard proctor maximum dry density, and optimum moisture content	Every 20,000 m^3

Iowa DOT Quality Assurance

Iowa DOT QA tests are conducted on a split sample at the exact location as the contractor's quality control test. In the event comparison test results are outside the above allowable differences, the engineer will investigate the reason immediately. The engineer's investigation may include testing other locations, reviewing observations of the contractor's testing procedures and equipment, and comparing test results obtained by the contractor with those obtained by the contract authority.

Iowa DOT QA testing will equal 10% of the testing conducted by the contractor. Testing will be acceptable if the QC and QA tests are within the acceptable ranges contained in Table 3.

Table 3. Acceptable range between QC/QA

Test	Acceptable difference between QC/QA
Moisture content	1.0%
Density	80 kg/m3 (0.8 kN/m3)
Optimum density and optimum moisture	80 kg/m3 and 1.5%

Control Charts

The QC/QA data is recorded in the form of control charts. Control charts contain data from a particular test versus a running test count for a particular area or soil type. These figures are convenient for visualizing QC and QA test data, as well as the failure criteria for the given parameter. In addition to containing each point measurement, these charts show the four-point running average of the test measurements, which is used as the acceptance/failure criterion for all testing. Control charts of data from the pilot project are discussed later.

Test Strip Construction

Test sections are compacted fill areas that are incorporated into the embankment and measure 50 m long, 10 m wide, and one lift thickness deep. They serve to establish proper rolling patterns, number of roller passes, and lift thickness required to attain acceptable compaction. Upon completion of a test section, four random locations are tested for lift thickness, moisture content, density, DCP index, and variation of DCP index. The test section is acceptable if all moisture content tests are within the specified control limits, all density measurements are greater than 95% standard Proctor dry density, and the DCP index and variation of the DCP index are within the specified control limits. Provided that the test section meets these criteria, construction of subsequent lifts with the same fill material should utilize the same techniques determined from the construction of the test strip. Embankment quality is verified with random QC/QA testing throughout the construction process. New test sections are to be constructed if there is a change in soil type or soil compaction methods/equipment or if QC/QA testing reveals that embankment lifts are not meeting the necessary quality standards.

PILOT PROJECT

The pilot project NHSX-34-9(96)-3H-51 was one of the US-34 bypass construction projects near Fairfield, IA (Figure 4 and 5). This project involved construction of the eastern portion of the bypass and was awarded to CJ Moyna and Sons, Inc. of Elkader, IA.

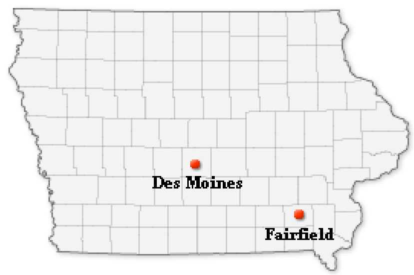

Figure 4. Map of Fairfield, Iowa

The construction spans approximately 4.6 km, and the plans call for the construction of three bridges, represented as gray rectangles; five bridge embankments; and four ramp sections, labeled A–D (Figure 5). This area of Iowa has an abundance of unsuitable soil in layers of weathered loess and Yarmouth Sangamon paleosol.

Figure 5. QM-E Pilot project, Iowa DOT project NHSX-34-8(96)-3H-51

Construction began in April of 2006 and halted in December 2006 for winter. QC/QA data was collected independently by the contractor, the Iowa DOT, and Iowa State University over this time.

ANALYSIS OF RESULTS AND KEY FINDINGS

The pilot project met the goals of providing a practical application for the QM-E provisions because considerable construction with unsuitable soil was required. Conditions throughout construction were very difficult due to relatively frequent rain and the poorly draining soils on the site. However, no significant delays were directly attributed to the QM-E provisions.

Figure 6 shows the stability, uniformity, unit weight, and moisture control charts from contractor and Iowa DOT data for an unsuitable soil at this project. There are many charts similar to this for other soils on the project; however, the general trends among them are similar.

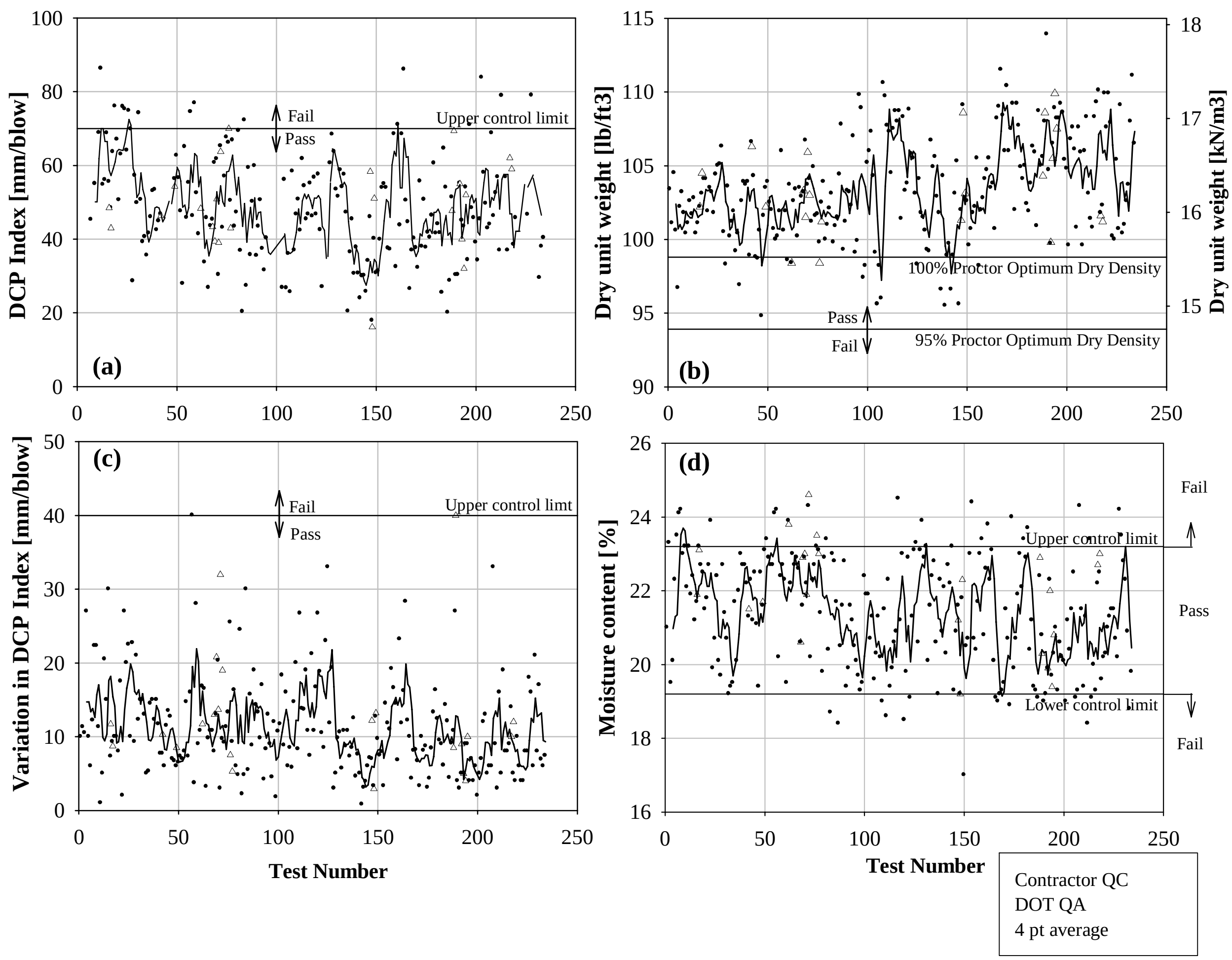

Figure 6. (a) Stability, (b) unit weight, (c) uniformity, and (d) moisture control charts for unsuitable soil

The DCP index control chart shows that observed DCP index values range from as high as 72 mm/blow to as low as 18 mm/blow, with an average DCP index of 49.3 mm/blow for all the testing. The four-point running average of DCP index did not exceeded the upper control limit of 70 mm/blow, and only a handful of single tests exceeded this limit. These observations show that at least the control limit of 70 mm/blow is not overly restrictive in unsuitable soil. In fact, if anything the control limit may be overly conservative. Regardless, the DCP test still provides records of fill strength after compaction placement, information that is not attained with density or moisture testing alone. Furthermore, density and moisture control QC/QA practices rely heavily on the results of Proctor tests. DCP test data alone are very useful for detecting fill instability that results from undercompaction, overly thick lifts, and compaction of soil well in excess of optimum moisture. Even more importantly, this information is available the instant the test is finished.

The unit weight control chart shows that the dry unit weight values ranged from as low as 14.9 kN/m^3 to as high as 18.0 kN/m^3, with an average dry unit weight of 16.2 kN/m^3 and optimum dry unit weight being 15.5 kN/m^3. The dry unit weight measurements almost always exceeded 100% Proctor optimum dry unit weight, and in some cases the measurements exceeded 110% proctor optimum dry unit weight. This may be the result of a change in the material properties that occurred after the classification tests were conducted. However, considering the trend in the DCP index was to decrease, indicating higher stability, it is of less concern.

The variation in the DCP index control chart shows that all but two of the tests were well below the control limits. One reason that these values tended to be so low is that the uniformity parameter was created to detect the “Oreo cookie” effect. This effect is more noticeable over multiple lift layers; however, the QM-E only requires the DCP test to be conducted for a single lift, and thus the variation in DCP index values tend to be low.

In general, the contractor focused most on conditioning the soil to within the proper moisture control limits. The contractor felt that as long as the soil was compacted within the proper moisture range the dry unit weight and DCP testing would pass the acceptance criteria. The moisture control limits were only exceeded twice near the beginning of construction, and the problem seems to have been corrected.

Comparisons of contractor QC and Iowa DOT QA testing reveal that the average differences between measurements at the same location for DCP index, variation in DCP index, moisture content, and dry unit weight were 7.6 mm/blow, 6.4 mm/blow, 0.8%, and 0.17 kN/m^3, respectively. These differences are within the range of acceptable variation specified by the QM-E.

Figure 6 also seems to show some trends with time. The DCP and unit weight control charts seem to show that the unsuitable soil got stiffer and denser as the project proceeded. Even the moisture and uniformity charts seem to show that the soil became more uniform and tended to have slightly lower moisture content later in the project. It is difficult to explain all of these occurrences; however, it seems probable that the fill materials properties changed at some point, especially considering that the dry unit weight measurements regularly began to exceed 105% relative compaction. This example illustrates two very important points regarding the QM-E. First, the control limits for moisture and dry unit weight are solely dependent on the results of Proctor testing. While the QM-E sets guidelines regarding the frequency of this type of testing, it is the engineer’s and quality control technician’s responsibility to determine whether or not additional testing is required. Finally, the control charts are very helpful tools for deciding when additional testing may be required. If the control charts show a systematic change in the values on any or all of the charts, it likely indicates that some sort of change has occurred, whether this is in soil properties or the moisture content of the soil at compaction. These changes may not always be detrimental, but the engineer must make a judgment call based upon the available data as to whether some sort of corrective measure is required.

Figure 7 shows the distributions of the contractor data. This data is the same as the data shown in Figure 6. The distributions are useful for looking at the frequency of each measurement.

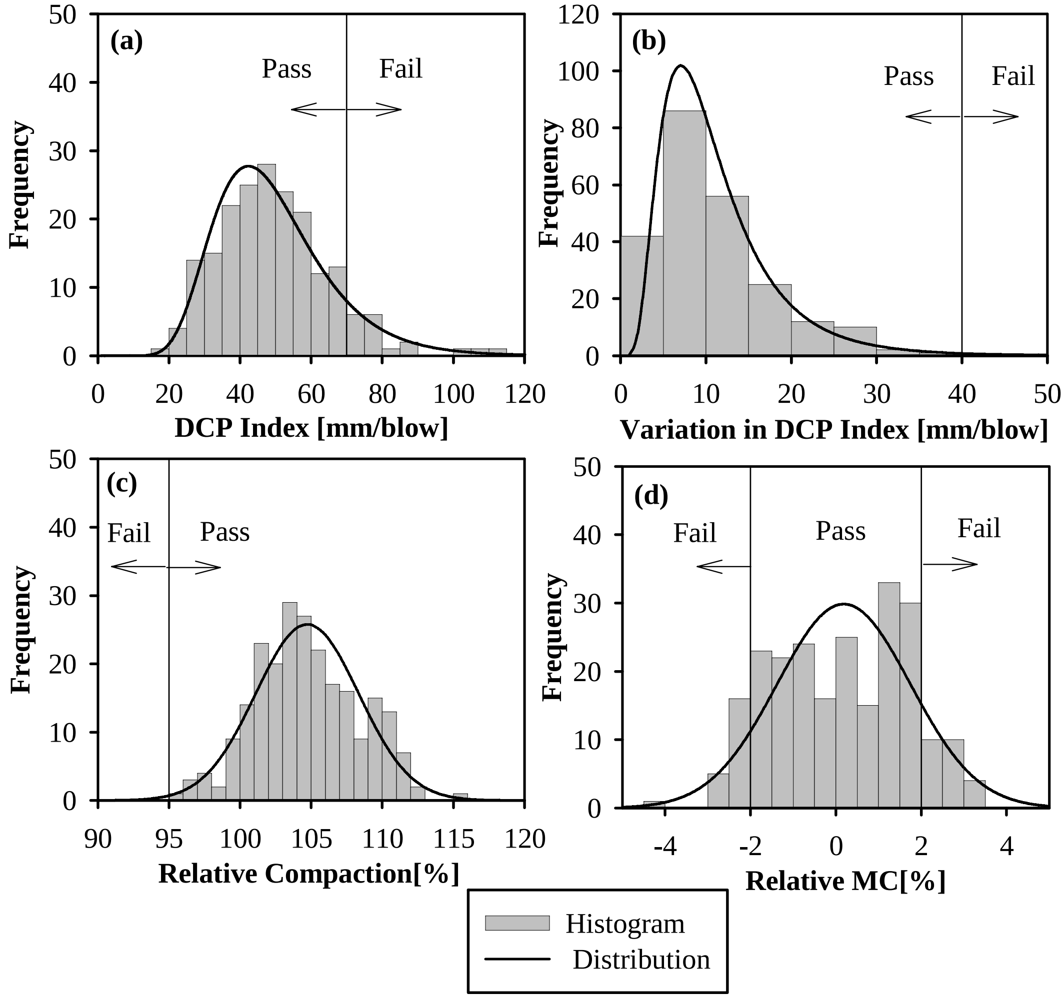

Figure 7. (a) DCP index, (b) variation in DCP index, (c) relative compaction, and (d) relative moisture distribution

The distribution of DCP index values shows that 12.7% of the testing exceeded the DCP index control limit of 70 mm/blow. While this may seem high, since the four-point running average of tests is used for the failure criteria, very few, if any, failures would occur. Additional distributions of DCP index data for varying soil types would be helpful for examining the probability of failures and potentially justify raising or lowering the control limits accordingly.

The distributions of variation in DCP index seem to reiterate the conclusions from earlier. None of the contractor tests exceed the control limit of 40 mm/blow, with only 17% even exceeding 20 mm/blow. One of the biggest problems that was identified with the uniformity parameter was that it is extremely rare and nearly impossible for a DCP test to pass the DCP index control limit for a single lift and fail the variation in DCP index control limit. This makes the test slightly redundant and seems to indicate that the control limit for variation in DCP index may be overly conservative as well. Additional research is necessary to develop this concept, and perhaps a new method of calculation is required.

The distributions for relative compaction and relative moisture content both appear to have a bimodal distribution. The highest peaks are at 103% relative compaction and 1.5% relative moisture content, while the smaller peaks are at 110% relative compaction and -1.5% relative moisture content. These peaks seem to suggest that the data contains two different soil types, supporting the prior assertion that fill properties may have changed at some point during construction.

Iowa State University also conducted independent classification and QA testing based out of the Iowa State University Geotechnical Mobile Lab throughout the project (Figure 8). Testing was conducted to monitor the Iowa DOT and contractor QC/QA testing. Figure 9 shows the results of testing conducted on August 17, 2006, between STA 143–145 of the project and comparisons to contractor QC test data. The fill material at the time of these tests was classified as the same unsuitable material from the above figures.

The DCP index and variation in DCP index control charts show that the Iowa State University and contractor testing seem to fall within similar ranges. The moisture and dry unit weight control charts show greater differences between Iowa State University and contractor data. Iowa State University moisture tests ranged from 19.8%–25.6 %, while contractor tests ranged from 19.2%–21.0%. The dry unit weight tests also ranged from 17.4–18.7 kN/m^3 and 16.4–17.2 kN/m^3 for Iowa State University and contractor tests, respectively. The Iowa State University classification tests resulted in a higher optimum dry unit weight and lower optimum moisture content in comparison to the values that were being used by the contractor at the time. Interestingly, even using the modified control limits, all of the contractor’s QC testing would have passed. While all of these discrepancies potentially represent some degree of bias in either the QC or QA testing, it is more likely that they are the result of natural variability within the sample, illustrating one of the fundamental challenges of all earthwork QC/QA programs.

Figure 8. Geotechnical Mobile Lab (left) and Iowa State University Q/A testing at the Crow Creek embankment on August 17, 2006 (right)

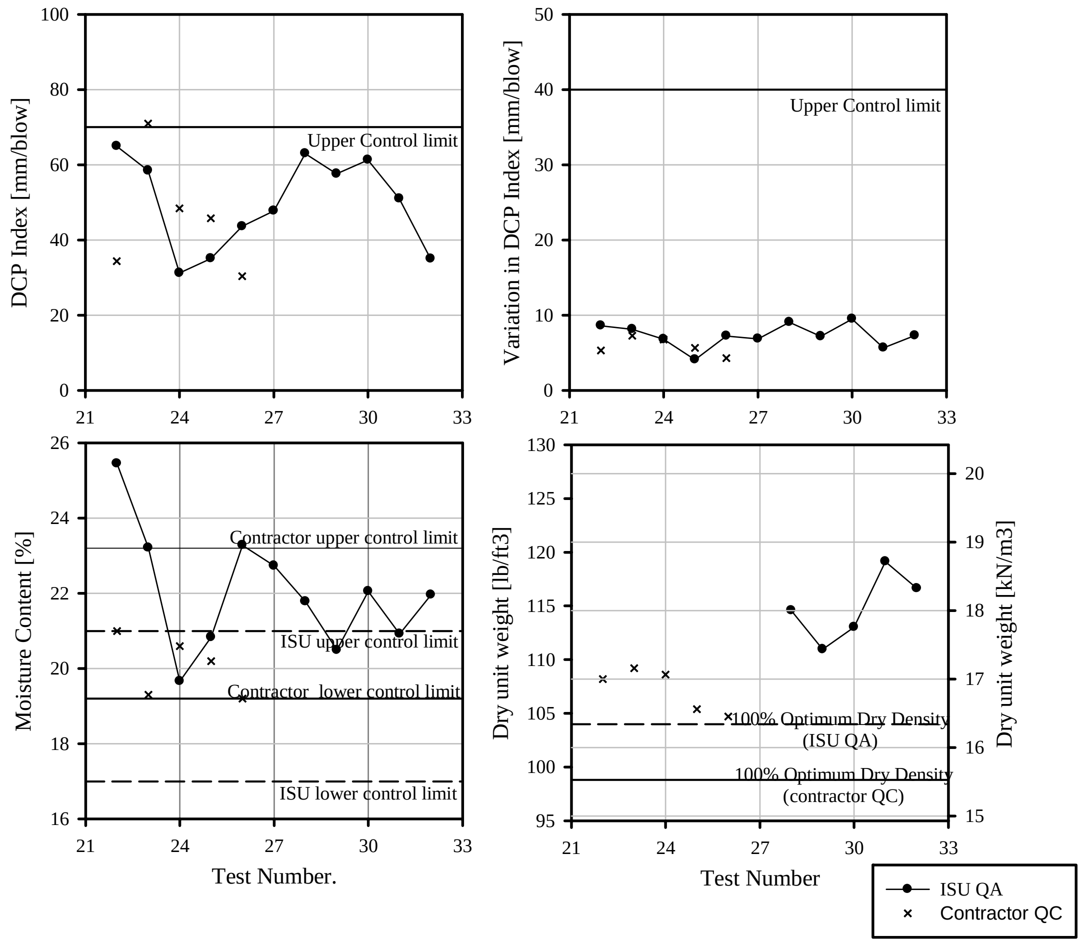

Figure 9. ISU QA testing data taken on August 17, 2006 between STA 143–145

CONCLUSIONS

The QM-E was successfully implemented for a pilot project that involved predominately unsuitable soil. Although the implementation was generally successful, a few areas of improvement were identified. First, the required DCP testing depth needs to be modified to be at least two lift thicknesses to improve the usefulness of the uniformity parameter, which was originally developed based upon testing over many lifts. The DCP index will still be reported for only the top lift; however, the variation in DCP index will be reported for the full depth of the test. While this change addresses some immediate concerns, additional research is required to further develop the understanding of the uniformity parameter, and a new calculation method may need to be developed. Secondly, a new provision must be added to require additional classification testing in the event that the four-point average of dry unit weight testing exceeds 105% optimum dry unit weight. This requirement will attempt to correct some of the issues that were observed at the pilot project with improper soil classification. Finally, though the DCP index control limit for unsuitable soil appears to be overly conservative, additional laboratory testing would be required to justify lowering the limits. It is the authors' opinion that an overly conservative DCP index is not necessarily a bad thing. The moisture control limits should be the controlling factor in embankment construction until research provides a better understanding of the relationship between strength-moisture-density. For now, the DCP testing should be supplemental to verify that the fill material is meeting a minimal strength requirement.

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Development, Testing, and Application of Precast Paving Notch

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ABSTRACT

Bridge owners are frequently faced by the need to replace critical bridge components during strictly limited or overnight road closure periods. This paper presents the development, testing, and application of a precast concrete bridge element specifically designed for the Iowa Department of Transportation to address this condition.

A paving notch (also known as a corbel or a paving support) consists of a horizontal shelf constructed on the rear of a bridge abutment and is used to support the adjacent roadway pavement. Over time, these paving notches have been observed to deteriorate due to a number of conditions, including improper reinforcing steel placement, backfill settlement, and an open expansion joint which tends to fill with dirt and debris and “push” the approach pavement off the paving notch over time.

A precast paving notch system was developed for use in either new construction or as rapid replacement. This system is designed to be installed in single-lane-widths to permit staged construction under traffic. The paving notch system consists of a rectangular, precast concrete element that is connected to the rear of the abutment using high-strength threaded steel rods and epoxy adhesive similar to that used in segmental bridge construction.

Researchers at Iowa State University have performed full-scale laboratory testing of the paving notch replacement system. Following these successful tests, a field application site has been selected for the implementation of this new system.

This paper presents the development of the precast paving notch system, the results of laboratory testing, and the installation and monitoring of the field application.

Key words: accelerated construction—bridge repair—paving support—post-tensioned pavement—post-tensioning—precast

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Experiences of Developing and Validating a New Mix Design Procedure for Cold In-Place Recycling Using Foamed Asphalt

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ABSTRACT

Asphalt pavement recycling has grown dramatically over the last few years as the preferred way to rehabilitate existing asphalt pavements. Rehabilitation of existing asphalt pavements has employed different techniques; one of them, cold in-place recycling with foamed asphalt (CIR-foam), has been effectively applied in Iowa. However, Iowa's current cold in-place recycling (CIR) practice utilizes a generic recipe specification to define the characteristics of the CIR mixture. The contractor is given latitude to adjust the proportions of stabilizing agent to achieve a specified level of density. As CIR continues to evolve, the desire to place CIR mixture with specific engineering properties requires the use of a mix design process. First, some strengths and weaknesses of the mix design parameters were identified, and the laboratory test procedure was modified to improve the consistency of the mix design process of CIR-foam. Both Marshall and indirect tensile strength test procedures were evaluated as foamed asphalt mix design procedures using reclaimed asphalt pavement (RAP) materials. Based upon the critical mixture parameters identified, a new mix design procedure using indirect tensile testing and vacuum-saturated wet specimens was developed. The second phase was then launched to validate the developed laboratory mix design process against various RAP materials in consideration of its predicted field performance. The optimum foamed asphalt contents, for all RAP materials, were consistently found at values between 1.5% and 2.5%. The dynamic modulus values were affected by both foamed asphalt contents and RAP aggregate structure. The flow number is affected dominantly by the RAP aggregate structure.

Key words: cold in-place recycling—dynamic modulus—flow number—foamed asphalt—indirect tensile strength—mix designs procedure—reclaimed asphalt pavement

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INTRODUCTION

A desire to maintain a safe, efficient, and cost-effective roadway system has led to a significant increase in the demand to rehabilitate the existing pavement. Asphalt recycling has grown dramatically over the last few years as the preferred way to rehabilitate existing pavements. Rehabilitation of existing asphalt pavements has employed different techniques, one of them being cold in-place recycling (CIR).

Recently in Iowa, cold in-place recycling with foamed asphalt (CIR-foam) has become more common for rehabilitating existing asphalt pavements due to its cost-effectiveness, the conservation of paving materials, and its environmental friendliness. Iowa's current CIR practice utilizes a generic recipe specification to define the characteristics of the CIR mixture. The contractor is given latitude to adjust the proportions of stabilizing agent to achieve a specified level of density. As CIR continues to evolve, the desire to place CIR mixture with specific engineering properties requires the use of a mix design process. However, there is no design procedure available for CIR-foam.

A new mix design procedure for CIR-foam was developed and validated. First, some strengths and weaknesses of the mix design parameters were identified, and the laboratory test procedure was modified to improve the consistency of the mix design process of CIR-foam. The phase two study was conducted to validate the developed laboratory mix design process against various RAP materials across the state of the Iowa. As part of the validation effort to evaluate the consistency of the new CIR-foam mix design process, simple performance tests, which included the dynamic modulus test, dynamic creep test, and raveling test, were conducted over a wide range of traffic and climatic conditions.

DEVELOPMENT OF CIR-FOAM MIX DESIGN PROCESS

Various foamed asphalt mix design parameters produced from numerous past studies for full-depth reclamation (FDR) and CIR were reviewed, and detailed laboratory test results are documented in Lee and Kim (2003). To conduct laboratory experiments for the CIR-foam mix design process, RAP materials were collected from the CIR-foam project site on US-20, which is located at about four miles west of the intersection of US-20 and Highway 13 near the city of Manchester. The existing asphalt pavement was milled throughout the day, and, to identify the possible variation in RAP gradations, the temperatures of the milled RAP materials were measured throughout the day. Based on the limited study of RAP materials, the time of the milling and the temperature of the pavement during the milling process did significantly affect the RAP gradation. To identify the impact of the RAP gradation on the CIR-foam mix design, three different RAP gradations were designed as "fine," "field," and "coarse."

The PG 52-34 asphalt binder was used as the stabilizing agent for the laboratory foamed asphalt mix design. Using the laboratory foaming equipment from Wirtgen, shown in Figure 1, the foaming water content of 1.3% created the optimum foaming characteristics in terms of an expansion ratio of 10-12.5 and a half-life of 12-15 at 170°C. As illustrated in Figure 2, the flowchart of the laboratory mix design process of CIR-foam was developed to identify the critical mix design parameters, which included laboratory test procedures, RAP gradation, and the optimum moisture content of RAP.

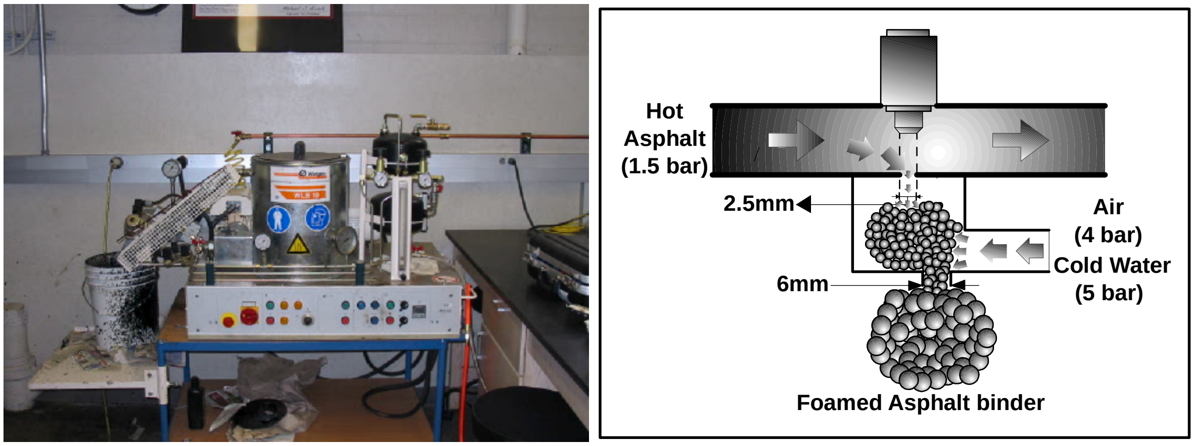


Figure 1. Laboratory foaming equipment (left) and production of foamed asphalt (right)

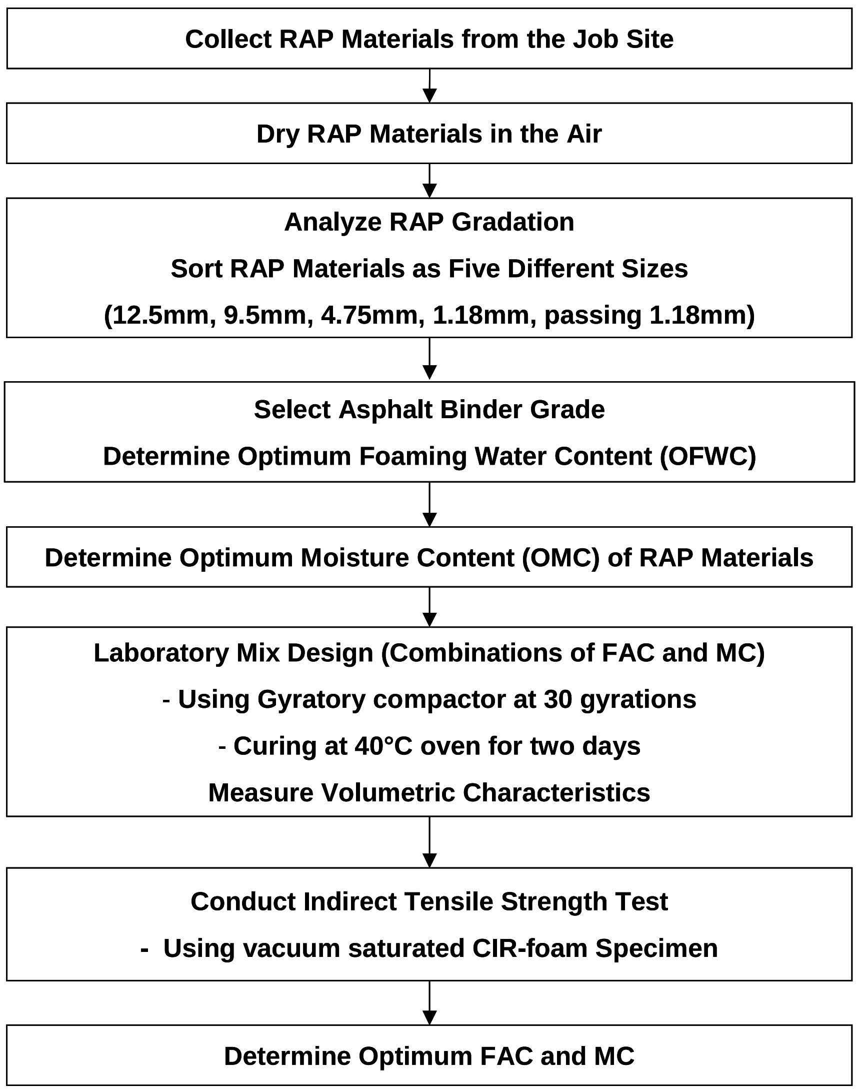


Figure 2. Laboratory foamed asphalt mix design flowchart

The laboratory mix design process for CIR-foam is described below to understand how to perform the CIR-foam mix design process in the laboratory:

- **Step 1**. RAP materials collected from the field should be dried in the air until the moisture content exhibits between 0.3% and 0.1%.
- **Step 2**. RAP materials collected from the field should be analyzed to determine gradation. First, for gradation analysis, RAP materials larger than 25 mm were discarded. To produce the laboratory CIR-foam mixtures of the field gradation, the remaining RAP materials were then divided into four or five stockpiles retained on each of the following sieves, 19.0 mm, 9.5 mm, 4.75 mm, 1.18 mm, as well as the materials that passed the 1.18 mm sieve.
- **Step 3**. Optimum foaming water content should be determined by achieving the maximum expansion ratio and half-life for a given asphalt binder. Expansion ratio is defined as the maximum volume over its original volume, and half-life is defined as the time in seconds for foam to become a half of its maximum volume.
- **Step 4**. The optimum moisture content during mixing and compaction is considered one of the most important mix design criteria for CIR-foam mixtures. The modified proctor test should be conducted in accordance with ASTM D 1557, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (2,700 kN-m/m^3)."
- **Step 5**. Thirty gyrations of the Superpave gyratory compactor should be used to produce CIR-foam test specimens with a diameter of 100 mm. The compacted foamed asphalt specimens should be cured in the oven for two days at 60 °C (Figure 3).

Figure 3. Gyratory compaction and curing process

- **Step 6**. The cured CIR-foam specimen should be placed under water at 25 ºC for 30 minutes and, to achieve the 100% saturation level, a vacuum of 20 mmHg should be applied for 30 minutes. The saturated sample is then placed under water for additional 30 minutes (Figure 4).

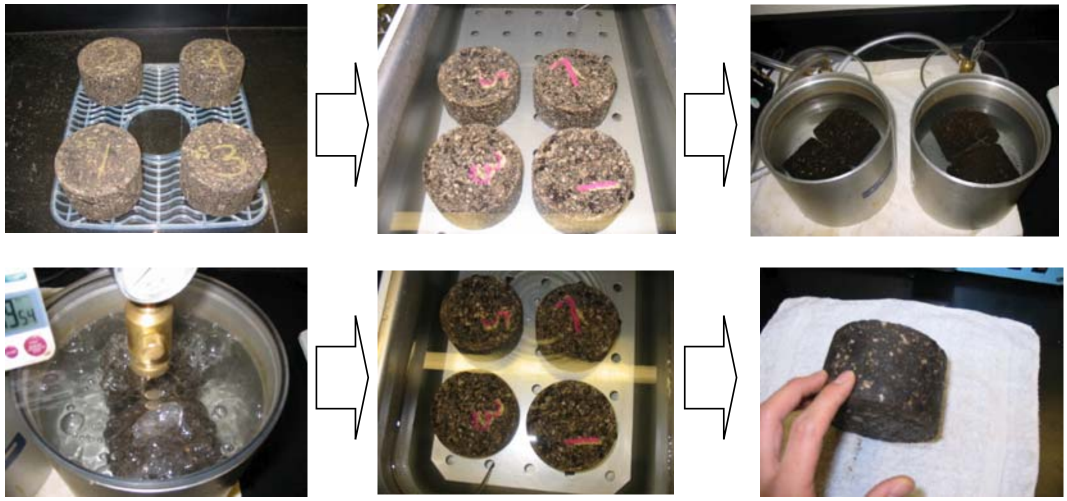


Figure 4. Vacuum saturation process

- **Step 7**. The indirect tensile strength test should be performed on three saturated CIR-foam mixtures for each of five foamed asphalt contents ranging from 1.0% to 3.0% at 0.5 increments (Figure 5).

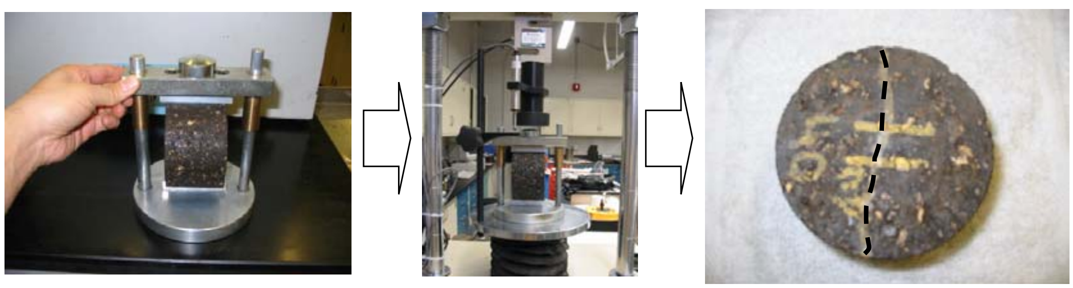


Figure 5. Indirect tensile strength test

- **Step 8**. The optimum foamed asphalt content (FAC) and optimum moisture content should be determined where the maximum indirect tensile strength is obtained.

COLLECTION AND EVALUATION OF RAP MATERIALS

In order to validate the mix design process developed during the first research phase, RAP materials were collected from seven different CIR project sites: three CIR-foam and four CIR-ReFlex sites. Figure 6 (left) shows the selected the CIR project sites across the state of the Iowa: Muscatine County, Webster County, Hardin County, Montgomery County, Bremer County, Lee County, and Wapello County.

First, RAP materials were divided into six stockpiles that were retained on the following sieves, 25 mm, 19 mm, 9.5 mm, 4.75 mm, 1.18 mm, as well as those passing through the 1.18 mm sieve. The sorted RAP materials were then weighed, and their relative proportions were computed. As shown in Figure 6 (right), gradation analyses for seven RAP sources were conducted, and the RAP materials from Muscatine County were the coarsest, followed by Montgomery, Webster, and Wapello Counties. Those from Hardin, Bremer, and Lee Counties were finer. All RAP materials were considered from fine to coarse with a very small amount of fine aggregates passing through the 0.075 mm (No. 200) sieve.

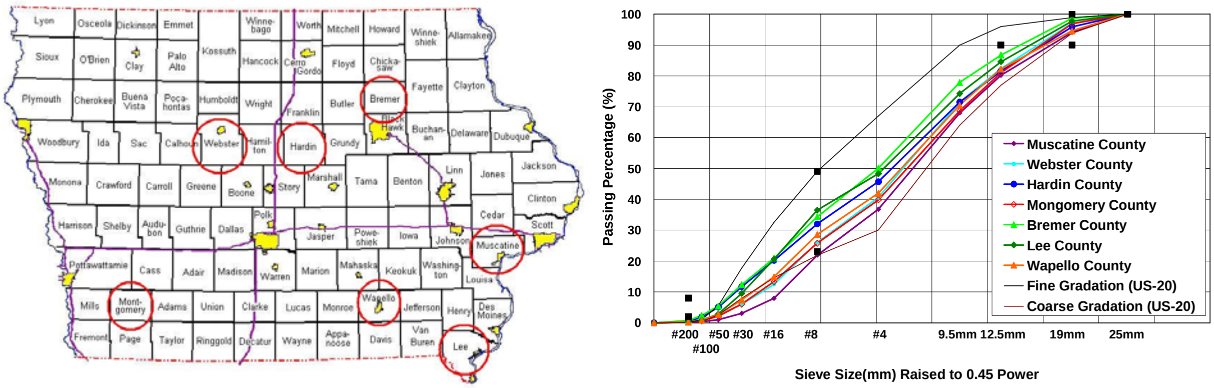


Figure 6. Location of selected CIR job sites (left) and gradation plots (right)

As summarized in Table 1, the characteristics of the RAP materials from seven different RAP sources were analyzed in terms of (1) residual asphalt content, (2) residual asphalt stiffness in penetration index, and (3) amount of fines passing No.8 sieve. The extracted asphalt content ranged from 4.6% for RAP materials collected from Wapello County to 6.1% from Hardin County. The extracted asphalt of RAP material from Montgomery County exhibited the highest penetration of 28, whereas that of Hardin and Lee Counties showed the lowest penetration of 15.

Table 1. Characteristics of seven different RAP materials

	RAP Characteristics		
RAP Source	**Residual AC (%)**	**Stiffness (Pen.)**	**% Passing No.8 Sieve**
Lee County	Middle (5.4%)	Hard (15)	Fine (36.5%)
Webster County	High (6.0%)	Hard (17)	Middle (28.6%)
Hardin County	High (6.1%)	Hard (15)	Fine (32.0%)
Wapello County	Low (4.6%)	Soft (21)	Coarse (26.0%)
Bremer County	Middle (5.0%)	Hard (17)	Fine (34.4%)
Montgomery County	High (5.7%)	Soft (28)	Coarse (25.8%)
Muscatine County	Low (4.7%)	Middle (19)	Coarse (21.9%)

VALIDATION OF A NEW MIX DESIGN PROCESS

To determine the consistency of a new CIR-foam mix design procedure, the newly developed CIR-foam mix design procedure was validated against seven different sources of RAP materials in Iowa. As shown in Figure 7, the indirect tensile strength test of the vacuum-saturated specimens was conducted using seven different RAP materials at five foamed asphalt contents, 1.0%, 1.5%, 2.0%, 2.5%, and 3.0%, given a fixed moisture content of 4.0%. The specimens were compacted by gyratory compactor at 30 gyrations or by a Marshall hammer at 75 blows and were cured at 40°C oven for three days or 60°C for two days (Lee and Kim 2007).

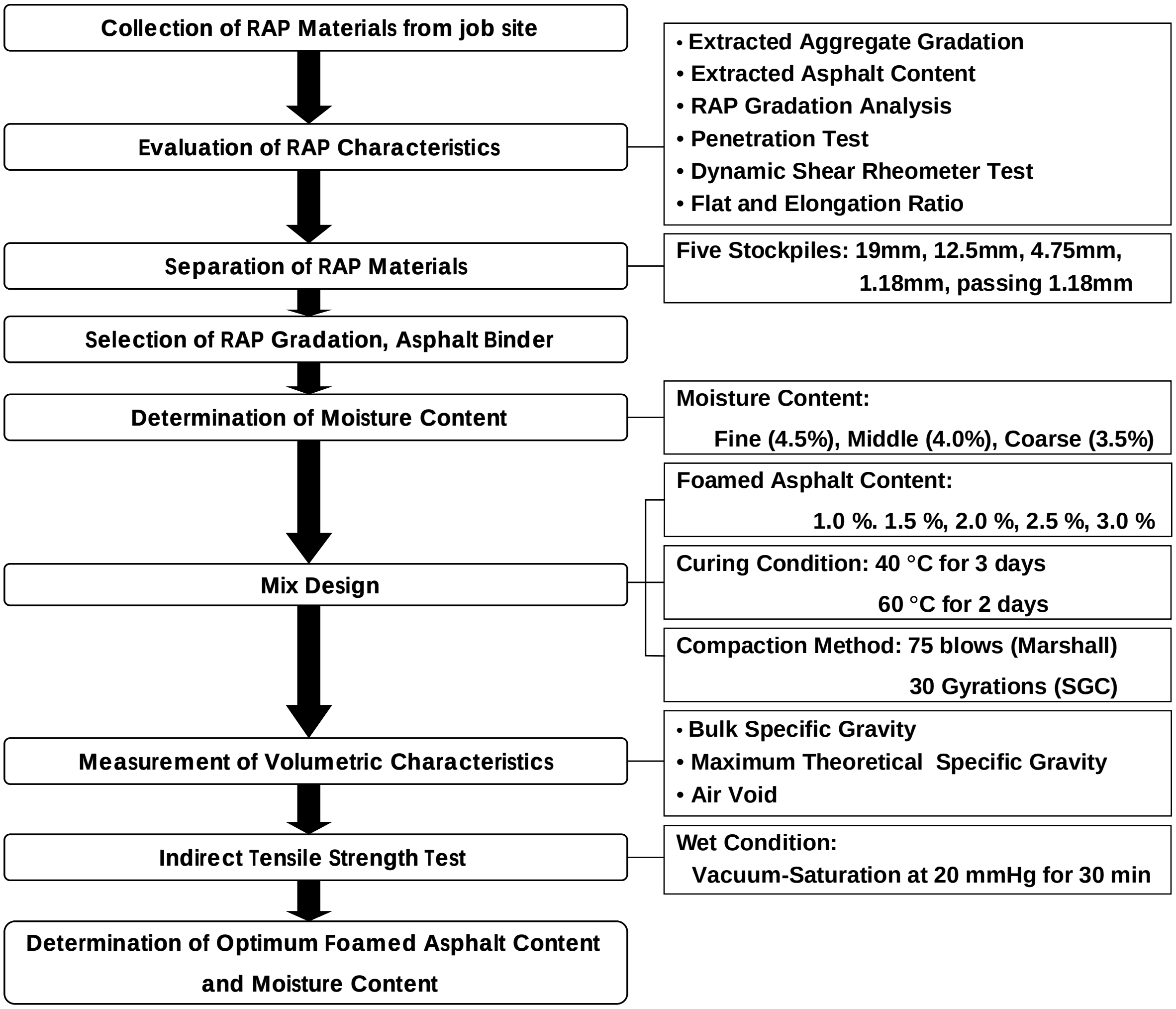


Figure 7. Laboratory mix design procedure used for validation study

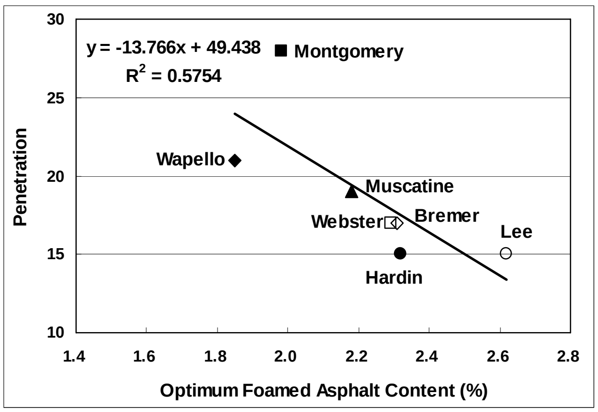
The indirect tensile strength of the gyratory-compacted and vacuum-saturated specimens was more sensitive to foamed asphalt contents than that of Marshall hammer-compacted and vacuum-saturated specimens. The indirect tensile strength of CIR-foam specimens cured for two days in the 60°C oven was significantly higher than that of CIR-foam specimens cured for three days in the 40°C oven.

The optimum foamed asphalt content (OFAC) was determined when the highest indirect tensile strength of vacuum-saturated specimens was obtained. The indirect tensile strength of the gyratory-compacted specimens is higher than that of Marshall hammer-compacted specimens. The indirect tensile strength of foamed asphalt specimens cured at 60°C for two days is significantly higher than that of foamed asphalt specimens cured at 40°C for three days.

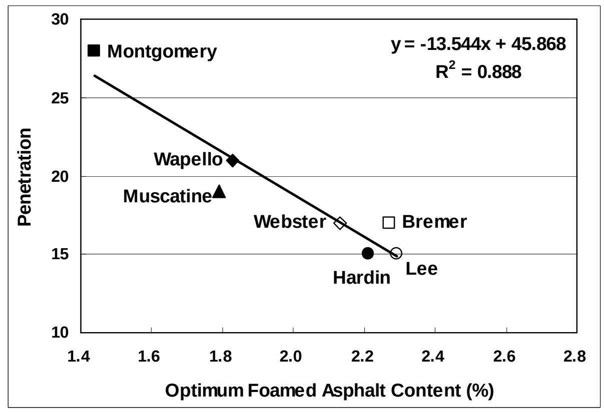
Due to its high moisture sensitivity, it is recommended that the indirect tensile strength test be performed on the vacuum-saturated CIR-foam mixtures. Further, the indirect tensile strength test is recommended for the CIR-foam mix design because it is a relatively simple procedure using the standard equipment available in a typical asphalt laboratory. As a result, the Iowa Department of Transportation and most contractors in Iowa can now easily perform the proposed CIR-foam mix design procedure.

Correlation between OFAC and RAP Characteristics

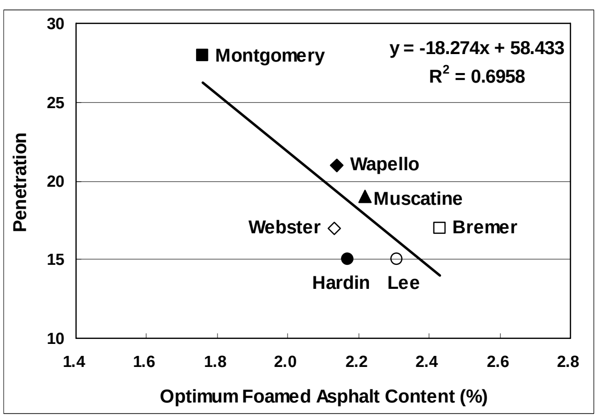
Attempts were made to discover a correlation between FAC and RAP characteristics, such as residual asphalt stiffness and residual asphalt content. The OFAC was determined based on a polynomial regression equation. A higher OFAC value was obtained from the RAP materials containing large amounts of hard residual asphalt. Figure 8 shows the correlation plots of residual asphalt stiffness values measured as a penetration index against OFAC at four different mix design conditions. As shown in Figure 8, a good correlation between OFAC and residual asphalt stiffness is exhibited, but no correlation between OFAC and residual asphalt contents was observed. The stiffer residual asphalt required more foamed asphalt, whereas the higher residual asphalt content did not require less foamed asphalt.



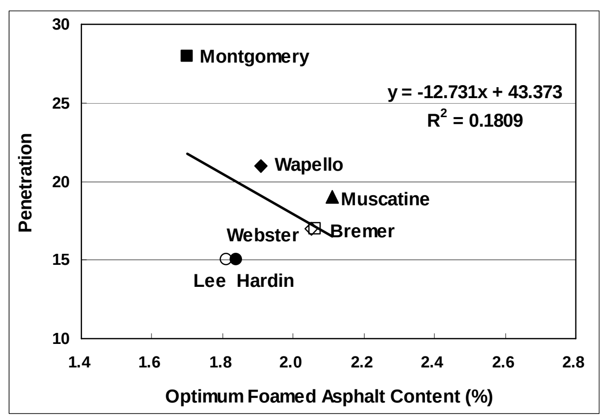
(a) Marshall compaction (40˚C)



(b) Marshall compaction (60˚C)



(c) Gyratory compaction (40˚C)



(d) Gyratory compaction (60˚C)

Figure 8. Correlations between optimum foamed asphalt content and residual asphalt stiffness

PERFORMANCE TEST RESULTS

The performance tests, which included the dynamic modulus test, dynamic creep test, and raveling test, were conducted to evaluate the consistency of a new CIR-foam mix design process to ensure reliable mixture performance over a wide range of traffic and climatic conditions.

Dynamic Modulus Test

For dynamic modulus and dynamic creeps tests, CIR-foam specimens with a 100 mm diameter and a 150 mm height were prepared at three foamed asphalt contents, 1.0%, 2.0%, and 3.0%, and at a 4.0% moisture content. CIR-foam specimens were compacted using the gyratory compactor at 30 gyrations, and the compacted CIR-foam specimens were cured in the oven at 40°C for three days.

The dynamic modulus test is used to determine the stiffness of asphalt mixtures on the response to traffic loading and various climate conditions. The dynamic modulus tests were performed on CIR-foam mixtures at six different loading frequencies, 0.1, 0.5, 1, 5, 10, and 25 Hz, and three different test temperatures, 4.4°C, 21.1°C, and 37.8°C, using simple performance test equipment, as shown in Figure 9.

Figure 9. Simple performance test equipment

Within each source of RAP materials, the dynamic moduli of RAP materials were not affected by loading frequencies but were significantly affected by the test temperatures. The rankings of RAP materials changed when the foamed asphalt was increased from 1.0% to 3.0%, which indicates that the dynamic modulus values are affected by both FAC and RAP aggregate structure. Based on the dynamic modulus test results performed at 4.4˚C, the coarser RAP materials with a small amount of residual asphalt content were more resistant to fatigue cracking. Based on the dynamic modulus test results performed at 37.8˚C, the finer RAP materials with the harder binder and with a higher amount were more resistant to rutting.

Dynamic Creep Test

With increasing truck traffic and tire pressure, rutting is one of the most critical types of load-associated distresses occurring in asphalt pavements. Therefore, it is important to characterize the permanent deformation behavior of asphalt mixtures in order to identify problematic mixes before they are placed in roadways. Dynamic creep tests were performed on CIR-foam mixtures under a loading stress level of 138 kPa at 40˚C using simple performance test equipment.

Based on the dynamic creep test, RAP materials from Muscatine County exhibited the lowest flow number of all foamed asphalt contents, whereas those from Lee and Webster Counties reached the highest flow numbers. The lower the foamed asphalt contents, the higher the flow number, which indicates that the foamed asphalt content with 1.0% is more resistant to rutting than 2.0% and 3.0%.

Characteristics of seven RAP materials are summarized in Table 2, along with the rankings in terms of flow number. RAP materials from seven different sources were ranked by flow number. Overall, the rankings of RAP materials did not change when the foamed asphalt was increased from 1.0% to 3.0%, which indicates that flow number is affected more dominantly by the RAP aggregate structure than by the foamed asphalt content. The finer RAP materials with a higher amount of the harder binder were more resistant to rutting. This result is consistent with the findings based on the dynamic modulus testing performed at 37.8˚C.

Table 2. Rankings of flow number from seven different RAP sources

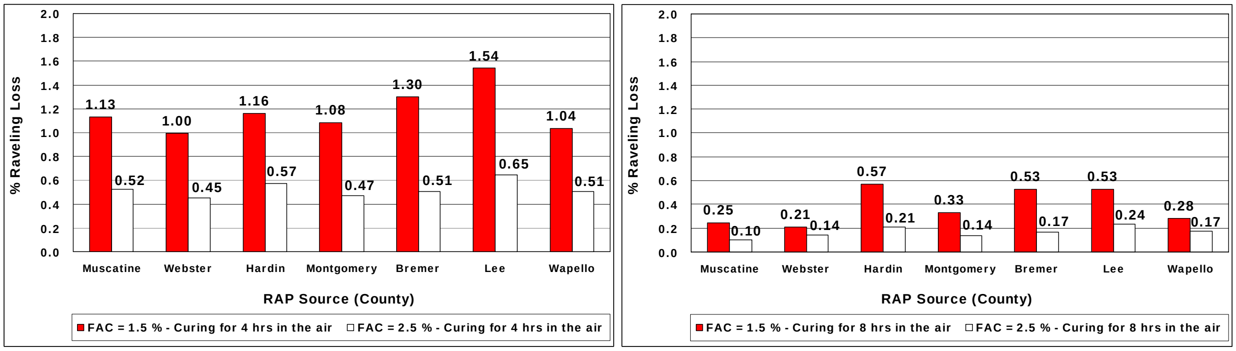
RAP Source	Stiffness (Pen.)	Residual AC (%)	% Passing No.8 Sieve	Ranking of Flow Number		
				FAC=1.0%	FAC=2.0%	FAC=3.0%
Lee County	15	5.4%	36.5%	1	1	1
Webster County	17	6.0%	28.6%	2	2	2
Hardin County	15	6.1%	32.0%	3	3	3
Wapello County	21	4.6%	26.0%	4	4	6
Bremer County	17	5.0%	34.4%	5	5	5
Montgomery County	28	5.7%	25.8%	6	6	4
Muscatine County	19	4.7%	21.9%	7	7	7

Based on both the dynamic modulus and dynamic creep test results, it can be postulated that RAP materials from Wapello and Webster Counties would be more resistant to both fatigue and rutting. RAP materials from Muscatine, Bremer, and Montgomery Counties would be more resistant to fatigue cracking but less resistant to rutting. RAP materials from Hardin and Lee Counties would be more resistant to rutting but less resistant to fatigue cracking.

Raveling Test

A CIR-foam layer is normally covered by a hot mix asphalt (HMA) overlay or chip seal in order to protect it from water ingress and traffic abrasion and to obtain the required pavement structure and texture. During curing in the field, some raveling occurred from the surface of the CIR pavement before HMA overlay was placed. The raveling test was performed at room temperature using a gyratory-compacted 150 mm CIR-foam specimen to evaluate resistance to raveling right after construction.

The percent mass loss of the foamed asphalt specimens at 1.5% FAC and 2.5% FAC for two different cuing time periods is plotted in Figure 10. Based on the raveling test results, the foamed asphalt specimens at 2.5% foamed asphalt content showed less raveling loss than those of 1.5% foamed asphalt content. It was found that the raveling test was very sensitive to the curing period and foamed asphalt content of the CIR-foam specimens. To increase cohesive strength quickly, it is necessary to use the higher foamed asphalt content of 2.5% instead of 1.5%.



(a) Curing time: four hours **(b) Curing time: eight hours**

Figure 10. Percent raveling losses of CIR-foam specimens from seven different RAP sources

CONCLUSIONS AND RECOMMENDATIONS

Asphalt pavement recycling has grown dramatically over the last few years as a viable technology to rehabilitate existing asphalt pavements. Rehabilitation of existing asphalt pavements has employed different techniques; one of them, CIR-foam, has been effectively applied in Iowa. This research developed and validated the mix design procedure for CIR-foam. It was also demonstrated that the field performance of various CIR-foam mixtures could be predicted based on the test results from newly purchased performance testing equipment. Based on the extensive laboratory experiments on CIR-foam, the following conclusions, recommendations, and suggestions for future studies are made:

Conclusions

- The indirect tensile strength of gyratory-compacted specimens is higher than that of Marshall hammer-compacted specimens. The indirect tensile strength of CIR-foam specimens cured in the oven at 60˚C for two days is significantly higher than that of CIR-foam specimens cured in the oven at 40˚C for three days.
- The OFAC is influenced by residual asphalt stiffness more than the residual asphalt content. The stiffer residual asphalt would require more foamed asphalt
- Dynamic modulus of CIR-foam is affected by a combination of the RAP sources and foamed asphalt contents. Coarse RAP materials with a small amount of residual asphalt content may be more resistant to fatigue cracking but less resistant to rutting.
- Based on the dynamic creep tests performed at 40˚C, CIR-foam with 1.0% foamed asphalt is more resistant to rutting than CIR-foam with 2.0% or 3.0% foamed asphalt. RAP aggregate structure has a predominant impact on its resistance to rutting. The finer RAP materials with the more and harder residual asphalt were more resistant to rutting.
- CIR-foam specimens with 2.5% foamed asphalt content are more resistant to raveling than ones with 1.5%.

Recommendations

- To determine the OFAC, indirect tensile strength testing should be performed on vacuum-saturated specimens, which should be placed in 25˚C water for 20 minutes, vacuumed saturated at 20 mmHg for 30 minutes, and left under water for an additional 30 minutes without vacuuming.
- The OFAC should be increased from 1.5% to 2.5% if the penetration index of the residual asphalt from RAP materials increases from 28 to 15.
- The proposed mix design procedure should be implemented to assure the optimum performance of CIR-foam pavements in the field.

Suggestions for Future Studies

- CIR-foam pavements should be constructed following the new mix design process, and their long-term field performance should be monitored and verified against the laboratory performance test results.
- New mix design and laboratory simple performance tests should be performed on the CIR-foam mixtures using stiffer asphalt binder grade, i.e., PG 58-28 or 64-22.
- The CIR-foam mix design should be adapted for CIR-emulsion mixtures.
- A comprehensive database of mix design, dynamic modulus, flow number, and raveling for both CIR-foam and CIR-emulsion should be developed to allow for an input into the Mechanistic-Empirical Pavement Design Guide.

ACKNOWLEDGMENTS

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Development of a Low-Cost, Continuous Structural Health Monitoring System for Bridges and Components

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ABSTRACT

The Iowa State University Bridge Engineering Center developed a low-cost, continuous structural health monitoring system that can be used to monitor typical girder bridges. The developed structural health monitoring system can be grouped into two main categories: an office component and a field component. The office component is a structural analysis software program that can be used to generate thresholds which are used for identifying isolated events. The field component includes hardware and field monitoring software which performs data processing and evaluation. The hardware system consists of sensors, data acquisition equipment, and a communication system backbone. The field monitoring software has been developed such that, once started, it will operate autonomously with minimal user interaction. In general, the structural health monitoring system features two key uses. First, the system can be integrated into an active bridge management system that tracks usage and structural changes. Second, the system helps owners to identify overload occurrence, damage, and deterioration.

Key words: bridge—evaluation—low-cost—overload—structural health monitoring

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INTRODUCTION

The ability to monitor the condition of a bridge to ensure its safe usage and to be able to effectively manage its operation is of significant interest to bridge owners. Over the decades, the most widely used condition monitoring methods rely on subjective, incremental visual assessments or localized testing techniques. However, these techniques often require traffic control to be implemented and may not be sensitive enough to identify damage and/or deterioration over time. In order to address this issue, the Iowa State University (ISU) Bridge Engineering Center developed an autonomous, continuous structural health monitoring (SHM) system that can be used to monitor typical girder bridges. The developed system features two key uses. First, the system can be integrated into an active bridge management system to track usage and structural changes. Second, the system helps owners to identify overload occurrence, vehicle collision to the structure, damage, and deterioration.

Numerous tools and technologies (currently available as well as emerging) associated with SHM applications have been well publicized (Aktan et al. 2003; Phares et al. 2003). The main issue now facing the bridge engineering community is not the lack of technologies that are available for SHM application, but rather how to accurately analyze a target bridge or its members and how to process continuously collected data such that the useful information can be extracted and used. It is also important that a SHM system be capable of monitoring long-term phenomena as well as capturing short-term events. In addition, the output of a SHM system must provide clear, usable benefits to bridge owners rather than inundating them with massive amounts of disjointed data. Such a need requires the development of a comprehensive approach to data management that also includes the development of high-performance localized data processing and evaluation algorithms. Significant effort has been given in the work summarized here to include data processing and evaluation algorithms that are based upon strong engineering principles while also taking full advantage of advanced data processing techniques.

OBJECTIVES

The primary objective of this research was to develop a low-cost, continuous SHM system that can be used to monitor typical girder bridges for detecting and identifying overload occurrence, vehicle collision to the structure, changes in structural behavior, identification of damage and deterioration, and for tracking usage. These specific needs were established to give owners tools to better manage bridge assets and were accomplished by completing three distinct work tasks as follows:

- Development of live load structural analysis software
- Development of field data collection and analysis software that integrates with select data acquisition hardware
- Demonstration of the developed SHM system

The product of this work is a turnkey SHM system that consists of hardware and software components. The hardware consists of off-the-shelf components that have been integrated to work together. Two software packages were also developed that allow for effective system use. First, a structural analysis package was developed that allows for bridge specific system configuration. Second, data collection/analysis/reporting package was developed that operates without user intervention to monitor for the above mentioned reasons.

STRUCTURAL HEALTH MONITORING SYSTEM

An overall schematic for the SHM system is illustrated in Figure 1. The SHM system can be principally grouped into two main components: an office component and a field component. The office component is basically a structural analysis software package that can be used to generate bridge-specific thresholds. The field component includes hardware and monitoring software which performs the data collection, processing, and evaluation. The hardware system consists of sensors, data acquisition equipment, and an optional communication system backbone. The field monitoring software was developed such that, once started, it will operate autonomously with minimal user interaction.

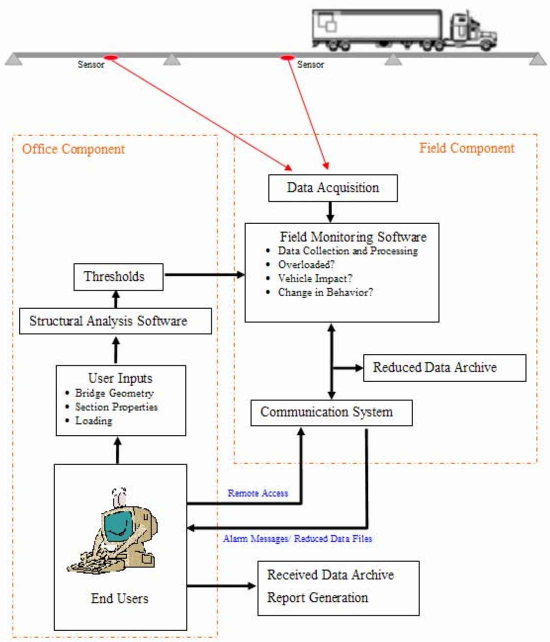


Figure 1. Overall schematic of SHM system

Structural Analysis Software

A Windows-based, two-dimensional, live load structural analysis program, BEC Analysis (see Figure 2), was created, using the Microsoft Visual Basic 6.0 programming language to simplify determination of some of the bridge specific SHM system parameters. BEC Analysis is capable of analyzing a bridge beam or girder with various boundary conditions and member geometries under various moving load conditions. One unique feature of BEC Analysis is that it allows users to easily determine maximum results (e.g., maximum moment and strain) at any location along the length of a bridge. In addition, it contains many convenient features which allow relatively quick analysis of a bridge. In general, one may use BEC Analysis for (1) analyzing beams or girders under moving loads, (2) computing absolute maximums in each span or at a desired location, and (3) generating envelopes of maximum moments and strains. The following summarizes some of the features that are included in BEC Analysis:

- Text fields and click-to-select options used to define bridge parameters
- Input defaults that will help novice users
- Library of various member cross section properties
- Calculator that computes section properties of virtually any member cross section
- Capable of analyzing up to ten spans

- Capable of modeling non-prismatic members
- Run multiple analyses without exiting the program
- Supports the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specification for Highway Bridges*, Sixteenth Edition (AASHTO 1996)
- Supports various loading conditions
- Graphic diagrams
- Print/save results

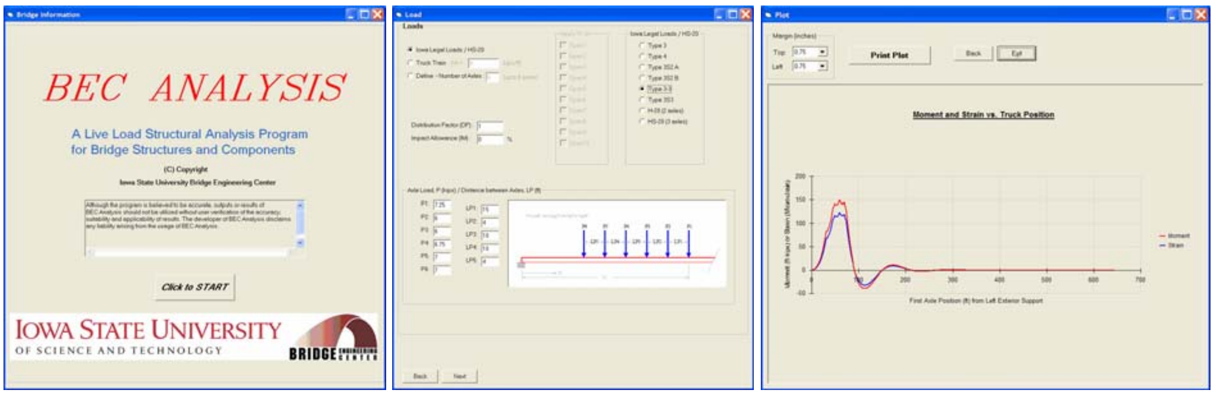


Figure 2. Example screen shots of BEC Analysis

BEC Analysis was designed to be used specifically for analyzing two-dimensional girder bridges subjected to moving loads. The commonly used two-dimensional stiffness matrix method was used as the computational backbone in BEC Analysis (Lee 2007). As a result, the program can determine the absolute maximum positive and negative moments and strains either in each span or at a designated location. In addition, envelopes can be generated. The envelopes contain the extreme values, both positive and negative, of moments and strains along the length of a model bridge. The results of the absolute maximums can be printed at the users' discretion. Moreover, users can review the analysis results graphically or save them for later review.

BEC Analysis consists of three modules: pre-processor, analysis, and post-processor. Each module was, respectively, developed to perform a certain task such as model generation, analysis, and result viewing. These three modules can be further categorized into six sub-groups: (1) Bridge Information windows (2) Span Description windows (3) Load window (4) Run Analysis window (5) Print/View/Plot windows. The pre-processor groups (1, 2 and 3) are used for data input, modeling, and on-screen graphic display. The analysis module (4) performs the analysis. The last module, postprocessor (5), was designed for reviewing the analysis results.

Field Monitoring Software

The field monitoring software (see Figure 3) was developed, in LabView 7.1, to function with IOtech instrumentation hardware (Lee 2007). The software was designed to collect, process, and evaluate the measured response of a bridge. Its use will allow bridge owners to quantitatively monitor a bridge for identifying overload occurrence, vehicle collision to the structure, damage, as well as gradual changes in behavior.

The field monitoring software consists of three groups of programs: (1) a preliminary data acquisition and analysis component intended for identifying basic characteristics, (2) a main data acquisition and processing component intended for data collection, reduction and evaluation processes, and (3) a report generation component intended for presenting results to the user. Each group of programs was designed to be accessed at any time. The preliminary data acquisition and analysis is a task that assists in reducing noise and detecting events so that only the pertinent strain information is obtained. It involves establishing the parameters that will be used during the data processing and evaluation processes that occur in other programs. The second group of programs controls the main data acquisition and the organization of the collected data and passes it to the processing components. During this process, collected data will be temporarily stored into designed segments and then internally passed through a series of data reduction programs in such a way as to allow the acquisition program to operate in real time while the processing programs operate in the background. These collected data are evaluated, reduced, written to a data file, and archived all within the local host PC. The results from the second group are a series of data files generated on a timely basis, each of which contains summarized information about the bridge performance. The third group of the field monitoring software is used for immediate viewing of summarized information and for generating reports.

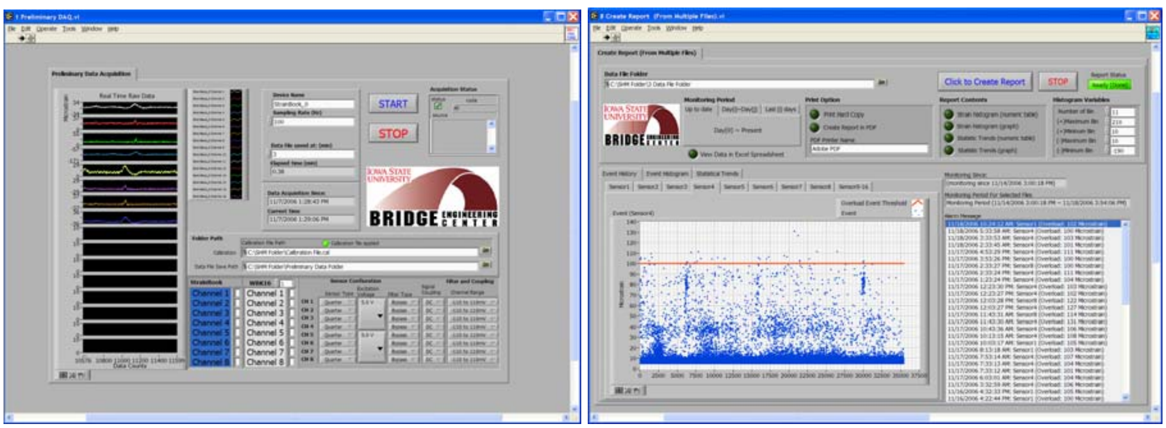


Figure 3. Example screen shots of field monitoring software

Monitoring Concept

An alarm event is determined by examining the peaks in a strain record. Alarm events can be generally thought of as either those caused by overloaded traffic, referred to as “overload,” and an abnormal rapid change in strain, referred to as “impact.” Some of the important terms that are used as the building blocks in the field monitoring software are defined as follows:

- Event: any peak in a strain record that exceeds a defined event detection threshold
- Alarm Event: overload event and/or impact event
- Overload event: event that exceeds the overload event threshold
- Impact event: event that that exceeds the impact event threshold.

In general, two steps are involved in the processing of the collected data: identification of events and examination of each event to see if it exceeds the predefined thresholds (Lee 2007). First, any peak in a measured strain record that exceeds the event detection threshold will be identified as an event. Once the event is detected, the software examines its magnitude in strain and the slope of the strain record that contains the event. If the event exceeds the overload event threshold, which can be determined using BEC Analysis or any other means, it will be recorded as an overload event. The impact event is identified by examining the slope associated with the event. If the slope exceeds the impact event threshold (possibly due to a vehicle collision to the structure), it will be recorded as an impact event. Identification of the impact event involves examining three parameters: the start index, the peak index, and the event. As illustrated in Figure 4, each parameter is expressed with b_i and y_i components, where b_i represents the time that the index or the event is recorded, while y_i represents their magnitude in strain. These three parameters are used to find the slope within the strain record that contains the event. The slope of an event can be defined as the ratio of the absolute difference between the magnitude of an event and that of an event detection threshold with respect to the time difference between a start index and a peak index. The slope can be determined as follows:

$$Slope = \frac{y_2 - y_1}{b_2 - b_1} \qquad (1)$$

Once the slope is determined, the software checks to see if the slope exceeds the predefined impact threshold. If exceeded, the software will recognize the event as an impact event. Note that the impact event threshold must be defined prior to running the field monitoring software (Lee 2007). This may require collecting sample strain data from ambient traffic to establish an appropriate strain rate.

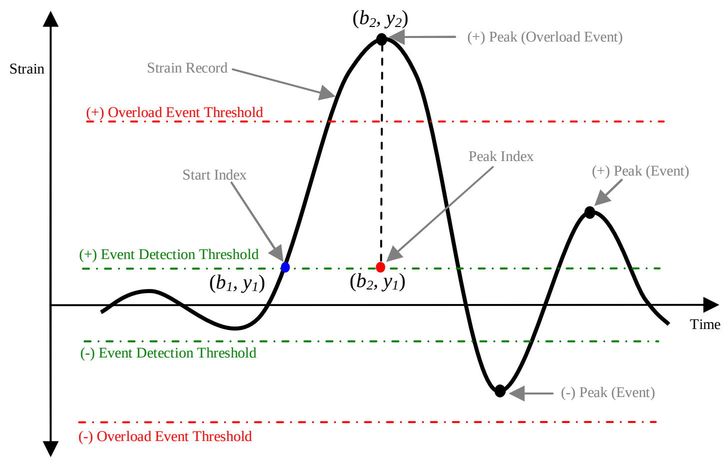


Figure 4. Parameters used to determine a slope in a strain record

The approach for compensating the temperature variations used here is based upon the idea that thermal expansion and contraction are very slow in comparison to changes associated with live loads. Therefore, one may assume that the change in strain due to temperature variations within a short period of time is insignificant. With this consideration in mind, it was decided that the strains be processed in small segments so that the temperature effects on each set of measured strains are minimal. To this end, the field monitoring software was developed to process measured strain segments every ten minutes (Lee 2007).

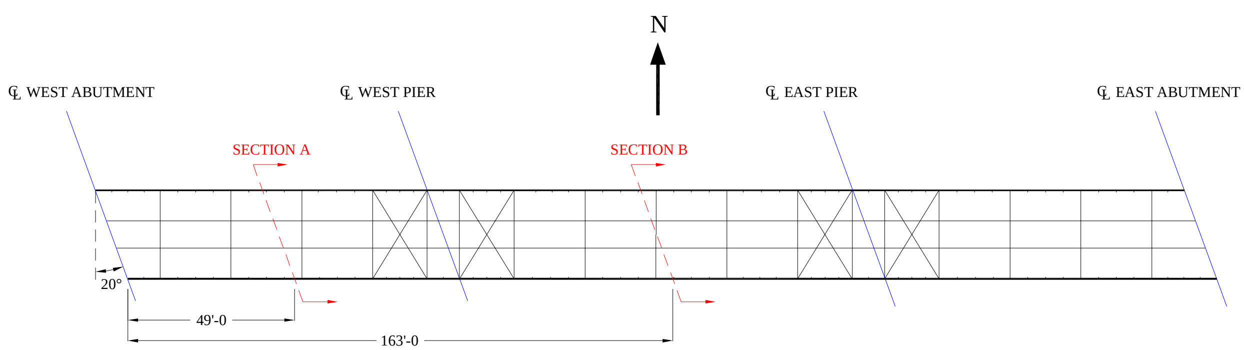
SYSTEM DEMONSTRATION

Once the development of the SHM system was completed, the system was tested and implemented on a highway bridge to demonstrate and verify its general usage. The bridge selected for demonstrating the use of the developed SHM system is the 320 ft. x 30 ft., three-span continuous, welded steel girder bridge shown in Figure 5. The bridge is located in central Iowa in Story County, Iowa, carrying US-30 over the

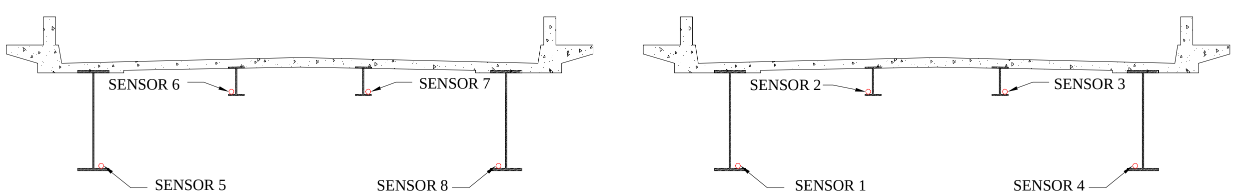
Skunk River near Ames, IA. The primary structural members are the two plate girders as the stringers are supported by floor beams which are then supported by the plate girders. The complete SHM system that was installed on the bridge uses an onsite computer to run the field monitoring software (i.e., process collected data and monitor for events and notify users any alarm events). The basic hardware components include sensors, the data acquisition hardware, and a communication system (Lee 2007). The selected quarter-bridge strain gages were installed at strategic points on the bridge. The locations of the strain gages were selected based primarily upon a preliminary engineering assessment but also with consideration of accessibility. To this end, four strain gages (Sensors 5 though 8) were installed in the positive moment region of the plate girders and stringers in the center span and four sensors (Sensors 1 though 4) in the west end span as shown in Figure 6. Note that all sensors were installed on the top of the respective member bottom flange.

(a) Side view (looking south east) **(b) Bottom view (looking east)**

Figure 5. Overall bridge photographs



(a) Plan view with sections A and B



(b) Section A in the west end span **(c) Section B in the center span**

Figure 6. Bridge strain gage location and reference sections

The data acquisition, processing, and communication system consists of the StrainBook/616 data acquisition instrument, a 1 GHz Dell desktop host PC, and a Linksys wireless router. These hardware components were installed in an environmentally controlled aluminum cabinet, shown in Figure 7, to protect them from weather and vandalism. The cabinet was mounted on the north corner of the west abutment wing wall and was supplied with electrical power through direct feed from an existing underground line (Note: power could also be supplied by solar power). The cabinet is equipped with a light bulb, a fan, and two thermostats to provide temperature control.

The StrainBook/616 data acquisition instrument and the host PC were both connected to the Linksys router with Ethernet cables, creating a local area network that allows direct communication among the hardware components (Lee 2007). The network at the bridge site was then, due to fortunate proximity, connected to the ISU network via wireless communication. For wireless communication between the bridge site and the ISU network, an antenna was mounted on an overhead sign frame that is located at the west end of the bridge (Note: connection to a network like this is not required). With an Internet connection available, users are able to:

- Access the host PC and operate the SHM system from anywhere in the world
- Receive processed and reduced data files and/or to be notified of any alarm events (e.g., overload occurrence and/or vehicle collision to the structure) via email.

Figure 7. Aluminum cabinet mounted on the north corner of the west abutment wing wall

Prior to the running the field monitoring software, the overload event thresholds for the sensors installed on the plate girders were determined using BEC Analysis. In each run, the bridge was subjected to various moving loads that include Iowa legal trucks: H 20 truck, HS 20 truck, and truck trains. After filter parameters were determined and all input settings were established (Lee 2007), the main program designed for data acquisition and processing was initialized at approximately 3 p.m. on November 14, 2006, after which continuous data collecting and processing have been completely autonomous and have required no intervention except when reviewing and generating evaluation reports. The contents in the evaluation report that was generated by one of the field monitoring software programs were reviewed.

Each data point in the event history plot in Figure 8a represents an event identified by the data processing algorithm. Along with the maximum daily event and average event, a linear best fit trend line for each sensor is given (see Figure 8b). In general, a sloping line with time is an indication in a change in bridge behavior/condition. After reviewing the evaluation report, several observations and interpretations were made for overall bridge performance during the 30 days of monitored period as follow:

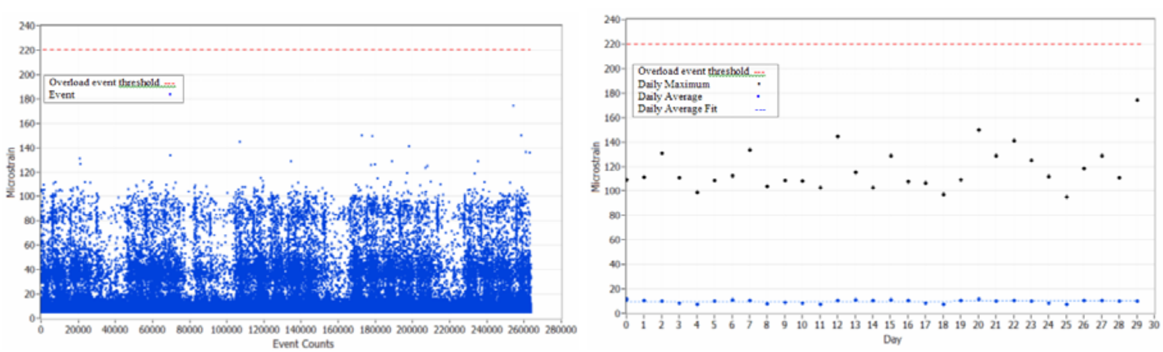
- No alarm event had occurred for the monitored period. The field monitoring software was programmed to list those events, if any, that exceed the overload event thresholds for each sensor, and no identified events exceeded the overload event thresholds that were set.
- The magnitudes of the daily maximum events fluctuate from day to day (see Figure 8b and Table

1). It should be noted, however, that the absolute maximums do not necessarily represent the gradual change in performance of the bridge. Rather, they simply represent individual events induced by "heavy" vehicles in different days.

- The daily average of identified event is less likely to show variability to single "heavy" traffic event (see Figure 8b and Table 1). Therefore, a gradual performance change can be estimated or predicted by investigating the daily average and the slope change over time. By reviewing the daily average of identified events for each sensor as illustrated in Figure 8b, it appears that the overall performance of the bridge was consistent for the monitored period (as would be expected). This observation was made by investigating the slope change (see Figure 8b and Table 2) of the daily average fit curve that is essentially zero for all sensors. If the condition of the bridge starts to change (due to deterioration, etc.) without a significant change in traffic pattern, the structural response of the bridge will also change and, therefore, the daily average is expected to change.

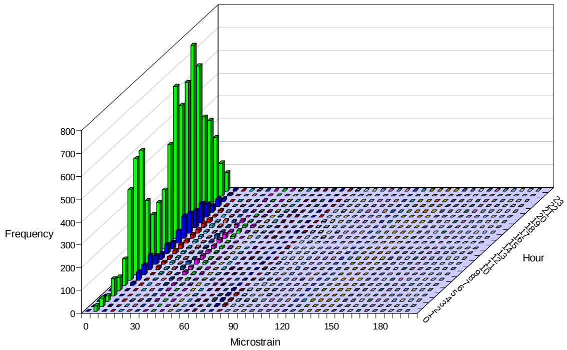
In order to provide hour-to-hour and day-to-day comparisons of the bridge response, 24-hour hourly event histograms and 30-day daily event histograms for Sensor 4 (typical of all sensors) were created and presented in Figure 9. After reviewing and comparing the histograms, several observations were made as follows:

- The numerical counts of identified events are different from hour-to-hour and day-to-day as expected. The variation in the number of identified events within the daily event histograms is less than that within the hourly event histogram. This was expected as hour-to-hour traffic patterns vary more than day-to-day traffic patterns.
- Although it does not represent exact traffic counts, the variation in the number of identified events within one chart is directly related to the traffic volume traversing the bridge in a given period.
- In all event histograms, there are dominant bins with high concentration of identified events.
- It is expected that, if the structural response of the bridge changes due to deterioration and/or damage with no significant change in traffic patterns, the dominant bins in the event histogram plot will be distributed across several bins and/or shifted.

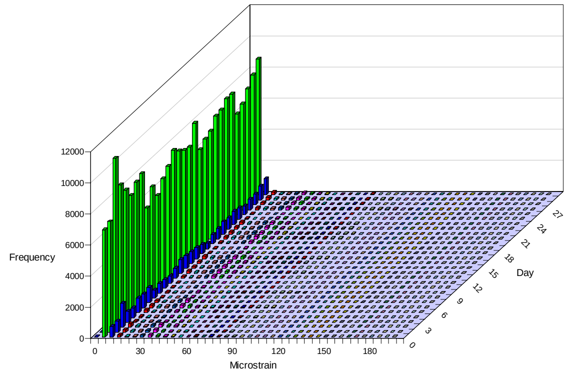


(a) 30-day event history for Sensor 4 **(b) 30-day daily statistical trends for Sensor 4**

Figure 8. Monthly evaluation from Nov. 14th through Dec. 14th of 2006



(a) Hourly event histogram



(b) Daily event histograms

Figure 9. Hourly and daily event histograms for Sensor 4 (bin width: 5 microstrain)

Table 1. Statistical trends (daily maximum/average in microstrain)

	Sensor1	Sensor2	Sensor3	Sensor4	Sensor5	Sensor6	Sensor7	Sensor8
Day 0	105/11	51/13	47/11	110/12	93/11	80/11	68/11	106/11
Day 1	102/11	41/12	46/13	111/11	99/11	76/11	71/11	105/10
Day 2	105/10	47/11	64/13	131/10	95/9	77/10	104/10	122/10
Day 3	102/9	41/12	47/13	111/9	91/9	74/9	68/8	93/9
Day 4	97/8	42/10	43/10	99/8	89/8	65/7	67/7	93/8
Day 5	105/10	37/11	47/13	109/10	88/10	68/11	73/10	106/10
Day 6	110/10	37/12	48/13	113/11	94/11	67/11	75/11	100/11
Day 7	137/10	51/11	53/12	134/11	112/10	80/10	101/10	110/10
Day 8	98/8	37/11	46/12	104/8	85/8	67/8	72/7	96/8
Day 9	165/10	41/12	45/13	109/9	156/10	84/10	74/8	103/9
Day 10	94/9	35/13	45/14	108/9	79/9	67/9	69/8	102/8
Day 11	82/8	34/10	39/11	103/8	75/8	54/7	62/7	91/7
Day 12	110/10	39/12	48/13	145/11	96/10	69/11	86/10	122/10
Day 13	119/10	38/12	52/14	115/11	124/10	71/11	86/11	103/11
Day 14	109/10	39/11	53/13	103/11	91/10	70/11	81/10	111/10
Day 15	130/11	78/12	91/14	129/12	114/11	69/11	82/11	100/11
Day 16	109/10	37/12	43/13	108/11	108/10	69/11	67/10	94/10
Day 17	101/9	38/13	46/14	106/9	87/9	66/9	69/8	99/8
Day 18	97/8	49/12	45/12	98/8	77/8	90/8	82/7	90/7
Day 19	104/10	35/11	45/12	109/11	90/11	65/11	71/10	104/10
Day 20	134/11	62/12	48/14	150/12	102/11	103/12	116/11	129/11
Day 21	107/10	40/11	41/13	129/10	96/10	69/11	74/10	134/10
Day 22	106/10	35/12	45/14	141/11	102/11	66/12	72/11	151/10
Day 23	126/10	50/11	54/13	125/10	99/10	92/10	90/10	131/10
Day 24	87/9	35/12	45/14	112/9	85/9	63/9	73/8	99/8
Day 25	86/8	36/11	39/12	96/8	79/8	57/7	62/7	84/7
Day 26	108/10	44/11	43/13	118/11	97/10	75/11	69/10	105/10
Day 27	109/10	40/12	46/14	129/11	98/10	68/11	70/11	109/10
Day 28	103/10	39/11	43/13	111/10	90/10	65/11	72/10	98/10
Day 29	141/10	48/12	58/13	174/11	120/10	96/11	97/10	167/10

Table 2. Overall summary of 30-day monitoring

Sensor	Overload event threshold (με)	Maximum event (με)	Average (με)	Daily average slope change
1	221	165	10	0
2	-	78	12	0
3	-	91	13	0
4	221	174	10	0
5	219	156	10	0
6	-	103	10	0
7	-	116	9	0
8	219	167	10	0

CONCLUSIONS

The following conclusions can be made regarding the cost, development, installation, and the overall performance of the SHM system:

- The developed low-cost SHM system is suitable for implementation of typical girder bridges. Excluding the communication and power equipments and research and development costs, the system can be implemented at the cost of $8,000 to $15,000 depending on the number of sensors used.
- The field monitoring software was developed such that it can handle up to 16 channels (one eight-channel StrainBook/616 plus one WBK16 eight-channel expansion module). Although the WBK16 was not included in the SHM system, its usage was tested during development.
- The installation of the strain gages and laying out the cables required no training or special equipment other than safety and normal access equipment. Although the time required for sensor installation was only around 30 minutes per gage including surface preparation, securing the sensor cable required more time and was relatively labor intensive. A two-man crew was used to install the strain gages and to secure the cables over a two-day period.
- Based upon comparisons with commercial analysis software, the live load structural analysis software, BEC Analysis, has been proven to be accurate.
- During a little over 30 days of monitoring period, the SHM system has performed as expected and has proven to be capable of continuously and autonomously monitoring the overall performance of the US-30 bridge.
- The SHM system has been proven to be a stand-alone, autonomous system capable of processing and evaluating the continuously collected strain data in the US-30 bridge.
- If properly implemented, the developed system will allow owners to monitor and control overloads and provide better access to valuable traffic information that can be used in planning, maintenance, and construction activities. Another benefit of the system is its relative ease of implementation and relative low cost. Overall, it is believed that the use of the SHM system developed herein will provide owners the tools to better manage bridge assets.

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Determining the Effectiveness of Temporary Traffic Control Measures in Highway Work Zones

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ABSTRACT

Highway work zones constitute a major safety concern for government agencies, legislatures, the highway industry, and the traveling public. Each year, hundreds of people lose their lives and many more are injured due to vehicle crashes in work zones on the national highway system. Over the years, temporary traffic control (TCC) measures have been developed to improve the safety in work zones. To improve the safety countermeasures and to identify the traffic control deficiencies in work zones, evaluating the effectiveness of existing TTC measures based on the real crash experience is necessary. In the present study, researchers evaluated the effectiveness of several commonly used TTC methods using logistic regression technique and various chi-square statistics. The assessed TTC methods included flagger/officer, stop sign/signal, flasher, no passing zone control, and pavement center/edge lines. A total of 655 severe crashes in Kansas highway work zones between January 2003 and December 2004 were used for the evaluation, which included 29 fatal crashes and 626 injury crashes. The results indicated that flagger, flasher, and pavement center/edge lines were effective in reducing the probability that work zone crashes would involve fatalities. In addition, the effectiveness of these devices in preventing some common human errors, such as “disregarded traffic control,” “inattentive driving,” “followed too closely,” and “exceeded speed limit or too fast for condition,” from causing severe crashes was also determined.

Key words: Kansas—logistic regression—safety—traffic control effectiveness—work zone

INTRODUCTION

Highway work zones constitute a major safety concern for government agencies, the legislature, the highway industry, and the traveling public. The number of people killed in motor vehicle crashes in work zones rose from 872 in 1999 to 1,028 in 2003 in the United States. In addition, approximately 40,000 people are injured each year as a result of motor vehicle crashes in work zones. Today, the majority of highway funds are being allocated to road and bridge preservation and enhancement, which means the traveling public is encountering more and more highway work zones.

Over the years, many temporary traffic control (TTC) measures have been developed and deployed in highway work zones. The primary function of these TTC measures in work zones is to provide highway users reasonably safe and efficient movement through work zones while protecting construction workers and equipment. There is a consensus that the use of TTC measures improves safety in the work zones if they are designed, installed, and maintained properly. However, it is not clear the extent to which safety has been improved by using these measures. To determine the effectiveness of the safety countermeasures in work zones, there is a need to quantify the effectiveness of existing TTC measures.

RESEARCH OBJECTIVES AND METHODOLOGY

Among all possible work zone crashes, crashes involving injuries or fatalities are the most severe and calamitous. Reducing these crashes will yield the most benefit to society. The objective of this research project was to quantify the effectiveness of several popular TTC measures, including flagger/officer, stop sign/signal, flasher, no passing zone, and pavement center/edge lines, in reducing fatalities when a severe crash occurs and in preventing common human errors from causing work zone severe crashes.

The project was conducted using a four-step approach. First, an extensive literature search was performed to review the previous investigations in this area. Second, fatal and injury crash data were extracted from the Kansas Department of Transportation (KDOT) accident database. A total of 655 severe crashes, including 29 fatal crashes and 626 injury crashes, in Kansas highway work zones between January 2003 and December 2004 were collected for the evaluation. Third, logistic regression analysis was used to evaluate the effectiveness of the safety measures in these work zones. Finally, conclusions and recommendations for future research were formulated based on the results of data analyses.

LITERATURE REVIEW

A highway work zone refers to a road section undergoing a construction or maintenance project. When the normal function of the highway is suspended around a work zone, a TTC plan must be developed to provide continuity of movement for motor vehicles. As included in the Manual of Uniform Traffic Control Devices (MUTCD), some TTC methods that are commonly used in work zones include flaggers, traffic signs, arrow panels and portable changeable message signs, channelizing devices, pavement markings, lighting devices, and temporary traffic control signals (FHWA 2003). To provide the background knowledge on work zone traffic control, a review of these traffic control devices and their related studies is presented in this section.

Flagger Control

Flaggers are qualified personnel with high-visibility safety apparel who are equipped with handheld devices such as STOP/SLOW paddles, lights, and red flags to control road users through work zones. Richards and Dudek (1986) suggested that flaggers have been most efficient on two-lane, two-way rural highways and urban arterials, where they had the least competition for drivers' attention; flaggers were also well suited for short-duration applications (less than one day) and for intermittent use at long-duration work zones. Garber and Woo (1990) concluded that the most effective combination of traffic control devices for work zones on multilane highways was cones, flashing arrows, and flaggers, and the effective combinations of traffic control devices for work zones on urban two-lane highways were both cones and flaggers as well as static signs and flaggers. Hill (2003) proved that flaggers were effective in reducing fatal work zone crashes. However, a study by Benekohal et al. (1995) indicated that there was a need for improving flagging for heavy truck traffic. Their survey showed that one-third of the surveyed truck drivers responded that flaggers were hard to see, and half of them thought the directions of flaggers were confusing.

Traffic Signs

As listed in the MUTCD, traffic signs in work zones include regulatory signs, warning signs, and guide signs. Traffic signs in work zones are important for informing travelers about interrupted traffic conditions. A survey indicated that half of truck drivers wanted to see warning signs 3–5 miles in advance (Benekohal, Shim, and Resende 1995). Garber and Woo (1990) found that static traffic signs could effectively reduce crashes in work zones on urban two-lane highways when used together with flaggers.

Arrow Panels and Portable Changeable Message Signs

Arrow panels and portable changeable message signs usually contain luminous panels with high visibility, which makes them an ideal traffic control supplement in both daytime and nighttime. Garber and Patel (1994) and Garber and Srinivasan (1998) conducted a two-phase research project to evaluate the effectiveness of changeable message signs for controlling speeds in work zones in Virginia. The changeable message signs displayed a real-time warning message to the speeding drivers after their excessive speeds were detected by its actuator. The researchers concluded that changeable message signs were a more effective means than traditional work zone traffic control devices in reducing the number of speeding vehicles in work zones. Richards and Dudek (1986) commented that changeable message signs could result in only modest speed reductions (less than 10 mph) when used alone and would lose their effectiveness when operated continuously for long periods with the same messages. Huebschman et al. (2003) argued that changeable message signs were actually no more effective than traditional message panels.

Channelizing Devices

Channelizing devices are used to warn road users of changed traffic conditions in work zones and to guide travelers to drive safely and smoothly through work zones. Channelizing devices include cones, tubular markers, vertical panels, drums, barricades, and temporary raised islands. The results of a study (Pain et al. 1983) showed that most of the channelizing devices were effective in alerting and guiding drivers, but the devices only obtained their maximum effectiveness when properly deployed as a system or array of devices. Garber and Woo (1990), however, found that the use of barricades in any combination of traffic control devices on urban multilane highways seemed to reduce the effectiveness of other traffic control devices.

Temporary Pavement Markings

Temporary pavement markings are used along paved highways in long- and intermediate-term stationary work zones to outline the travel paths. Pavement markings can be used to control speeds. For instance, a traffic control strategy using optical speed bars modified to meet the conditions of highway work zones has been applied to control speeds in work zones. Optical speed bars are an innovative speed control technique that uses transverse stripes spaced at gradually decreasing distances on pavement to affect the driver's perception of speed. Meyer (2004) conducted a study to evaluate the effectiveness of this strategy in reducing work zone speed in Kansas. The study showed that the speed bars had both a warning effect and a perceptual effect and were effective in controlling speeds and reducing speed variations.

Lighting Devices

Lighting devices are used based on engineering judgment to supplement retroreflectorized signs, barriers, and channelizing devices. Four lighting devices commonly used in work zones are floodlights, flashing warning beacons, warning lights, and steady burn electric lamps. These devices raise drivers' attention, warn drivers of complicated travel conditions, and/or illuminate work zones at night. Some studies (Huebschman et al. 2003; Arnold 2003) found that flashing warning lights, especially police vehicles with flashing lights, were one of the most effective approaches for reducing speeds in work zones.

Temporary Traffic Control Signals

Temporary traffic control signals are typically used for conditions such as temporary one-way operations in work zones with one lane open and work zones involving intersections. The MUTCD suggests that temporary traffic control signals should be used with other traffic control devices, such as warning and regulatory signs, pavement markings, and channelizing devices. Some analyses of work zone fatal crashes have shown that certain temporary traffic control signals, such as STOP/GO signals, have been very effective in reducing fatal crashes in work zones (Hill 2003).

DATA COLLECTION

A total of 655 severe work zone crashes, including 29 fatal cases and 626 injury cases, were extracted from the KDOT accident database. Table 1 shows the variables and their observations. Because the observations in the database were in text format, a numerical value was assigned to each observation to facilitate the regression analyses. At the end of data collection, crash information represented by numerical values was compiled into a spreadsheet where a crash was described in one data row. Then, the spreadsheet was inputted into the SAS software for analyses.

Table 1. Crash data and assigned numerical numbers

Variable	Observation	Assigned Numerical Value
Crash Severity	Fatal	1
	Injury	2
Traffic Control	Flagger/officer	1
	Stop sign/signal	2
	Flasher	3
	No passing zone control	4
	Center/edge lines	5
Driver Error	Inattentive driving	1
	Disregarded traffic signs, signals, or markings	2
	Followed too closely	3
	Exceeded speed limit or too fast for conditions	4

BINARY LOGISTIC REGRESSION METHOD

Binary logistic regression technique was used to evaluate the effectiveness of the TTC methods commonly used in work zones. Binary logistic regression is a statistical method developed specifically for describing the relationships between a set of independent explanatory variables and a dichotomous response variable or outcome. Because the analysis of the effectiveness of TTC measures involves establishing the relationships between the occurrence of a crash and the presence of a TTC measure, this regression technique has been applied in previous traffic crash studies (Hill 2003; Dissanayake and Lu 2002). A binary logistic regression model is a direct probability model that has no requirements on the distributions of the explanatory variables or predictors (Harrell 2001). It is flexible and is likely to yield accurate results when applied to traffic crash analysis in which the safety effectiveness of TTC measures needs to be quantified.

The following briefly describes the theoretical basis of the binary logistic regression method. Let Y be an event ($Y = 1$ and $Y = 0$ denote occurrence and nonoccurrence, respectively) and let a vector $\boldsymbol{X}$ be a set of predictors $\{X_1, X_2, \ldots, X_k\}$. The expected value of Y given $\boldsymbol{X}$ is the probability (P) of the occurrence of Y given $\boldsymbol{X}$**,** which can be expressed in linear regression form as follows:

$$E\{Y|\boldsymbol{X}\} = P\{Y = 1|\boldsymbol{X}\} = \boldsymbol{X\beta} \qquad (1)$$

where $\boldsymbol{\beta}$ is the regression parameter vector and $\boldsymbol{X\beta}$ stands for $\beta_0 + \beta_1 X_1 + \ldots + \beta_k X_k$.. Because the probability determined by this equation can exceed one, the following binary logistic regression model is generally preferred for the analysis of binary responses:

$$P\{Y = 1|\boldsymbol{X}\} = [1 + exp(-\boldsymbol{X\beta})]^{-1} = exp(\boldsymbol{X\beta})/[1 + exp(\boldsymbol{X\beta})] \qquad (2)$$

The above equation can be expressed in the following logistic form:

$$logit\{Y = 1|\boldsymbol{X}\} = log[P/(1 - P)] = \beta_0 + \beta_1 X_1 + \ldots + \beta_k X_k \qquad (3)$$

For the above model, given the estimated β's as $\hat{\beta}_0, \hat{\beta}_1, \cdots, \hat{\beta}_k$, the estimated probability $\hat{P}$ that an event Y happens, can be computed as follows:

$$\hat{P}\{Y = 1|\boldsymbol{X}\} = exp(\boldsymbol{X}\hat{\boldsymbol{\beta}})/[1 + exp(\boldsymbol{X}\hat{\boldsymbol{\beta}})] \qquad (4)$$

where $\boldsymbol{X}\hat{\boldsymbol{\beta}}$ stands for $\hat{\beta}_0 + \hat{\beta}_1 X_1 + \cdots + \hat{\beta}_k X_k$

The significance of a predictor can be tested using the methods of the likelihood ratio test, the Wald test, and the score test (Hosmer and Lemeshow 2000). The likelihood ratio test compares the deviation of the model with the predictor to that without the predictor. The Wald test is obtained by comparing the maximum likelihood estimate of the slope parameter, β_1, to an estimate of its standard error. The score test is based on the distribution theory of the derivatives of the log likelihood. Quantifying the safety impact of an explanatory variable can be treated as a special logistic regression case:

$$logit\{Y = 1|X = 0\} = \beta_0 \qquad (5)$$

$$logit\{Y = 1|X = 1\} = \beta_0 + \beta_1 \qquad (6)$$

Accordingly, the estimated probability that an event happens ($Y = 1$) when the test factor is present ($X = 1$) is as follows:

$$\hat{P}\{Y = 1|X = 1\} = exp\{\hat{\beta}_0 + \hat{\beta}_1\}/(1 + exp\{\hat{\beta}_0 + \hat{\beta}_1\}) \tag{7}$$

The estimated probability that this event happens ($Y = 1$) when the test factor is absent ($X = 0$) is

$$\hat{P}\{Y = 1|X = 0\} = exp\{\hat{\beta}_0\}/(1 + exp\{\hat{\beta}_0\}) \tag{8}$$

In this study, odds ratio was used to measure the difference between the univariate logistic regression model pairs. Odds ratio is defined as the ratio of the two odds given the two values of the test variable. Given the estimated odds of an event $\{Y = 1|X\}$ as

$$Odds\{Y = 1|X\} = \hat{P}\{Y = 1|X\}/(1 - \hat{P}\{Y = 1|X\}) \tag{9}$$

the odds ratio for the single-variable case is

$$Odds\ ratio\ (X = x_1 : X = x_2) = exp[\hat{\beta}_1\ (x_1 - x_2)] \tag{10}$$

EVALUATING THE EFFECTIVENESS OF WORK ZONE TTC METHODS

Based on the available crash information, the effectiveness of several commonly used work zone TTC methods was evaluated. The effectiveness was evaluated in terms of reducing the severity of work zone crashes and preventing major human errors from causing severe work zone crashes. The crash data used for the evaluation included the fatal and injury work zone crashes in Kansas highway work zones between January 2003 and December 2004. The evaluated TTC methods included flagger/officer, stop sign/signal, flasher, no passing zone, and center/edge lines; the major human errors that were included in the evaluation were “inattentive driving,” “disregarded traffic control,” “followed too closely,” and “exceeded speed limit or too fast for condition.”

Effectiveness of Flagger/Officer Control

For estimating the effectiveness of flagger/officer control in reducing the severity of work zone crashes, the response variable Y represented a severe crash ($Y = 1$ for fatal crashes and $Y = 2$ for injury crashes) and the explanatory variable X represented the presence of a flagger ($X = 1$ for presence and $X = 0$ for absence). The logistic regression model was estimated as follows:

$$logit\{Y = 1|X\} = -2.42 - 0.81X. \tag{11}$$

The three test-of-significance statistics (likelihood ratio, score, and Wald) all indicated a high level of significance (i.e., 0.01) for the flagger variable.

According to this model, the conditional probability of having fatalities, given the occurrence of a severe crash (either fatal or injury), when flagger control was present was estimated as follows:

$$\hat{P}\{Y = 1|X = 1\} = exp\{\hat{\beta}_0 + \hat{\beta}_1\}/(1+ exp\{\hat{\beta}_0 + \hat{\beta}_1\})= 0.04 \quad (12)$$

The corresponding probability without a flagger control was as follows:

$$\hat{P}\{Y = 1|X = 0\} = exp\{\hat{\beta}_0\}/(1+ exp\{\hat{\beta}_0\})= 0.08 \quad (13)$$

The estimated odds ratio between the occurrence of a fatal crash with flagger control and without flagger control was:

$$Odds\ ratio\ (X = 1 : X = 0) = exp[\hat{\beta}_1\ (x_1 - x_2)] = exp[-0.81\times(1 - 0)] = 0.44. \quad (14)$$

Hence, statistically, using a flagger in a work zone could reduce the odds of a severe crash resulting in fatality by 56%. In terms of probability, the presence of a flagger in a work zone could lower the probability of causing fatalities by 4% (or from 0.08 to 0.04) when a severe crash occurred.

Previous work zone crash studies (Bai and Li 2006; 2007) have shown that human errors contribute to a significant proportion of work zone severe crashes. Reducing risky driver errors would be an important objective for work zone TTC methods to accomplish. The effectiveness of the flagger/officer control in work zones in preventing major human errors from causing severe (fatal and injury) crashes was also evaluated in this study. In the evaluations, the response variable *Y* represented a severe crash that was either caused by a studied human error ($Y = 1$) or not caused by this human error ($Y = 0$). For example, to evaluate the effectiveness of a flagger in preventing "disregarded traffic control" from causing fatal or injury crashes, the logistic regression model was fitted as follows:

$$logit\{Y = 1|X\} = - 1.78 - 0.77X. \quad (15)$$

According to this model, the conditional probability of the crash caused by "disregarded traffic control," given the occurrence of this severe crash, when flagger control was present was estimated as follows:

$$\hat{P}\{Y = 1|X = 1\} = exp\{\hat{\beta}_0 + \hat{\beta}_1\}/(1+ exp\{\hat{\beta}_0 + \hat{\beta}_1\})= 0.07 \quad (16)$$

The corresponding probability without a flagger control was as follows:

$$\hat{P}\{Y = 1|X = 0\} = exp\{\hat{\beta}_0\}/(1+ exp\{\hat{\beta}_0\})= 0.14 \quad (17)$$

The estimated odds ratio between the severe crash being caused by "disregarded traffic control" human error with flagger control and without flagger control was as follows:

$$Odds\ ratio\ (X = 1 : X = 0) = exp[\hat{\beta}_1\ (x_1 - x_2)] = exp[-0.77\times(1 - 0)] = 0.46. \quad (18)$$

These results indicate that using a flagger in a work zone could reduce the odds of a severe crash being caused by "disregarded traffic control" human error by 54%. In terms of conditional probability, the presence of a flagger in a work zone could lower the probability of causing a severe crash due to "disregarded traffic control" by 7% (or from 0.14 to 0.07) when a severe crash occurred. Table 2 lists the parameters and the estimated probabilities and odds ratio of the fitted logistic regression models for the effectiveness of the flagger/officer control in reducing crash severity and preventing human errors, such as “disregarded traffic control,” “inattentive driving,” and “exceeded speed limit or too fast for condition,” from causing severe crashes. As illustrated in the table, using a flagger/officer in a highway work zone could lower the odds of having a severe crash caused by “inattentive driving” or “exceeded speed limit or too fast for condition” by about 40%. The effectiveness of a flagger/officer in preventing the impact of “followed too closely” was not determined because none of the statistical tests supported the significant relationship between the traffic control and the driver error.

Table 2. Model parameters and evaluation results for flagger control

Parameter	Coefficient $\hat{\beta}_0$	Coefficient $\hat{\beta}_1$	p-Value of Significance Test* LR***	p-Value of Significance Test* Score	p-Value of Significance Test* Wald	Probability $X = 1$	Probability $X = 0$	Odds Ratio $(X = 1 : X = 0)$**
Effectiveness in reducing crash severity	-2.42	-0.81	<ins>0.01</ins>	<ins>≤0.01</ins>	<ins>≤0.01</ins>	0.04	0.08	0.44
Effectiveness in preventing “disregarded traffic control”	-1.78	-0.77	<ins>≤0.01</ins>	<ins>≤0.01</ins>	<ins>≤0.01</ins>	0.07	0.14	0.46
Effectiveness in preventing “inattentive driving”	0.34	-0.51	<ins>0.01</ins>	<ins>0.01</ins>	<ins>0.01</ins>	0.46	0.58	0.60
Effectiveness in preventing “exceeded speed limit or too fast for condition”	-1.01	-0.46	<ins>0.03</ins>	<ins>0.02</ins>	<ins>0.02</ins>	0.19	0.27	0.63

*: p-Value is the output value of the statistical tests of significance. A p-value less than 0.1 indicates that the test variable is significant at 0.1 level of significance, and is underlined in the table.
**: X = 1 when the traffic control was present and X = 0 when it was absent.
***: Likelihood Ratio.

Effectiveness of Stop Sign/Signal

The stop sign/signal control was tested for its effectiveness in reducing crash severity and preventing the major human errors from causing severe crashes. The tests of significance showed that the presence of a stop sign/signal control device in a work zone was not significantly related to the involvement of fatalities in severe crashes. In addition, the tests showed that this TTC method could actually catalyze the “followed too closely” human error to cause severe crashes. As listed in Table 3, when a stop sign/signal was used, the odds of having crashes caused by “following too closely” was roughly two and a half times higher than the odds without such a device.

Table 3. Model parameters and evaluation results for stop sign/signal control

Parameter	Coefficient $\hat{\beta}_0$	Coefficient $\hat{\beta}_1$	p-Value of Significance Test* LR***	p-Value of Significance Test* Score	p-Value of Significance Test* Wald	Probability $X = 1$	Probability $X = 0$	Odds Ratio $(X = 1 : X = 0)$**
Effectiveness in preventing “followed too closely”	-2.38	1.26	<ins>≤0.01</ins>	<ins>≤0.01</ins>	<ins>0.01</ins>	0.25	0.08	3.53

*: p-Value is the output value of the statistical tests of significance. A p-value less than 0.1 indicates that the test variable is significant at 0.1 level of significance, and is underlined in the table.
**: X = 1 when the traffic control was present and X = 0 when it was absent.
***: Likelihood Ratio.

Effectiveness of Flasher Device

Statistical tests showed that the use of flashers in work zones was not directly related to the involvement of the four major human errors in the severe work zone crashes. However, the effectiveness of flashers in mitigating the severity of work zone crashes was supported by statistical tests and thus was determined. Using the SAS software, the following logistic regression model was generated:

$$logit\{Y = 1|X\} = -2.24 - 0.86X. \tag{19}$$

Listed in Table 4 are the results of the three tests of significance and the respective probabilities of a severe crash resulting in fatalities with and without a flasher control device. The odds ratio of having a fatal crash with and without a flasher control device is also included in the table. The results indicated that using a flasher device in a work zone could reduce the odds of a severe crash resulting in fatalities by 58%.

Table 4. Model parameters and evaluation results for flasher control

Parameter	Coefficient		p-Value of Significance Test*			Probability		Odds Ratio
	$\hat{\beta}_0$	$\hat{\beta}_1$	LR***	Score	Wald	*X = 1*	*X = 0*	*(X = 1 : X = 0)***
Effectiveness in reducing crash severity	-2.24	-0.86	0.20	<u>0.09</u>	0.13	0.04	0.10	0.42

*: p-Value is the output value of the statistical tests of significance. A p-value less than 0.1 indicates that the test variable is significant at 0.1 level of significance, and is underlined in the table.
**: X = 1 when the traffic control was present and X = 0 when it was absent.
***: Likelihood Ratio.

Effectiveness of "No Passing Zone" Control

The results of the three tests of significance, including likelihood ratio test, score test, and Wald test, all suggested that the use of work zone no passing zone controls was significantly related to the odds of a severe crash caused by "disregarded traffic control" human error. Table 5 lists the evaluation results for work zone no passing zone controls. The results indicated that, in a work zone no passing zone, the odds of a severe crash caused by "disregarded traffic control" human error would be 29% less than that in work zones without no passing zones.

Table 5. Model parameters and evaluation results for no passing zone control

Parameter	Coefficient		p-Value of Significance Test*			Probability		Odds Ratio
	$\hat{\beta}_0$	$\hat{\beta}_1$	LR***	Score	Wald	*X = 1*	*X = 0*	*(X = 1 : X = 0)***
Disregarded traffic control	-2.20	-0.35	<u>0.06</u>	<u>0.04</u>	<u>0.05</u>	0.07	0.10	0.71

*: p-Value is the output value of the statistical tests of significance. A p-value less than 0.1 indicates that the test variable is significant at 0.1 level of significance, and is underlined in the table.
**: X = 1 when the traffic control was present and X = 0 when it was absent.
***: Likelihood Ratio.

Effectiveness of Pavement Center/Edge Lines

Statistical study showed that the use of center/edge lines in work zones was effective not only for reducing crash severity, but also in preventing human errors such as "exceeded speed limit or too fast for condition" and "followed too closely" from causing severe crashes. Table 6 shows the results in terms of the estimated probabilities and the odds ratio. The regression analyses' results suggested that the use of center/edge lines in work zones may reduce the odds of causing fatalities when severe crashes occurred by 55%. In addition, having center/edge lines in work zones may also lower the odds of a severe crash caused by speeding by 29%, and the odds of a severe crash caused by "followed too closely" by 19%.

Table 6. Model parameters and evaluation results for center/edge lines

Parameter	Coefficient $\hat{\beta}_0$	Coefficient $\hat{\beta}_1$	p-Value of Significance Test* LR***	p-Value of Significance Test* Score	p-Value of Significance Test* Wald	Probability $X = 1$	Probability $X = 0$	Odds Ratio $(X = 1 : X = 0)$**
Effectiveness in reducing crash severity	-3.63	-0.80	<u>0.01</u>	<u>0.02</u>	<u>0.03</u>	0.01	0.03	0.45
Effectiveness in preventing "exceeded speed limit or too fast for condition"	-1.61	-0.35	<u><0.01</u>	<u>0.01</u>	<u>0.01</u>	0.12	0.17	0.71
Effectiveness in preventing "followed too closely"	-1.30	-0.20	<u>0.06</u>	<u>0.07</u>	<u>0.07</u>	0.18	0.21	0.81

*: p-Value is the output value of the statistical tests of significance. A p-value less than 0.1 indicates that the test variable is significant at 0.1 level of significance, and is underlined in the table.
**: X = 1 when the traffic control was present and X = 0 when it was absent.
***: Likelihood Ratio.

CONCLUSIONS

Work zone safety has been a research focus for many years, and improving the safety in highway work zones is a high-priority task for traffic engineers. Evaluating the effectiveness of the TTC methods used in highway work zones is a practical approach to safety improvement. In this study, the effectiveness of several TTC methods in mitigating work zone crash severity and preventing common human errors from causing severe work zone crashes was quantified using a logistic regression technique. These TTC methods included flagger/officer, stop sign/signal, flasher, no passing zone, and center/edge lines. The evaluation was intended to provide valuable knowledge for developing more effective traffic control strategies. The results may also provide useful indications regarding safety levels in certain types of work zones where the evaluated TTC methods are regularly installed.

In this research, the TTC methods were evaluated to quantify their effectiveness in both reducing crash severity and preventing common human errors. According to the logistic regression analyses, the presence of a flagger or officer directing traffic could reduce the odds of having fatalities in a severe crash by 56%; having flashers or center/edge lines in work zones could reduce the odds by more than 50% as well. However, based on the available crash data, the statistics could not establish close associations between the usages of stop signs/signals and no passing zones in work zones and the involvement of fatalities in severe crashes.

Regarding the effectiveness TTC methods in preventing common human errors from causing severe crashes in work zones, the evaluation showed that flaggers/officers could considerably lower the odds of severe work zone crashes caused by human errors such as "disregarded traffic control," "inattentive driving," and "exceeded speed limit or too fast for condition." No passing zones in work zones were effective in reducing the odds of "disregarded traffic control" causing severe crashes. In addition, having center/edge lines in work zones could lower the odds of human errors, including "exceeded speed limit or too fast for condition" and "followed too closely," causing severe work zone crashes. However, having stop signs/signals in work zones would dramatically increase the odds of severe crashes caused by "followed too closely" human error.

This research project can be extended in several ways, and recommendations for future research include four items. First, fatal crash data from other sources could be added to increase the total number of fatal cases in order to improve the reliability of the analysis. In this project, researchers only examined data from the state of Kansas due to limited resource availability. In the future, researchers could collect data from the work zones in other states to improve the accuracy of the analysis. Second, evaluating the effectiveness of the TTC methods may be extended to property damage–only crashes. When possible, the evaluation should also consider the data such as traffic volume and vehicle-miles traveled so that the TTC measures' effectiveness in reducing the total number of crashes can be determined. Third, there is a need to evaluate the effectiveness of certain combinations of TTC devices that are commonly used in work zones. The results of such studies will provide a comprehensive understanding on how these TTC measures interactively improve safety in work zones. Finally, there is a need to continuously develop new safety countermeasures and deploy them in the work zones.

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Investigating the Human Factors Involved in Severe Crashes in Highway Work Zones

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ABSTRACT

Highway work zones create an inevitable disruption in regular traffic flows and result in traffic crashes with injuries or fatalities. In many of these crashes, driver errors have been reported as the most significant causal factors. Understanding the role of human factors in these work zone crashes and then lowering the probability that they will cause crashes is critical for improving safety in highway work zones.

In this study, the role of human factors in severe work zone crashes involving fatalities or injuries was investigated. The objectives of the research were as follows: (1) to explore the influence of human factors on the occurrences and characteristics of fatal and injury work zone crashes and (2) to investigate the effectiveness of work zone temporary traffic control (TTC) devices in preventing human errors from causing severe crashes in work zones. The severe crashes that occurred in Kansas highway work zones during 1992 and 2004 were studied in detail. Statistical techniques such as Pearson chi-square, likelihood ratio chi-square, and logistic regression were utilized in the investigation. Through the systematic study, the researchers discovered the major impacts of several common driver errors on severe work zone crashes; the researchers also evaluated the effectiveness of some TTC methods in reducing these driver errors in highway work zones. The results of this study can provide knowledge that can facilitate the development of effective countermeasures for eliminating risky driver errors, ultimately in order to improve safety level in work zones.

Key words: human factors—Kansas—safety—traffic control effectiveness—work zone

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INTRODUCTION

Work zones on the national highway system result in safety concerns for the traveling public. Crash investigations have shown that highway work zones not only increase crashes rates (Garber and Zhao 2002; AASHTO 1987; Ullman and Krammes 1990), but also significantly increase the number of severe crashes (Garber and Zhao 2002; Ullman and Krammes 1990; Pigman and Agent 1990; Nemeth and Migletz 1978; AASHTO 1987). According to crash statistics, 1,068 people were killed in work zones in 2004, adding to about 49,620 more work zone–related injuries (FHWA 2006). The direct cost of highway work zone crashes, estimated based on the crash data from 1995 to 1997, was as high as $6.2 billion per year, an average cost of $3,687 per crash (Mohan and Gautam 2002). The recent Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) included a number of provisions addressing highway work zone safety and other work zone–related issues (FHWA 2005). Eliminating work zone crashes, especially severe crashes involving injuries and fatalities, has been a high-priority task for traffic engineers.

Traffic crashes result from human-machine interactions, and human factors play a critical role in the occurrence of crashes. Among all human factors, driver errors have been found to be the major causes of traffic crashes. Studies have shown that 45% to 75% of traffic crashes in the United States were in part caused by driver errors (Wierwille et al. 2002). In Arizona, for example, driver error is estimated to have caused about half of all traffic crashes (Hutabarat, Lam, and Lawrence 2004). In highway work zones, human errors are the major contributing factor as well. Among the various human errors, studies suggest that following too closely, inattentive driving, and misjudging traffic conditions were the most common causes of work zone crashes (Mohan and Gautam 2002; Pigman and Agent 1990).

In Kansas, too, previous crash analyses showed that driver errors were the dominant contributing factors to the severe crashes in highway work zones (Bai and Li 2006; Bai and Li 2007). Understanding driver errors is thus a key step towards work zone safety improvement in Kansas.

In this study, the human factors that have been associated with the severe crashes involving injuries and fatalities in Kansas highway work zones were analyzed. The primary objectives of the study were (1) to investigate the major human factors, especially driver errors, that resulted in severe crashes in Kansas highway work zones and (2) to understand the impacts of human errors on highway work zone safety. The crash data in Kansas highway work zones between 1992 and 2004 were analyzed through various statistical methods, including frequency analyses, chi-square tests, and logistic regression. The results of this in-depth study will help determine the role of human factors in the occurrence of crashes and will provide knowledge for traffic engineers and researchers to develop safety countermeasures that are effective in preventing major driver errors from causing severe crashes in work zones.

DATA COLLECTION

Data for fatal and injury crashes in Kansas highway construction zones between 1992 and 2004 were collected from the Kansas Department of Transportation (KDOT) database. The original data was formatted such that a single crash was frequently described in multiple data rows and crash information was recorded using text. The original data were recompiled to facilitate the computer aided data analyses using SAS software. First, at-fault drivers were identified and their characteristics, along with other crash information, were collected into spreadsheets in which a crash was described in only one data row. Then, for the cases with missing or unclear information, the original accident reports, including detailed crash descriptions in text and sketches, were examined to ensure the data accuracy. The collected crash information contained 3 categories and 17 variables, as listed in Table 1. The observations of these crash variables were assigned unique integers so that the spreadsheet contained only numerical data.

During the study period, Kansas work zones had 157 fatal crashes and 4,443 injury crashes. It would have been extremely time consuming and not statistically meaningful to compile and analyze the entire fatal and injury datasets. Instead, the 157 fatal crashes and a sample of 460 injury crashes were used to save data collection time while maintaining reasonable accuracy in the results of the analysis. In addition to these fatal and injury crashes, a dataset that contained 655 recent crashes in 2003 and 2004, including 29 fatal crashes and 626 injury crashes, was also prepared for more advanced statistical studies of driver errors using chi-square tests and logistic regression.

Table 1. Data categories and variables

No.	Category	Variable
1	Crash variables	Age Gender Crash time Light condition Speed limit Area information No. of cars in collision Vehicle type
2	Driver errors	Inattentive driving Too fast for condition/exceeded speed limit Disregarded traffic signs, signals, or markings Followed too closely
3	Traffic control methods	Flagger/officer Stop sign/signal Flasher No passing zone Center/edge lines

The sample size for injury crashes was determined based on the method introduced by Thompson (2002). Considering that the data would be used for frequency analysis of characteristics, as reflected through the proportions of the different crashes marked by different variable observations, the sample size was determined so that the proportions could be estimated accurately. Based on normal approximation, to obtain a proportion estimator $\hat{p}$ with a probability of at least 1- α that was no farther than *d* (error) from the true population proportion *p*, a corresponding sample size would need to be chosen such that

$$P(|\hat{p} - p| > d) < \alpha \qquad (1)$$

According to Thompson (2002), when there is no estimate of *p* available and the population size *N* is large, the following equation can be used to determine the minimum sample size n_{min}:

$$n_{\min} = \frac{1}{(N-1)/Nn_0 + 1/N} \approx \frac{1}{1/n_0 + 1/N} \qquad (2)$$

where

$$n_0 = \frac{z_{\alpha/2}^2\, p(1-p)}{d^2} = \frac{0.25 z_{\alpha/2}^2}{d^2} \qquad (3)$$

and $z_{\alpha/2}$ is the upper α/2 point of the standard normal distribution.

For multiproportional estimations, Thompson (2002) showed that n_0 was equal to 510 when $\alpha = d = 0.05$ and when the population size N was large. Given the population of 4,443 injury crashes in this study, the minimum sample size needed for frequency analysis at 95% confidence level (an error d less than 5%) was determined using the above equation as follows:

$$n_{\min} \approx \frac{1}{1/n_0 + 1/N} = \frac{1}{1/510 + 1/4443} = 457 \tag{4}$$

The result was rounded to 460.

HUMAN FACTORS IN HIGHWAY WORK ZONES

The human factors involved in severe work zone crashes were examined systematically based on Kansas work zone crash data. The results of an overview of the human factors, such as at-fault driver age and gender as well as driver errors associated with severe work zone crashes, were first presented. The four most frequent driver errors causing work zone crashes were then addressed in detail. These included "inattentive driving," "too fast for condition/ exceeded speed limit," "disregarded traffic signs, signals, or markings," and "followed too closely." The investigation of these driver errors involved various statistical methods such as chi-square tests and logistic regression.

Overview

Based on the injury and fatal crash data in Kansas highway work zones between 1992 and 2004, male drivers were responsible for most of the crashes in highway work zones. Frequency analyses showed that male drivers caused 75% of the fatal crashes and 66% of the injury crashes. Note that male drivers only constituted about 50% of the total licensed drivers in Kansas, according to 2004 statistics (FHWA 2004).

Figure 1 illustrates the percentages of the at-fault drivers in the fatal and injury work zone crashes by age. The age distributions of the Kansas licensed drivers (FHWA 2004) are also included in the figure. Among all age groups, drivers younger than 25 years of age caused the largest proportion of the severe crashes involving fatalities or injuries, followed by drivers aged between 25 and 34 years. In addition, the percentage of the severe crashes caused by young drivers under 25 years of age was significantly higher than their percentage in the total licensed drivers.

When comparing the percent frequencies of fatal crashes to injury crashes, drivers between 15 and 34 years of age caused a higher percentage of injury crashes than fatal crashes. However, drivers aged 35 to 44 years caused the highest percentage (24%) of fatal crashes among all the age groups. This percentage was 9% higher than the injury crashes caused by the same age group and 6% higher than their percentage in the total Kansas licensed driver population. Senior drivers (65 or older) were found to be responsible for a larger proportion of fatal crashes than for injury crashes (18% vs. 8%) in work zones.

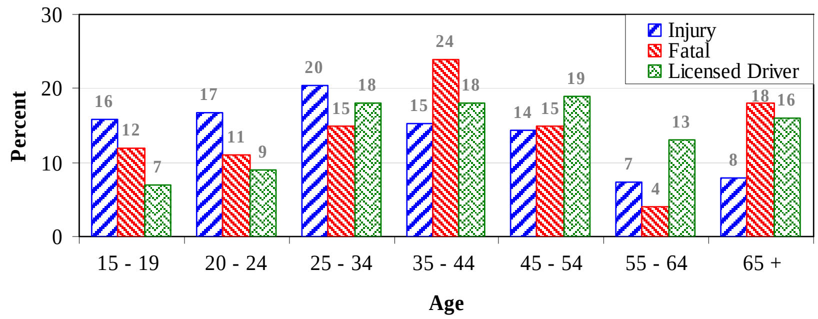

Figure 1. Fatal and injury crash frequencies and Kansas licensed drivers by age (in percent)

Studies have showed that pedestrian factors, environmental factors, and vehicle factors are not significant causes of severe work zone crashes; human errors, on the contrary, contribute to most of the crashes involving fatalities and injuries in work zones (Bai and Li 2006; Bai and Li 2007). Based on the Kansas work zone crash data, "inattentive driving" contributed to more than half of the severe crashes, followed by "too fast for conditions/exceeded speed limit," "followed too closely," and "disregarded traffic signs, signals, or markings." In addition, as exhibited in Figure 2, some driver errors such as "too fast for condition/ exceeded speed limit," "disregarded traffic signs, signals, or markings," and "under influence of alcohol" resulted in notably higher percentages of fatal crashes than injury crashes. Therefore, these driver errors could be factors leading to crashes of higher severity in highway work zones. On the other hand, "followed too closely" caused 14% more injury crashes than fatal crashes (18% vs. 4%).

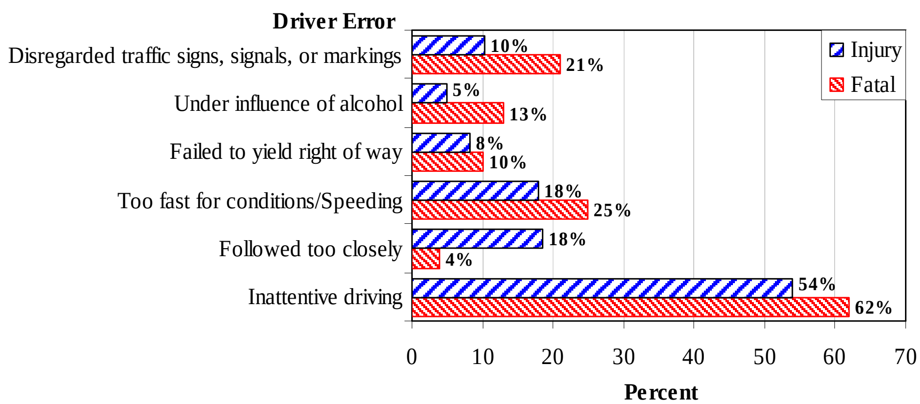

Figure 2. Fatal and injury crash percent frequencies by driver error

Examination of the Most Common Driver Errors in Work Zones

The previous crash overview has indicated that "inattentive driving," "too fast for condition/exceeded speed limit," "disregarded traffic signs, signals, or markings," and "followed too closely" were the most frequent driver errors associated with severe traffic crashes in highway work zones. Further in-depth studies of these major driver errors were conducted to identify their impacts on severe work zone crashes and the temporary traffic control (TTC) methods that may be effective in eliminating the errors. In these studies, chi-square tests were first utilized to determine the interactions of these driver errors with other crash variables; multivariate frequency analyses were then applied to determine the safety impacts of the driver errors together with the interrelated variables. The crash variables that were inputted into the chi-square tests included at-fault driver age and gender, crash time, light condition, speed limit, area information, number of vehicles in collision, and vehicle type. In addition, the effectiveness of the TTC methods (including flagger/officer, stop sign/signal, flasher, no passing zone, and center/edge lines) in preventing these driver errors from causing severe crashes in work zones were evaluated using logistic regression techniques. The severe crashes involving fatalities or injuries that occurred during 2003 and 2004 in Kansas highway work zones were used in these analyses. The results would enable a better understanding of the driver errors and provide knowledge that may lead to safety improvements in highway work zones.

Inattentive Driving

As used in this study, the term "inattentive driving" refers to such driver errors included in the standard State of Kansas motor vehicle accident report as "inattention," fell asleep," "other distraction in or on vehicle," "distraction-cell phone," or "distraction-other electronic devices." Inattentive driving was found to be the most frequent driver error leading to severe crashes in highway work zones. Statistical tests including Pearson chi-square test and likelihood ratio chi-square test indicated that inattentive driving was significantly associated (at a 0.05 level of significance) with the number of vehicles involved in crashes, the speed limit, and area information. Figure 3 shows the distributions of severe work zone crashes, with and without inattentive driving as a contributing factor, over the number of crash vehicles. As seen in the figure, inattentive driving contributed to more than half of the severe work zone crashes involving multiple vehicles, while only about one-third of the single-vehicle crashes had inattentive driving as a causal factor. An explanation might be that inattentive driving could increase the probability of having multiple-vehicle crashes in work zones. Crash frequencies by speed limit and by contribution of inattentive driving showed, as illustrated in Figure 4, that inattentive driving caused larger proportions of severe crashes in work zones with lower speed limits (e.g., work zones with speed limits lower than 41 mph) than in high-speed work zones. In addition, the results of the analysis also suggested that inattentive driving contributed to a larger proportion (88% vs. 83%) of the severe crashes in rural work zones than in urban work zones.

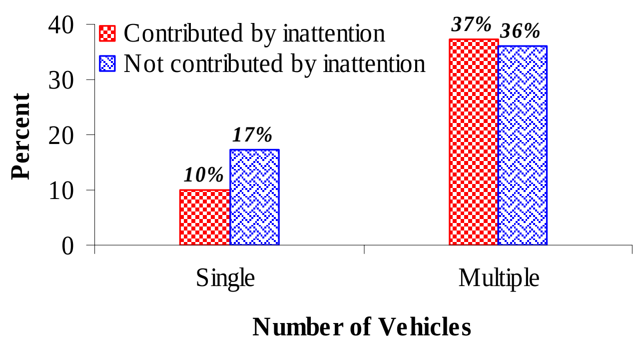

Figure 3. Crash distribution over number of vehicle and involvement of "inattentive driving"

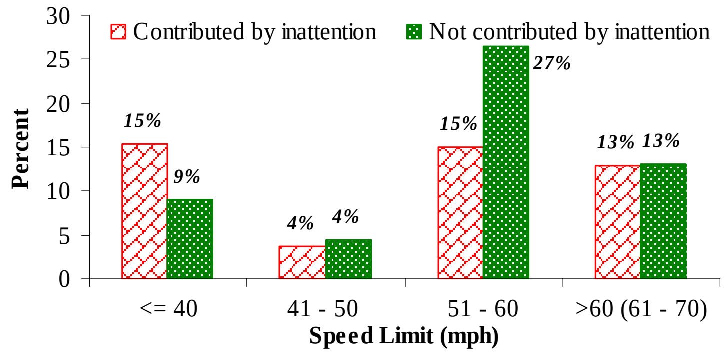

Figure 4. Crash distribution over speed limit and involvement of "inattentive driving"

Using a logistic regression technique, the researchers evaluated the effectiveness of several TTC methods commonly used in work zones to prevent "inattentive driving" from causing severe crashes. These TTC methods included flagger/officer, stop sign/signal, flasher, no passing zone, and center/edge lines. Among these traffic controls, statistical analyses showed that a flagger/officer was effective in reducing inattentive driving in work zones. By denoting $Y = 1$ as the occurrence of a severe crash caused by inattentive driving and X as the presence of flagger control ($X = 1$ as present and $X = 0$ as not present), the following logistic regression model was fitted at a 0.1 level of significance:

$$logit\{Y = 1|X\} = 0.34 - 0.51X \tag{5}$$

The odds ratio between a severe crash caused by inattentive driving with and without flagger control was estimated as 0.60. Therefore, using a flagger could reduce the odds that "inattentive driving" would cause severe crashes by 40% in highway work zones.

Too Fast for Condition/Exceeded Speed Limit

"Too fast for condition/exceeded speed limit" was another common driver error that contributed to a large proportion of the severe crashes involving fatalities or injuries in highway work zones. Chi-square tests indicated that this driver error was statistically related to crash time, speed limit, and area information. Hence, crash frequencies were analyzed for the combinations between the driver error "too fast for condition/exceeded speed limit" and these variables. The frequencies of the severe crashes in work zones in terms of speed limit and involvement of the driver error "too fast for condition/exceeded speed limit" were shown in Figure 5. This driver error contributed to much larger proportions of the crashes in work zones with high speed limits (51 mph to 70 mph) , which may indicate that speeding in high-speed work zones was more likely to cause severe crashes than in low-speed work zones. In addition, frequency analysis results indicated that the driver error "too fast for condition/exceeded speed limit" contributed to a larger proportion (21% vs. 7%) of the crashes in rural work zones than in urban work zones. Based on the analysis of crash time, a much smaller proportion of the crashes during morning peak hours (6:00 a.m. to 10:00 a.m.) were contributed by this driver error.

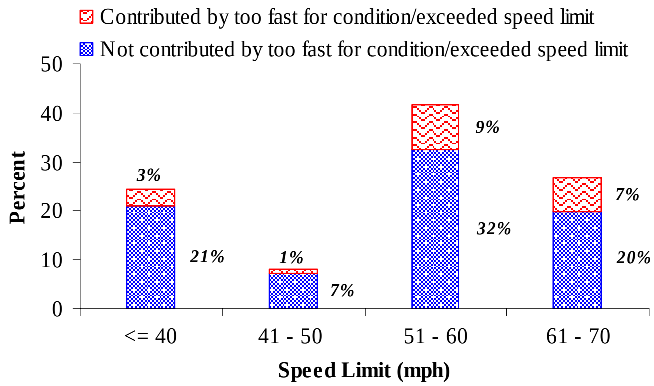

Figure 5. Crash distribution by speed limit and involvement of "too fast for condition/exceeded speed limit"

When studying the effectiveness of TTC methods in preventing severe crashes in work zones caused by "too fast for condition/exceeded speed limit," two traffic control methods, including flagger/officer and pavement center/edge lines, were discovered to have positive impacts on preventing speeding from causing severe crashes in work zones at the 0.10 level of significance. In modeling the effectiveness of using a flagger/officer, the logistic regression model was estimated as follows:

$$logit\{Y = 1|X\} = -1.01 - 0.46X \qquad (6)$$

where $Y = 1$ represents the occurrence of a severe work zone crash caused by the driver error "too fast for condition/ exceeded speed limit" and X denotes the presence of flagger control. According to this model, the odds ratio of a severe crash caused by "too fast for condition/ exceeded speed limit" with vs. without flagger control was 0.63. In another words, using a flagger to direct traffic in a work zone lowered the odds of having a severe crash caused by this driver error by 37%.

Results of logistic regression analysis suggested that having center/edge lines on work zone pavement was also effective in reducing the probability of severe crashes caused by "too fast for condition/exceeded speed limit." The statistical model for the effectiveness of this TTC method is as follows:

$$logit\{Y = 1|X\} = -1.61 - 0.35X \qquad (7)$$

where $Y = 1$ is the occurrence of a severe work zone crash caused by the driver error "too fast for condition/exceeded speed limit" and X is the presence of pavement center/edge lines. The odds ratio of a severe crash caused by this driver error with vs. without center/edge lines was estimated as 0.71. Therefore, having pavement center/edge lines in work zones would reduce the odds of a severe crash caused by this driver error by 29%.

Disregarded Traffic Signs, Signals, or Markings

According to the Pearson and likelihood ratio chi-square tests, the probabilities that the driver error "disregarded traffic signs, signals, or markings" would cause severe crashes in work zones were different with different speed limits. As seen in Figure 6, although the percentage of crashes caused by this driver error were all small in various speed zones, the proportions of crashes caused by this driver error in low-speed zones (speed limits lower than 51 mph) were considerably larger than in work zones with speed limits higher than 50 mph. For example, 12.5% of the crashes (3% out of 24%) in work zones with speed limits lower than 41 mph were caused by "disregarded traffic signs, signals, or markings," while the corresponding percentage in 61 to 70 mph zones was less than 4% (1% out of 26%).

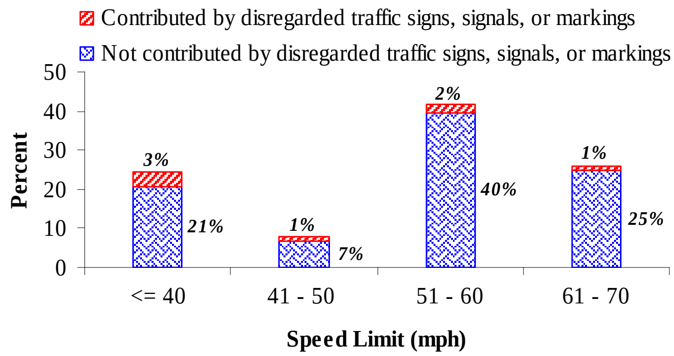

Figure 6. Crash frequencies by speed limit and involvement of "disregarded traffic signs, signals, or markings"

To evaluate the effectiveness of TTC methods in reducing the driver error "disregarded traffic signs, signals, or markings," logistic regression models were fitted by letting $Y = 1$ denote the occurrence of a work zone crash and X denote the presence of the TTC method under evaluation ($X = 1$ as present and $X = 0$ as not present). At the 0.10 level of significance, the flagger/officer and no passing zone controls were found effective in preventing the driver error from causing severe crashes in work zones. For the flagger/officer traffic control, the logistic regression model is as follows:

$$logit\{Y = 1|X\} = -1.78 - 0.77X \quad (8)$$

The odds ratio of a severe work zone crash caused by the driver error "disregarded traffic signs, signals, or markings" when a flagger/officer control was/was not present was estimated at 0.46. For the no passing zone control, the regression model was developed as follows:

$$logit\{Y = 1|X\} = -2.20 - 0.35X \quad (9)$$

The odds ratio of a severe work zone crash caused by this driver error when a no passing zone control was/was not present was 0.71. Based on these results, the flagger/officer and no passing zone controls could reduce the odds of this driving error causing a severe work zone crash by 54% and 29%, respectively.

Followed Too Closely

Pearson and likelihood ratio chi-square tests showed that the driver error "followed too closely" was statistically related to crash variables such as crash time, light condition, vehicle type, speed limit, and area information. Crash frequency analyses in terms of crash time and presence of the driver error "followed too closely" showed that the proportion of the crashes caused by "followed too closely" in daytime work zone crashes was considerably higher than in nighttime crashes (8:00 pm to 6:00 am), as shown in Figure 7. This fact confirms that "followed too closely" was more likely to occur during the daytime, when traffic volume was high. Correspondingly, a much larger proportion of the crashes in good light conditions than in poor light conditions were contributed to by "followed too closely."

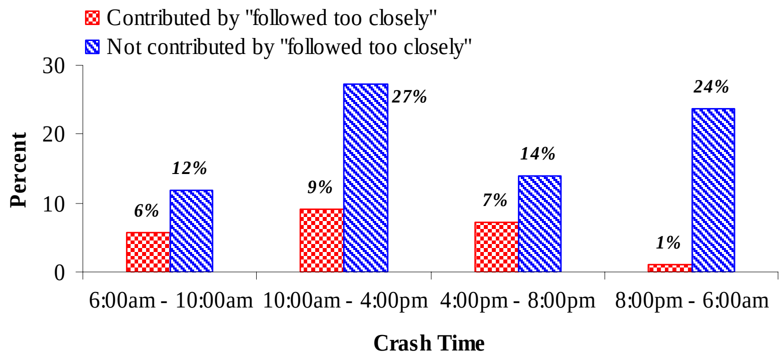

Figure 7. Crash frequencies by crash time and involvement of "followed too closely" driver error

In regard to vehicle type, a much larger proportion of the crashes involving only light duty vehicles than the crashes involving heavier trucks was attributed to the driver error "followed too closely." Regarding the relationship between speed limit and "followed too closely," as shown in Figure 8, a relatively small proportion of the crashes in high-speed zones (e.g., work zones with speed limits between 61and 70 mph) were associated with this driver error. This could be a result of the relatively low traffic density on high-speed roads. Similarly, analyses also showed that a considerably larger proportion of urban work zone crashes than rural work zone crashes were attributed to the "followed too closely" driver error.

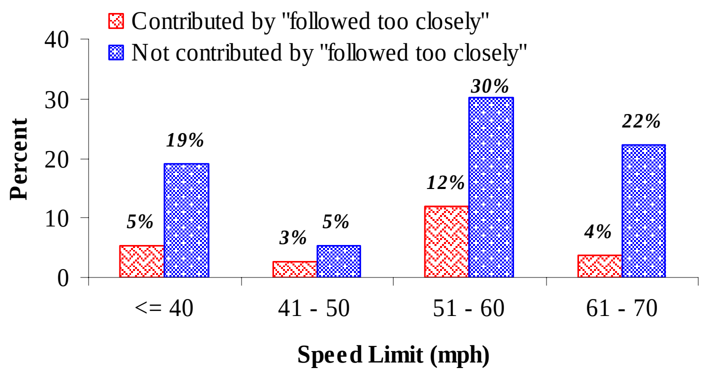

Figure 8. Crash frequencies by speed limit and involvement of "followed too closely" driver error

Based on logistic regression analyses, traffic controls were identified that could affect the probability that "followed too closely" would cause severe crashes. The researchers found that pavement center/edge lines were effective in preventing this driver error from causing severe crashes in highway work zones. The regression model developed for evaluating the effectiveness of center/edge lines is as follows:

$$logit\{Y = 1|X\} = -1.30 - 0.20X \qquad (10)$$

where $Y = 1$ is the occurrence of a severe crash caused by "followed too closely" and X is the presence of the pavement center/edge lines ($X = 1$ for present and $X = 0$ for not present). The odds ratio of a severe crash caused by "followed too closely" when pavement center/edge lines were/were not present was estimated at 0.81. This indicates that having center/edge lines in work zones may reduce the odds of severe crashes caused by "followed too closely" by 19%.

The stop sign/signal control, on the other hand, was found to be countereffective in preventing the "followed too closely" driver error. The logistic regression model for stop sign/signal control (X represents the presence of stop sign/signal) was fitted as follows:

$$logit\{Y = 1|X\} = - 2.38 + 1.26X \tag{11}$$

The odds ratio of a severe crash caused by the driver error "followed too closely" when stop sign/signal control was/was not present was estimated at 3.53. Therefore, statistically speaking, work zones having stop sign/signal control may have roughly 3.5 times the chances of experiencing a severe crash caused by the "followed too closely" driver error than work zones without stop signs/signals. Therefore, the installation of stop signs/signals in work zones should be carefully planned, and supplemental traffic control devices should be used to avoid crashes caused by the "followed too closely" driver error.

CONCLUSIONS AND RECOMMENDATIONS

Work zone safety has been a focus of research for many years, and improving safety in highway work zones is a high-priority task for traffic engineers. Because a significant proportion of work zone crashes are attributed to driver errors, preventing driver errors from causing severe crashes in work zones is a top-priority objective. Based on the severe crashes involving injuries or fatalities in Kansas highway work zones from 1992 to 2004, the human factors, including at-fault driver characteristics and the driver errors frequently leading to severe crashes, were systematically examined. The effectiveness of several TTC methods in preventing driver errors from causing crashes in work zones was also evaluated using the logistic regression method. This knowledge regarding the role of human factors in work zone crashes will lead to a better understanding of the influence of human factors and will enable the development of safety improvements in work zones.

Study results showed that most of the work zone crashes involving fatalities or injuries were caused by male drivers. Drivers younger than 25 years of age frequently caused severe crashes in work zones. In terms of fatal crashes only, the drivers between 35 and 44 years of age and senior drivers older than 64 years had the highest crash frequencies. In addition, driver errors were the most common causal factors of the severe crashes in work zones, and "inattentive driving" was the most prominent, followed by "too fast for conditions/ exceeded speed limit," "followed too closely," and "disregarded traffic signs, signals, or markings." The authors did not find statistical relationships between the age and gender of at-fault drivers and these driver errors. Further examinations of the most common driver errors showed the following:

1. "Inattentive driving" caused proportionally more multivehicle crashes than single-vehicle crashes in work zones, and this error is most likely to cause severe crashes in work zones with speed limits no higher than 40 mph. A traffic control study showed that using a flagger/officer to direct traffic in a work zone could effectively reduce the odds of severe crashes caused by "inattentive driving."
2. "Too fast for condition/exceeded speed limit" tended to cause proportionally more severe crashes in high-speed (51 to 70 mph) work zones and rural work zones. In addition, using flagger/officer controls or having center/edge lines in work zones may considerably lower the odds of severe crashes caused by this driver error.
3. "Disregarded traffic signs, signals, or markings" caused a larger proportion of severe crashes in work zones with speed limits lower than 51 mph than in work zones with higher speed limits. Logistic regression analyses indicated that flagger/officer controls and no passing zone controls in work zones could effectively prevent this driver error from resulting in severe crashes involving fatalities or injuries.

4. The “followed too closely” driver error caused larger proportions of severe crashes during daytime hours and in work zones with speed limits between 41 and 60 mph. Based on the crash data, the authors found that work zone center/edge lines may lower the odds of severe crashes caused by this driver error by 19%. On the contrary, having stop signs/signals in work zones would dramatically increase the odds of a severe crash caused by this driver error.

Based on these results, potential safety improvements in work zones are recommended. First, to reduce inattentive driving, more effective warning methods need to be developed in work zones to alert the inattentive drivers of the upcoming work zone conditions. Such approaches may include the installation of temporary rumble strips or other raised pavement markings that can effectively alert vehicle passengers by physical vibration. Second, the high frequency of speeding-caused crashes indicates a need for the development of more effective and more strictly enforced speed control strategies in highway work zones. Third, traffic control strategies should be developed to effectively control and enforce safe headways between consecutive vehicles, especially when the platoon contains heavy vehicles. Finally, and importantly, public education programs should be launched to raise the awareness of highway work zone hazards, especially for the young drivers who have the highest probability of causing severe crashes in work zones.

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The Effect of Loading Level and Rate in the Indirect Tensile Test

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ABSTRACT

This paper investigates the effect of loading level and loading rate on the damage development and strength on the asphalt mixtures tested at three low temperatures. The current AASHTO specification for asphalt mixture low-temperature characterization consists of two tests: the indirect tensile creep (ITC) test and the indirect tensile strength (ITS). In the ITC, the load was applied until the horizontal deformation on one face reached 0.00125 mm to 0.019 mm and then was held constant for 1,000 seconds. In the ITS, a constant loading rate was maintained until failure. In this study, different loading levels were used in the creep tests: 25% to 60% of the peak load observed in the strength test. Three different loading rates were employed to perform the ITS test at all tested temperatures. The experimental results show that test temperature has a significant effect on the behavior of the material. Data analysis from the creep test indicates that significant damage develops when the loading level during creep reaches more than 55% of the material strength. Analysis of the strength test data illustrates that the loading rate significantly affects the response of the material. For each test temperature, asphalt mixtures have lower peak loads and larger displacements when tested at a lower rate. The measured tensile strength value significantly increases with the increase of the loading rate.

Key words: asphalt—creep—indirect tension—mixture—strength

INTRODUCTION

Low-temperature cracking is considered one of the primary distress modes of asphalt pavements built in the northern United States and Canada. These cracks result in accelerating the deterioration of the pavement, thus causing the pavement to require maintenance sooner than anticipated. Thus, the evaluation of the fracture resistance of asphalt mixtures is of interest to owner/agencies seeking better performing pavements in these northern climates.

Numerous research efforts based on empirical and theoretical methods have been conducted in the past few decades to better understand this distress and to select materials with improved fracture resistance. The indirect tensile test (IDT), which was identified and further developed during the Strategic Highway Research Program (SHRP), exists as an American Association of State Highway and Transportation Officials (AASHTO) specification and is commonly used to evaluate the low-temperature properties of asphalt mixtures (AASHTO 2002). In the AASHTO specification, a cylindrical specimen 150 mm in diameter by 38–50 mm in thickness is loaded in compression across a diametral plane, similar to the splitting tension test, also known as the Brazilian test. Both a creep compliance test and a tensile strength test are performed and used in the thermal cracking simulation software develop under the SHRP, called TCMODEL (Hiltunen, Roque, and Buttlar 1995), which was adopted for use in the new Mechanistic-Empirical Pavement Design Guide developed under National Cooperative Highway Research Program (NCHRP) 1-37A (ARA 2004). In the creep test, a load level that produces a horizontal deformation between 0.00125 mm and 0.019 mm is held constant for 1,000 seconds. The horizontal and vertical deformations are recorded during the loading process and are used to calculate creep compliance and stiffness as a function of time. The strength test determines the tensile strength of a specimen by loading the specimen at a constant rate of 12.5 mm/minute until failure. The specimen dimensions and peak load are then used to calculate the failure strength.

It is well documented that asphalt mixtures exhibit complex temperature-sensitive behavior. The response of the material to a given load depends significantly on test temperature and loading condition. This paper investigates the effect of loading level and loading rate on the damage development and strength of an asphalt mixture tested at three low temperatures. In this study, different loading levels were used in the creep tests: 5% to 55% of the peak load observed in the strength test. Three different loading rates were employed to perform the strength test at all tested temperatures.

EXPERIMENTAL PROCEDURE

The asphalt mixture used in this study was prepared using the Superpave design procedure outlined in SP-2 (AI 1996). One asphalt binder with a performance grade 58 -34 consisting of a styrene-butadiene-styrene modifier was used. Granite aggregate was selected to prepare the mixture, and the nominal maximum aggregate size was 12.5 mm. Cylindrical specimens 150 mm in diameter by 170 mm in height were compacted using a Superpave gyratory compactor. A 4% target air void content was achieved after compaction. The compacted samples were then cut into three slices 50 mm in thickness, with the upper and lower 5 mm of each original specimen discarded.

The indirect tensile test setup utilized in this research is shown in Figure 1. A sample 150 mm in diameter by 50 mm in height was loaded in static compression across its diametral plane. Different load levels were applied in the creep test to investigate the effect of load levels on the development of microdamage during the creep test. A constant loading rate of 10 kN/s was used at the beginning of the creep test. Following the creep test, an indirect tensile strength test was performed at the same temperature. Three stroke rates, 1 mm/minute, 3 mm/minute, and 12.5 mm/minute, were applied during the strength test until failure, in order to evaluate the influence of the loading rate on the strength.

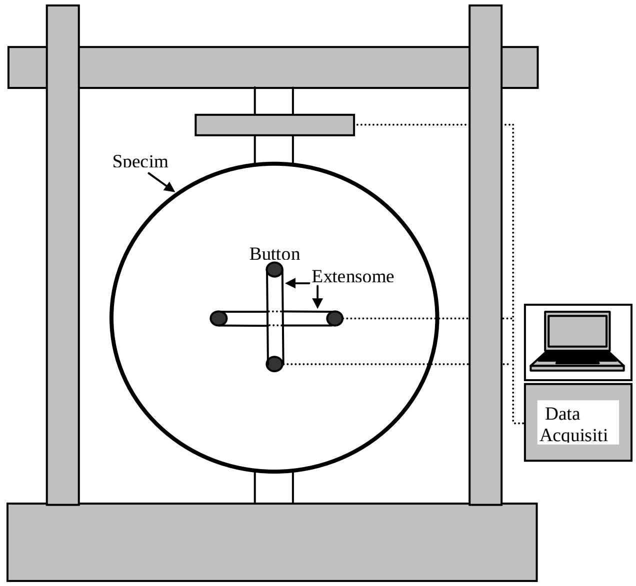

Figure 1. Schematic of experimental setup

Tests were performed with an MTS servohydraulic testing system. The TestStar IIs' control system was used to set up and perform the tests and to collect the data. The software package MultiPurpose TestWare was used to custom-design the tests and collect the data. All tests were performed inside an environmental chamber for temperature control. Liquid nitrogen tanks were used to obtain the required low temperature. The temperature was controlled by an MTS temperature controller and verified using an independent platinum (RTD) thermometer.

Three test temperatures, -12°C, -24°C, and -36°C, were selected based on the lower limit of the asphalt binder's performance grade. The specimen used for the creep test was also used for the strength test, 30 minutes after the creep test. A total of 11 specimens were tested. Table 1 shows the loading level and loading rate for each specimen.

Table 1. Creep load level and strength test load rate

		Creep Test		Strength Test	
Specimen ID	Test Temp (°C)	Load Level (kN)	Creep Load/ (Failure Load at 12.5mm/min) (%)	Stroke Rate (mm/min)	Failure Load (kN)
1	-12	10	17.4	1	30.2
2	-12	3	5.2	3	35.9
3	-12	15	26.0	12.5	57.6
4	-24	12	16.5	1	50.3
5	-24	30	41.3	3	60.1
6	-24	40	55.1	12.5	56.3
6-1	-24	N/A	N/A	12.5	72.6
7	-36	25	30.2	1	65.6
8	-36	30	36.2	3	71.2
9	-36	40	48.2	12.5	82.9
10	-36	55	66.3	N/A	N/A

DISCUSSION OF RESULTS

As shown in Table 1, three different load levels were applied for the two higher test temperatures, while four load levels were applied for the lowest temperature. The tensile strength test was conducted on all specimens 30 minutes after the completion of creep testing. The only exception is Specimen 10, which was broken when a high load level of 55 kN was applied for about 30 seconds.

Indirect Tensile Creep Test

The load, load point displacement, and horizontal and vertical displacements at the center of the tensile region were recorded over the loading process during the creep test. Both the horizontal displacement and vertical displacement were found to have a similar trend with load time for all test temperatures. Figures 2 through 5 show the horizontal displacement as a function of time for all three temperatures. The experimental data shown in these figures illustrate that all applied creep load levels meet the AASHTO specification to produce an instant horizontal displacement between 0.00125 mm and 0.019 mm. Specimen 3, which was loaded at 15 kN and had an instant horizontal displacement of 0.023 mm, is the only exception.

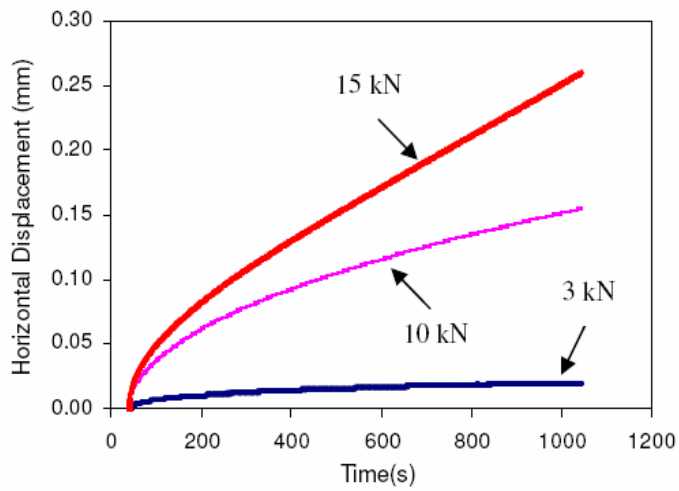

Figure 2. Horizontal displacement for the creep test at -12 °C

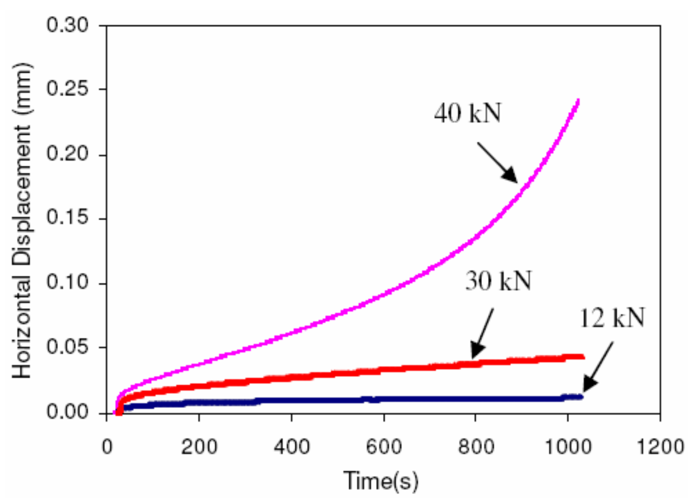

Figure 3. Horizontal displacement for the creep test at -24 °C

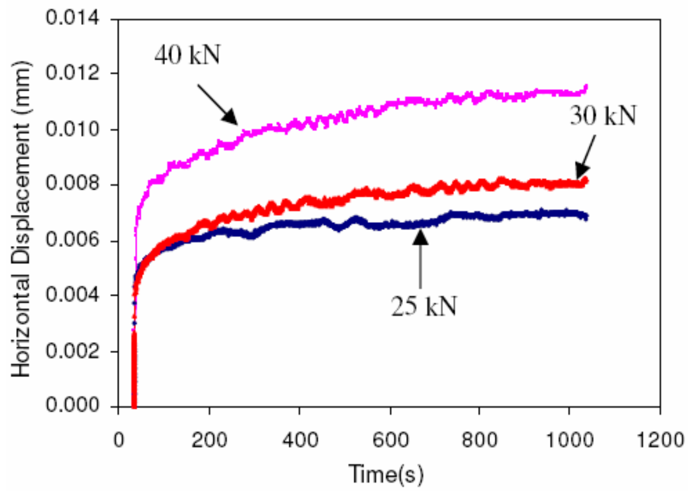

Figure 4. Horizontal displacement for the creep test at -36 °C

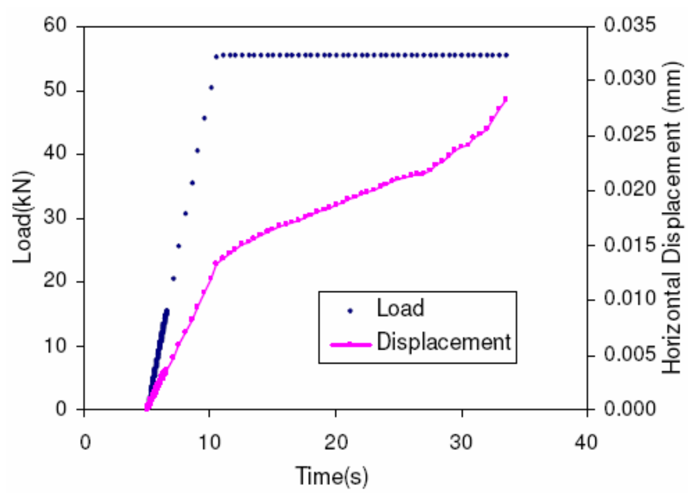

Figure 5. Horizontal displacement for the creep test with 55 kN at -36 °C

From the mechanical response for all three temperatures during the creep tests, one can see that under a similar load level the asphalt mixture is more brittle and has a lower displacement potential at lower temperatures. The displacement curves under different temperatures follow a similar pattern. A higher creep load level always leads to a higher displacement at each test temperature, and the displacement accumulates with time. For all tested specimens except Specimen 6, to which was applied a relatively high load that is about 55% of the peak strength test load, the slope of the horizontal displacement curve was found to decrease with loading time. For Specimen 6, the slope of the displacement curve increased with loading time after the initial decrease with the loading being applied. This is most likely due to microdamage development within the material at the high load level. Figure 5 shows the horizontal displacement for Specimen 10, which failed after it was loaded for about 30 seconds with a creep load level of 55 kN at -36 °C. The displacement curve from Specimen 6 had a very similar shape when the specimen was loaded to failure. This indicates that utilizing 55% of the strength creep load resulted in significant damage development within the material.

Indirect Tensile Strength Test

The mechanical responses for all three temperatures during the strength tests are shown in Figures 6 through 8. At the highest temperature (-12°C), the asphalt mixture is more ductile and has a lower peak load and a larger displacement potential. At the lowest temperature (-36°C), the material is brittle and has a higher peak load but a small deformation ability. At -24°C, the mixture exhibits an intermediate response.

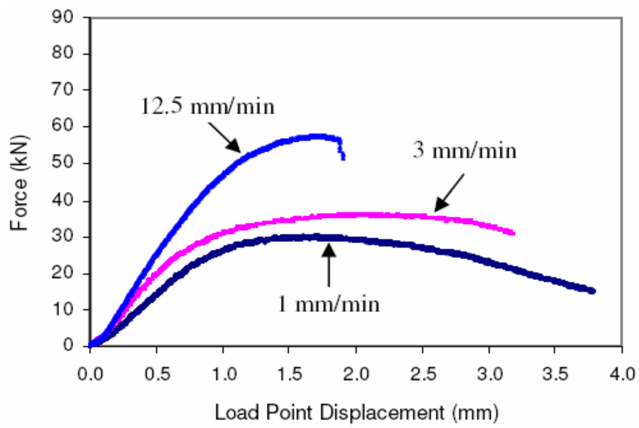

Figure 6. Force and displacement for strength test at -12 °C

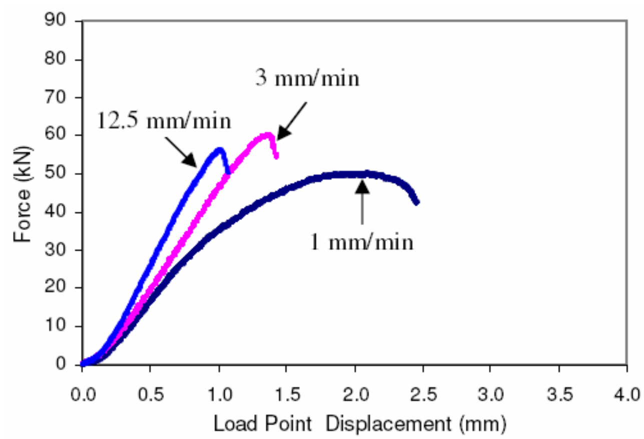

Figure 7. Force and displacement for strength test at -24 °C

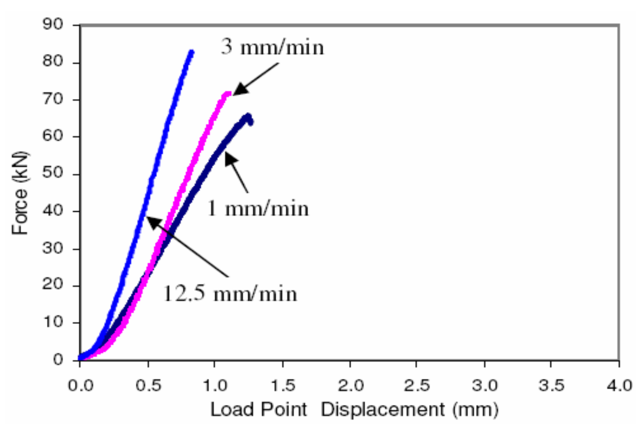

Figure 8. Force and displacement for strength test at -36 °C

It is well documented that an asphalt mixture exhibits complex temperature-sensitive behavior at room and higher temperatures, and it is generally believed to present a linear elastic behavior at low temperatures, under which the mechanical response of the loaded material is treated as independent of the loading rate. The experimental data obtained in this study show that the loading rate has a significant influence on the response of the material at each test temperature. A higher loading rate always generates a higher failure load and a smaller displacement at failure. The only exception is the experimental data for Specimen 6, which was tested at a rate of 12.5 mm/minute at -24°C. The failure peak load from this specimen is obviously smaller than that of the specimen tested at 3 mm/minute. This is most likely due to the significant damage developed during the creep test, as shown in the creep test analysis. An additional specimen, 6-1, was procured and tested at a loading rate of 12.5 mm/minute at -24°C to obtain the true strength failure peak load. The experimental data from this extra specimen was used as the strength value to calculate the percentage of the creep load level shown in the Table 1.

Figure 9 shows the failure peak load as a function of loading rate for all three tested temperatures. It should be noted that the data point at -24°C and 12.5 mm/minute is from Specimen 6-1. The experimental results show a significant effect of the loading rate on the failure peak load as the failure peak load increases with the increase of the loading rate for all test temperatures. However, it was found that the strength–load rate curve gets flatter with a decrease in the test temperature, which indicates that the influence of loading rate on material strength is diluted with the decrease in temperature. In other words, the loading rate has more of an effect on the measurement of material strength at a higher temperature than at a lower temperature. Test results also confirm the significance of temperature on the material tensile strength, since tensile strength increased as temperature decreased.

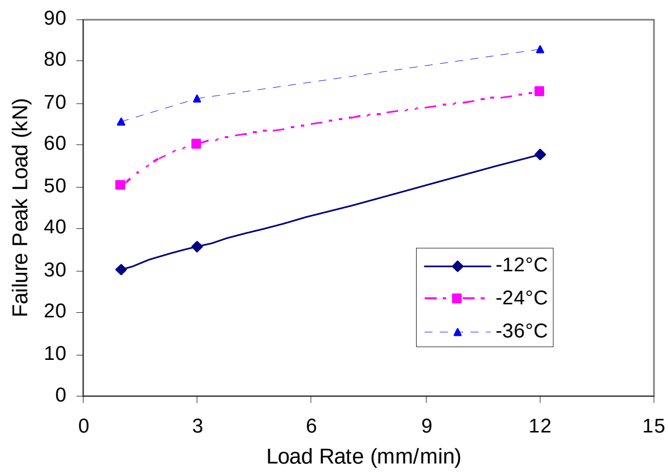

Figure 9. Failure peak load and load rate

CONCLUSIONS

This paper investigated the effect of loading level and loading rate on damage development and the strength of the asphalt mixtures tested at three low temperatures. Indirect tensile creep tests with different loading levels and strength tests with three different loading rates were performed using three different test temperatures for one asphalt mixture.

The experimental data show that test temperature strongly affects the behavior of the asphalt mixture. Specifically, the material exhibits more brittle behavior with a decrease in test temperature. The creep load level was found to significantly influence the response of the material. A higher load level always generated more displacement for all test temperatures. The loading rate was found to have significant effect on the measured tensile strength or peak load at failure, even at a relatively low temperature. Specifically, the measured tensile strength increases with the increase of the loading rate for all test temperatures. However, this effect was found to be diluted with the decrease in test temperature.

ACKNOWLEDGMENTS

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Speed Limit–Related Issues on Gravel Roads

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ABSTRACT

Speed limits on gravel roads in Kansas are regulated at 55 mph, as per the Kansas Statutes. In some instances, this regulatory speed limit has been altered on some gravel roads in several counties. These changes were expected to provide a safer driving and living environment for gravel road users and rural residents. However, the effectiveness of such changes still needs to be evaluated so that future recommendations can be made as feasible as possible. With the lack of detailed literature on this subject, studies need to be developed to find out whether the 55 mph speed limit is appropriate for current conditions on gravel roads.

The methodology in this study is built upon three facets. The first is field studies that were performed at a number of locations on gravel roads. Actual speed data were analyzed to obtain percentile speeds and other relevant measures based on different categories of roads and weather conditions. The second part is a questionnaire survey, which was performed to solicit opinions from county road agencies. A list of road characteristics was rated by respondents to determine how important these characteristics would be in setting speed limits on gravel roads. The third part involved a statistical analysis of crash data related to gravel roads in Kansas during recent years. Most variables were found to be significantly interrelated with speed limits and had an impact on crash severity. The findings of this research are useful for county engineers for understanding the nature of speed limit–related issues on gravel roads.

Key words: gravel roads—rural roads—speed limit

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INTRODUCTION

There are over 1.4 million miles of unpaved roads in the United States, which accounts for 35.3% of the total road network and carries around 12.4% of the total annual vehicle miles traveled (FHWA 2005). In 2005, a total of 39,189 fatal crashes occurred and 43,443 people died in traffic crashes in the United States; 536 fatal crashes occurred on gravel roads (1.4% of total), and as a result 594 persons died (1.4% of total) (USDOT 2007). In the state of Kansas, there are over 98,000 miles of gravel roads, accounting for about 72.5% of the total state mileage, which is a notable proportion compared to the national average (FHWA 2005). In the year 2005, there were 384 fatal crashes on all types of roads in Kansas, 35 of which (9.1%) occurred on gravel roads, which was more than six times the national level (1.4%). In Kansas, altogether 428 people died as a result of motor vehicle crashes, and 38 (8.9%) of those died on gravel roads, which was also a significant percent compared to the national level (USDOT 2007).

In the highway classification used by the Kansas Department of Transportation (KDOT), five classes (A through E) are classified. Gravel roads fall into the fifth class, "E," and are classified as "E-2" by their surface type, which is considered inferior to paved roads (H-2) and bituminous roads (G-1) and superior to graded and drained roads (C), unimproved roads (B), and primitive roads (A) (Russell et al. 1996). A 55 mph blanket speed limit (BSL) was implemented for all gravel roads by the Kansas Statutes, and some lower speed limits were regulated on a number of special sections as needed. In 28 states, 55 mph is accepted as a regulatory speed limit for gravel roads (USDOT 2001). Table 1 lists those states that use a different BSL than 55 mph on gravel roads.

Table 1. List of the states using a different BSL from 55 mph on gravel roads

BSL	States
35 mph	Alabama, Georgia, and Virginia
40 mph	Massachusetts, South Carolina
45 mph	Maine
50 mph	Delaware, Iowa (between sunset and sunrise), Maryland, Nebraska, Rhode Island (45mph during the nighttime), Vermont, and Washington
60 mph	Arkansas (50 mph for trucks), Texas (55 mph during the nighttime)
65 mph	Alaska, Arizona, Minnesota (during the daytime), Mississippi (55mph for trucks or truck-trailers), Tennessee, Wyoming
70 mph	Montana (65 mph during the nighttime)
75 mph	Nevada, New Mexico

Source: USDOT, National Highway Traffic Safety Administration, 2001.

Improvement of traffic safety on gravel roads is a concern of many county engineers. The effectiveness of using the blanket speed limit vs. speed zones needs to be studied. The objective of this study is to find out how the 55 mph BSL is being complied with by road users, how county engineers think about speed limit–related issues on gravel roads, and how speed limit is related to crash severity on gravel roads.

METHODOLOGY

Three steps were carried out in this study:

1. Field speed study
2. Questionnaire survey
3. Crash analysis

First, vehicle speeds were observed and recorded in special traffic counting devices, and then the data were analyzed to obtain important percentile speeds and speed distributions according to weather and road conditions. Second, a survey was administered to around 105 counties in Kansas to collect opinions and comments from county engineers. Third, the Kansas Accident Recording System (KARS) was used to perform a statistical analysis of crashes on gravel roads. The contingency table test method was used to analyze the crashes during the 2003–2005 time period, with the expectation of finding out whether speed limits have an impact on the severity of crashes on gravel roads.

FIELD SPEED STUDY

The speed studies were conducted on gravel roads in Riley County, Kansas. The county map categorizes gravel roads into county roads and township roads. County roads are graded and maintained by contractors of the Riley County Highway Department, and township roads are maintained by township road personnel. County roads were observed to have wider roadways, better graded surfaces, and drainage than township roads.

Speed Data Collection

The principle in selecting study sites was that the sites should be under general conditions and distant enough from special sections like curves, bridges, or intersections, because speeds are easily influenced by external environments that cause drivers to change their speeds to keep safe. The speed study was performed on 21 locations, 10 of which are on county roads and the other 11 of which are on township roads. Figure 1 shows the road locations studied.

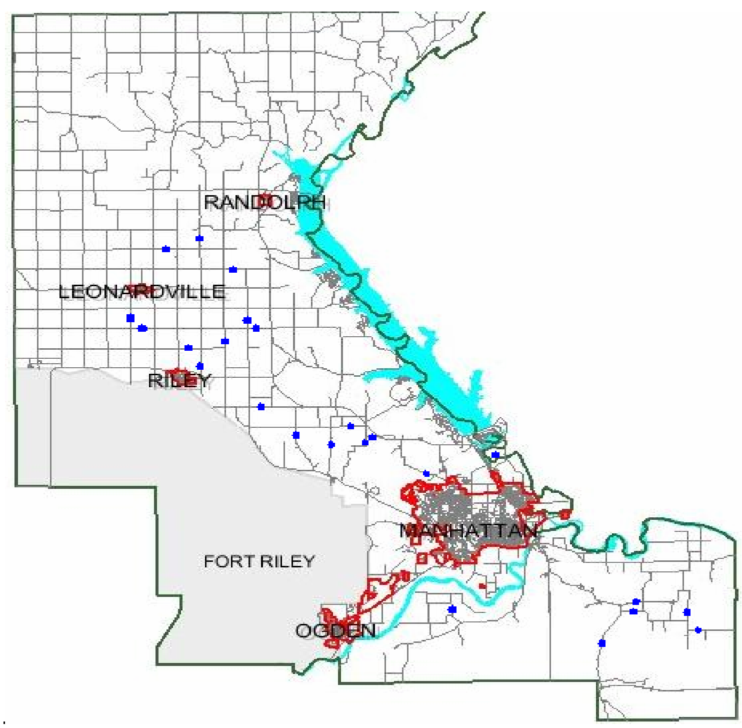

Figure 1. Field speed study locations in Riley County, KS

Two sets of traffic counters were used for speed data collection. Each set consisted of a traffic counter, two road tubes, and accessories, as shown in Figure 2. When an axle passes over the tubes, the air pulses are conveyed into the counter and two time stamps are recorded as raw data. The spacing of the two tubes was eight feet. The raw stamp time data was analyzed with specific analysis software provided by the traffic counter manufacturer. Speed information, together with other data like volume and average daily traffic (ADT), was turned out in the output. The counting devices worked quite successfully on gravel roads, especially on very low-volume roads, since the devices can work in the field for a long period without attendance. The collections were performed at least one week at each location. This was due to the consideration that weekdays might have different ADT, speeds, and vehicle classifications than weekends. Sometimes data collection was interrupted by large vehicles, especially farm equipment, which can easily damage or cut the tubes while passing over them.

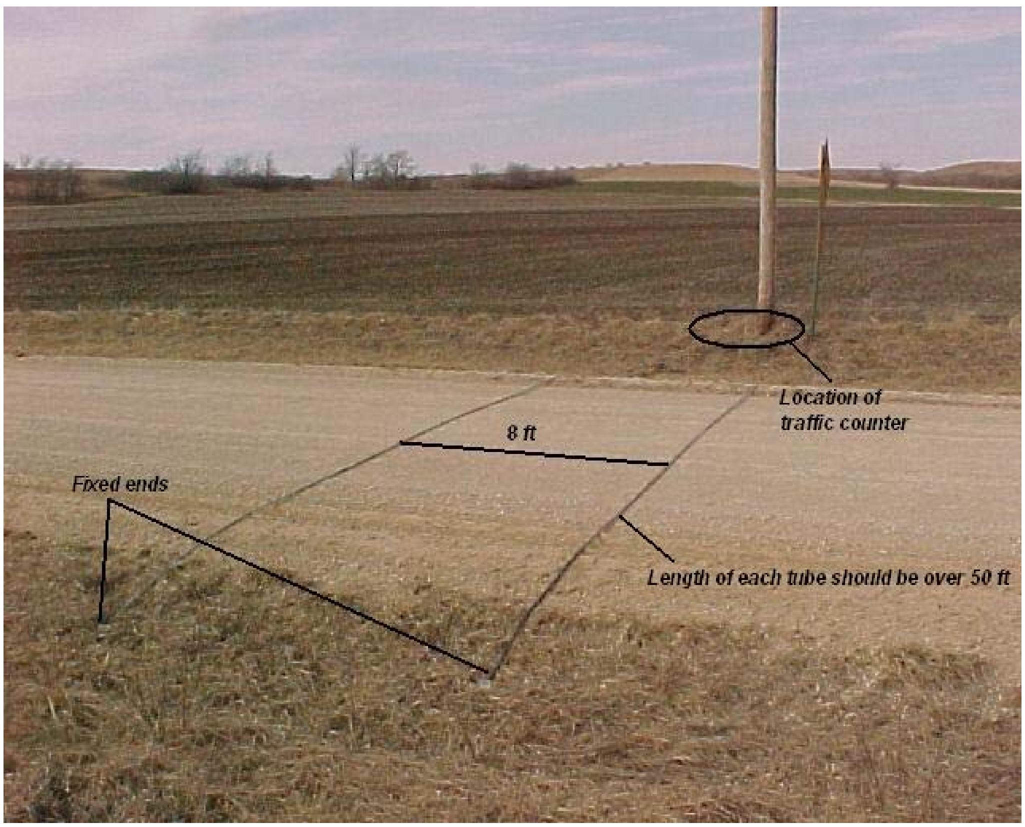

Figure 2. Installation of speed measuring devices on gravel roads

Speed Data Analysis

A total of 7,175 vehicles were observed in this study, of which 3,787 (52.8%) were measured on county roads and 3,388 (47.2%) on township roads. Table 2 summarizes the speed data collected at the study locations. The width of these gravel roads varied from 16 to 26 ft., and ADT ranged from 24 to 179 vehicles per day. Heavy vehicles accounted for a fairly large proportion of traffic on some gravel roads, ranging from 4.7% to 45.8%. The 85th percentile speeds ranged from 35 to 58 mph, and mean speeds ranged from 29 to 49 mph.

Table 2. Summary of Speed Studies on Gravel Roads in Riley County, Kansas

Road Name	Road Type[1]	Road Width (ft)	Total Volume Observed (vehs)	Heavy Vehicle (%)	ADT (vehs/ day)	Mean Speed (mph)	Pace (mph)	% in Pace	85th Speed (mph)	% of Vehicles > 55 mph
W 32nd	C	24	201	45.8	72	36	26-35	55.6	46	0
Deep Creek	C	26	360	37.8	52	49	41-50	48.6	58	23.4
Daniels	C	24	481	20.9	69	44	41-50	45.2	52	5.2
Pillsbury Crossing	C	24	484	4.7	95	36	31-40	54.3	44	0.5
Tabor Valley #1	C	24	340	19	38	43	41-50	45	53	10.3
Tabor Valley #2	C	24	256	15.8	37	43	39-48	47.4	50	5.2
Fairview Church #1	C	24	332	19	55	37	36-45	43.8	49	4.1
Fairview Church #2	C	24	165	18.2	24	39	31-40	47.7	49	9.1
Alembic	C	24	646	15.8	46	44	41-50	40.3	53	9.3
Walsburg	C	24	522	19.3	67	46	46-55	37.9	57	18.8
Marlatt	T	24	142	18.6	47	38	36-45	57.6	45	0
N 60th	T	22	220	19.4	37	41	34-43	40.7	50	2.4
LK&W	T	20	138	11.9	20	37	31-40	53	44	0
N 52nd	T	20	359	35.2	91	34	31-40	38.2	44	0
Rocky Ford	T	22	529	10.7	179	29	21-30	51.2	35	0.5
Kitten Creek	T	22	160	7.1	34	34	31-40	50	40	0
Silver Creek	T	20	121	16.2	25	40	31-40	52	48	3.1
W 59th	T	22	635	22.5	103	35	31-40	47.4	43	1.3
N 48th	T	20	600	19.2	98	35	31-40	53.1	42	0
Union	T	20	271	10.5	46	36	31-40	43.5	45	0
Homestead	T	18	213	21.8	46	37	28-37	34.7	47	4

Note: 1. C denotes county roads and T denotes township roads.

County Roads vs. Township Roads

The comparison of average speed data between county and township roads is presented in Table 3. On average, county roads were 3.4 ft. wider than township roads and had relatively smaller ADT than township ones. The average 85th percentile speed on county roads was 51.1 mph, which was 16.4% higher than that on township roads (43.9 mph). The percent of vehicles traveling over 55 mph was 9.2% on county roads, which is 0.8% higher than that on township roads (8.4%). The average mean speed and pace on county roads were, respectively, 5.7 and 6 mph higher than on township roads. The percent in pace on county roads was 2% less than on township roads. The percent of heavy vehicles was also 1.2% higher on county roads than on township ones.

Table 3. Speed data based on road types

Road type	Road Width (ft)	Total Volume Observed (vehs)	Heavy Vehicle (%)	ADT (vehs/ day)	Mean Speed (mph)	Pace (mph)	% in Pace	85th Speed (mph)	% of Vehicles > 55 mph
County	24.2	3,787	19.9	55.5	41.7	37-46	45.5	51.1	9.2
Township	20.8	3,388	18.7	66	36	31-40	47.4	43.9	8.4
Combined	22.4	7,175	19.3	61	38.7	34-43	46.4	47.3	5.2

Bad Weather vs. Normal Condition

Speed data were also analyzed for different weather conditions. Four types of weather conditions were considered: snow (including snowpacked surface), rain, fog, and clear. A total of 373 vehicles were observed under bad weather conditions, of which 201 vehicles were in snow or on a snowpacked surface, 164 were in rain, and 8 were in fog. The speed data is presented in Table 4. It is obvious that weather has an influence on the traffic on gravel roads. All the speed values under bad weather conditions are lower than those under clear conditions. No vehicle exceeded 55 mph under bad weather conditions. The 85th percentile speed and mean speed in snow were slightly higher than those in rain, while the percent of traffic in pace in snow was larger than that in rain.

Table 4. Speed data under different weather conditions

Weather	Mean Speed (mph)	Pace (mph)	% in Pace	85th Speed (mph)	% of Vehicles > 55 mph
Snow (snowpacked)	36	26-35	55.6	46	0
Rain	33	31-40	38.6	43	0
Fog*	35	N/A	N/A	N/A	0
Clear	38.7	34-43	46.4	47.3	5.2

* Only mean speed was computed due to the small number of observations under fog condition.

Moreover, speed data was compared between weekday and weekend. However, no meaningful difference was found.

COUNTY SURVEY

A questionnaire survey was developed specially for county engineers to solicit their opinions and comments on speed limit–related issues on gravel roads. A total of 105 counties were contacted, 59 (56.2%) counties have returned their responses to date, and more responses are hoped to be received in the following months. Questions in the survey covered the following topics:

- Basic information of gravel roads, including mileage, proportion by deterioration, maintenance frequency, funding, gravel type, and source
- Issues of speed limits on gravel roads, such as whether and how speed limits are posted, criteria and manners of setting speed limits, and public complaints received
- Opinions on the acceptability of blanket speed limits and speed zones

- Rank of the importance of thirteen factors that are likely to have an impact on setting speed limits on gravel roads

Most of the responses included sufficient input, and 25 of them provided valuable additional comments. Most of the questions were multiple choice and thus can be quantified. Table 5 is a summary of these responses. Questions 1 to 5 concern specific information about gravel roads in each county and therefore are not presented here. Questions 7 to 12 consist of information regarding how counties are implementing speed limits and their opinions toward setting speed limits on gravel roads. Question 13 solicits supplementary comments from those positive respondents.

Table 5. Summary of answers to county survey questions (number responding: 59, or 56.2%)

Questions	Answers	Number of Responses	%
Question #6		59	100
Are there speed limits posted on special sections of gravel roads?	Yes	30	50.8
	No	29	49.2
Question #7		59	100
Are there speed limits posted on general sections of gravel roads?	Yes	18	30.5
	No	41	69.5
Question #8		58	98.3
Criteria used on setting speed limits? (multiple choices)	1. Statutory regulations/Blanket speed limit	28	48.3
	2. Engineering study	24	41.4
	3. Professional judgment	10	17.2
	4. Public hearing	1	1.7
	5. Public survey	1	1.7
	6. Other	10	17.2
Question #9		55	93.2
How are speed limits adopted?	1. Applied to all the gravel roads	26	47.3
	2. In special speed zones	20	36.4
	3. Other	12	21.8
Question #10		59	100
Ever received any public complaints on gravel roads?	Yes	59	100
	No	0	0
What kinds of complaints have been received? (multiple choices)	1. Poor road conditions	52	88.1
	2. Dust Pollution	43	72.9
	3. Vehicle speeding	37	62.7
	4. High speeds	30	54.5
	5. Narrow width	28	47.5
	6. Safety	25	45.5
	7. Low speeds	5	8.5
	8. Noise	5	8.5
	9. Other	6	10.2

Table 5. Continued

Question #11		58	98.3
Respondents’ opinions on establishing speed limits on gravel roads? (multiple choices)	1. Use BSL and do not post it on roads.	40	69.0
	2. Prefer a lower speed limit than 55 mph for gravel roads.	22	37.9
	3. Prefer speed zones on some roads because they work better than BSL.	9	15.5
	4. Only some roads need speed limits and the rest do not need.	6	10.4
	5. A BSL does not contribute traffic safety.	4	6.9
	6. Use blanket speed limit and post it on roads.	3	5.2
	7. Prefer a higher speed limit than 55 mph for gravel roads.	1	1.7
	8. Other (specified by respondents):		
	a. Recommend 55 mph as the BSL,	6	10.4
	b. Speeds should be regulated depending on the amount of traffic and existing conditions,	4	6.9
	c. No speed limit unless needed for safety.	1	1.7

Question #12									55	93.2
Rank the importance of listed factors when establishing speed limits on gravel roads.	Factors\Importance	H	%	M	%	L	%	N	%	
	1. Surface condition	38	69.1	13	23.6	2	3.6	1	1.8	
	2. Sight distance	35	63.6	16	29.1	2	3.6	1	1.8	
Note:	3. Accident history	32	58.2	17	30.9	4	7.2	1	1.8	
H = High;	4. Road damage by heavy vehicles	29	52.7	17	30.9	4	7.2	2	3.6	
M = Moderate;	5. Road width	26	47.3	22	40.0	4	7.2	1	1.8	
L = Low;	6. Statutory regulations	25	45.5	19	34.5	6	10.9	2	3.6	
N = None.	7. Curvature	23	41.8	23	41.8	5	9.1	1	1.8	
	8. Traffic volume	19	34.5	22	40.0	9	16.4	1	1.8	
	9. 85th percentile speed	16	29.1	22	40.0	13	23.6	2	3.6	
	10. Roadside development	11	20.0	29	52.7	12	21.8	2	3.6	
	11. Maintenance period	11	20.0	29	52.7	6	10.9	4	7.2	
	12. Road length	6	10.9	15	27.3	26	47.3	5	9.1	
	13. Public attitude towards speed regulations	5	9.1	24	43.6	20	36.4	3	5.5	

Question #13		25	42.4
Comments on the acceptability of the criteria currently used in setting speed limits on gravel roads.	Summary of comments:		
	1. The enforcement of speed limits on gravel roads is a problem. Insufficient enforcement would leave posted speed limits in a dilemma.	8	13.6
	2. The State Statutes is preferred to control the safety on gravel roads.	5	8.5
	3. There is no need to post speed limit on gravel roads.	4	6.8
	4. Funds are critical for routine maintenance and posting speed limit signs on gravel roads.	2	3.4

As summarized in Table 5, of the 59 responding counties, 50.8% have speed limit signs posted at special sections, and 30.5% have speed limit signs posted at general sections on gravel roads. Statutory regulation on BSL is the leading criteria for setting speed limits in 48.3% of these counties, followed by engineering study and professional judgment.

All the responding counties have received complaints on gravel roads from the public. Poor road condition was the one most frequently reported, 88.1% of the counties having received it, and the next were dust pollution (72.9%), vehicle speeding (62.7%), high speeds (54.5%), narrow width (47.5), and safety (45.5%).

Fifty-eight counties (98.3%) responded to Question 11. Sixty-nine percent of the counties supported adoption of an unposted BSL for gravel roads, and 5.2% of the counties supported a posted BSL. Of the respondents, 15.5% preferred speed zones to a BSL. About 10.4% thought that only some roads need speed limits and the rest do not need them. As for speed limits on gravel roads, 37.9% of the counties preferred a value lower than 55 mph, and 10.4% preferred the 55 mph BSL unchanged. Only 1.7% preferred a value higher than 55 mph.

Question 12 shows the rank of importance of the factors to account for when establishing speed limits on gravel roads. Surface condition was recognized as the most important factor by 69.1% of the respondents. The following important factors were, in order, sight distance, accident history, surface damage by heavy vehicles, road width, statutory regulations, curvature, and traffic volume. 85th percentile speed ranked in the ninth position and was supported by only 29.1% of the respondents. Public attitude towards speed regulations and road length were the two least important factors. It was noted that 74.5% of the respondents ranked traffic volume as highly or moderately important, though traffic volume on gravel roads is very low (usually under 100 vehicles per day) and has very limited influence on speed distribution and crash risk.

Additionally, eight counties mentioned that if speed limits were posted on gravel roads the enforcement would be very difficult because most counties do not have enough police officers to patrol gravel roads. Otherwise, it is a dilemma that posted speed limits are not patrolled and are therefore overlooked by drivers.

CRASH ANALYSIS

Analysis of crash data was performed to find out the impact of speed limits on crash severity and to identify other critical factors that have influence on crash severity or are interrelated with speed limits on gravel roads. The KARS database, maintained by KDOT, was used in this study. KARS consists of details of all police-reported crash data on public roads in Kansas. To reduce the influence of the lapse of time, a time period of 2003–2005 was considered, and relevant data was extracted from the original data set. Contingency table testing was applied to identify the significance of relationships between two variables from a set of thirteen variables that had been identified from KARS. Table 6 lists the categories of these variables.

Table 6. Categories of the variables for crashes on gravel roads

Variables	Categories
Speed limit	30 mph, 35 mph, 40 mph, 45 mph, 50 mph, and 55 mph.
Crash severity	Fatal, Disabled, Non_incapacitating, Possible injured, and Not injured.
Accident class	Non-collision, Overturned, Collision with other vehicle, Animal, and Fixed Object.
Contributing circumstances	Driver, Environment, Road, and Vehicle.
Collision with other vehicles	Head on, Rear end, Angle-side impact, Sideswipe, and Backed-into.
Collision with fixed object	Utility devices, Fence/gate, Ditch, Embankment, and Tree.
Driver age group	Old (65 years), Middle (>25&<65 years), and Young (25 years).
Driver factor	Too fast for conditions, Inattention, Under the influence of alcohol, Avoidance/Evasive action, Fail to yield the right of way, and Exceeding posted speed limit.
Driver gender	Female and male.
Light condition	Daylight, Dawn, Dusk, and Dark.
Road characteristics	Straight and Curve.
Surface condition	Dry, Wet, Snow/Slush, Ice/Snowpacked, and Mud/Dirt/Sand.
Weather condition	No adverse conditions, Rain, Snow, Fog/Smoke, and Strong Wind.

Source: KARS, KDOT, 2007

Speed limits were found to have significant impacts on the distribution of crashes by severity. As shown in Table 7, roads with higher speed limits were likely to have a bigger percentage of injury crashes than roads with lower speed limits. The biggest proportion of fatal crashes occurred on 50 mph gravel roads, and then on 55 and 45 mph roads. Disabled crashes accounted for the biggest percentage on 45 mph roads, and then on 40 and 55 mph. Gravel roads with a speed limit no higher than 30 mph had the smallest percent of injury crashes. Additionally, 45 mph roads gravel roads had the largest percentage of severe crashes (fatal and disabled), and 50 mph roads had the largest percentage of injury crashes.

Table 7. Percentage of crashes on gravel roads for different speed limits and crash severities

Speed limit (mph)	Crash severity					
	Fatal	**Disabled**	**Non-incapacitating**	**Possible Injured**	**Not Injured**	**Total**
30	0.0%	1.4%	10.7%	10.3%	77.5%	100%
35	0.6%	3.2%	16.3%	9.8%	70.1%	100%
40	0.0%	3.9%	11.7%	13.6%	70.9%	100%
45	1.2%	6.6%	12.6%	8.9%	70.7%	100%
50	1.5%	2.3%	18.8%	15.0%	62.4%	100%
55	1.3%	3.5%	16.9%	11.5%	66.8%	100%

The results of the contingency table test are presented in Table 8. Whether two variables are significantly dependent on each other at the 95% level can be determined from the comparison of chi square computed and chi square table. If the chi square computed is larger than the chi square table, the null hypothesis that the two variables are independent is rejected. In Table 8, it can be seen that most of the chi square computed values were larger than the corresponding chi square table values, except for the one between speed limit and driver gender. Therefore, it was concluded that all the 11 variables have significant impacts on the severity of gravel road crashes and are closely interrelated with speed limits, except for driver gender. In other words, the gender of drivers did not show significant influence upon the crashes on gravel roads at different speed limits.

Table 8. Summary of crash analysis results on gravel roads

Variables	Crash severity			Speed limit		
	chi square Computed	chi square Table	95% Confidence Dependence	chi square Computed	chi square Table	95% Confidence Dependence
Accident class	1549.8	21.03		1160.6	21.03	
Contributing circumstances	857.5	21.03		243.0	16.92	
Collision with other vehicles	186.2	15.51		209.0	21.03	
Collision with fixed object	176.5	26.30		283.0	21.03	
Driver age group	191.6	15.51		36.1	12.59	
Driver factors	161.4	31.41		226.0	25.00	
Driver gender	82.9	9.488		1.38	7.815	×
Light condition	116.0	16.92		579.9	21.03	
Road Characteristics	9.9	7.815		41.6	7.815	
Surface condition	141.7	21.03		56.5	21.03	
Weather condition	49.6	21.03		24.6	21.03	

CONCLUSIONS

Field study showed that traffic speeds were distinct on gravel roads with different road characteristics and under different weather conditions, though the speed limits on these roads were uniformly regulated by the State Statues. The crash analysis also supported the idea that many variables have shown impacts on both the severity of crashes and the crash frequency on gravel roads with a variety of speed limits. Therefore, a feasible speed limit should take into account those critical factors that are likely to affect traffic safety on gravel roads, such as the factors discussed above.

Many of the county engineers were in favor of a lowered regulatory speed limit for gravel roads, while a few engineers strongly recommended leaving the 55 mph regulatory speed limit unchanged. Though the popular belief is that a lower speed limit would contribute to less severe crashes, the crash analysis has not agreed with this point. The roads with speed limits of 45 mph and 50 mph had slightly higher percentages of fatal and disabled crashes than 55 mph roads. If the regulatory speed limit were lowered from 55 mph to 45 or 50 mph, no evidence would ensure the decrease of severe crashes on gravel roads. As the issues of safety and high speeds on gravel roads very much concern the public, as indicated by the survey, another survey is being developed targeting rural residents who use gravel roads on a daily basis.

As many of the county engineers mentioned, speed limit is effective in controlling the traffic only after it is posted on the roads and well enforced. However, it is not easy for most of the counties to achieve this, since quite a few counties emphasized that the road funds are not sufficient for these endeavors. Therefore, other than altering the regulatory speed limit, some other approaches could be taken to achieve safer driving on gravel roads, such as driver education and graduated licensing, which have been proven to very effective in other studies (Levy 1990; TRB 1996).

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A Bridge Structural Health Monitoring and Data Mining System

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ABSTRACT

Structural health monitoring (SHM) is becoming a more widely accepted way to improve bridge management. In 2005, a fiber optic SHM system was developed and deployed by the Iowa State University Bridge Engineering Center to continuously monitor bridge performance under ambient traffic loads and to detect potential gradual deterioration or sudden damage, specifically for Iowa's fracture-critical bridges. Strain time history data collected by this system were utilized to construct a baseline model that is based upon extreme-matching distribution. Structural responses deviating from the baseline distribution are considered as indication of damage or degradation.

As a means to improve the damage/deterioration prediction capabilities of the above mentioned system, it is postulated that the dispersion of the extreme-matching data could be minimized using truck (position, type, etc.) information. Thus, techniques for the determination of detailed truck information are being investigated. Finite element analysis was carried out to verify the proposed truck detection and damage detection algorithms.

In this paper, the SHM system, relevant autonomous data mining results, and numerical verification are presented. Moreover, ongoing efforts in estimating the truck geometry/type, weight, and velocity are described.

Key words: fiber Bragg grating—strain—structural damage identification—structural health monitoring—truck identification

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INTRODUCTION

Bridge aging and deterioration are impacting the nation's transportation safety and efficiency. In the United States, more than 50% of all bridges were constructed before 1950, and as of the year of 2003 27.1% of them (160,570) were rated as structurally deficient or functionally obsolete. It will cost $9.4 billion a year for 20 years to eliminate these bridge deficiencies (U.S. DOT 2003). To optimize the maintenance funding allocation and improve bridge management, it is essential to understand the real performance. Further, detecting damage/deterioration as early as possible will help owners get out in front of the deterioration curve and stay there. However, current predominant visual inspections have been shown by the Federal Highway Administration to be inefficient and unreliable at detecting localized damage (FHWA 2001). Other discrete short-term performance assessment approaches (e.g., traditional nondestructive testing techniques) only provide snapshots of structural conditions. They are inadequate for long-term structural integrity evaluation, especially for problems like fatigue damage detection. All these underline the importance of developing reliable and cost-effective tools for long-term structural health status evaluation.

Technological advancements in sensing, computing, and networking within the last decade have facilitated the exploration of long-term structural health monitoring (SHM) techniques, which conceptually allow for noninterruptive remote monitoring and evaluation of bridges or bridge components for extended periods of time. These approaches have been more and more widely accepted as a potential tool to prevent catastrophic failures and to improve bridge management.

On the other hand, dependent upon the number of deployed sensors and data acquisition frequency, the volumes of the data that are typically collected by long-term SHM systems are unmanageably large. Therefore, data mining, which addresses the problem of how to extract useful information from the data sets, is crucial for a successful SHM system deployment. Reliable damage identification is the objective of an SHM data mining procedure. In the literature, various damage detection algorithms have been proposed based on different mechanical principles. In general, they can be classified into two categories: dynamic-based or non-dynamic-based (Sohn et al. 2003). Dynamic-based damage detection algorithms use dynamic response parameters, such as natural frequencies, modal shapes, and damping, as the damage indicator. One attractive feature of dynamic-based monitoring that has contributed to its popularity is that the dynamic properties of a bridge are generally not impacted by the loading conditions. This eliminates unknowns involved in the monitoring that utilizes ambient traffic loads. However, application has shown that these dynamic-based methods have their own limitations when applied to complicated structures. First, variation in excitation or environmental conditions can cause changes in the dynamic properties that are large enough to mask changes due to damage formation. Secondly, the global dynamic parameters are not sensitive to certain types of local damages that are located in small structural response regions (such as joint details). Moreover, the multidegree of redundancies associated with highway bridges may also reduce the sensitivity of dynamic-based damage detection approaches. Due to these factors, a strain-based method is considered to potentially be more appropriate for fatigue formation detection. In addition, compared to dynamic parameters, strain is easier to monitor and better understood by bridge engineers.

In 2005, upon the request of Iowa Department of Transportation (Iowa DOT), the Bridge Engineering Center at Iowa State University began to develop and deploy a remote strain-based SHM system, which continuously monitors the performance of a fracture-critical steel girder bridge under ambient traffic loads. These fracture critical bridges were found to be fatigue damage–prone in the unstiffened web gap region due to out-of-plane bending. As a retrofit, the floor beam connection plates in the negative moment regions were cut back to reduce stress levels. However, fatigue crack formation at cut-back areas has not been fully eliminated. Therefore, having the ability to continuously monitor for fatigue damage at the retrofit cut-back areas was considered to be necessary. The developed SHM system was installed on the

US-30 South Skunk Bridge near Ames, IA. It autonomously collects, reduces, and analyzes strain data and generates periodic structural performance reports that may be used to support maintenance decision making. Although the system was developed for fracture-critical bridges, it could be easily extended to other steel girder bridges.

The hardware of the US-30 bridge SHM system consists of three components: a data acquisition component, a data processing and management component, and a data communication component. For data acquisition, 40 fiber Brag grating (FBG) strain sensors were strategically deployed throughout the bridge. They are classified as either target sensors (TSs) or non-target sensors (NTSs) according to their location. Specifically, TSs were installed at the retrofit cut-back areas to sense the local strain response, while the NTSs were installed at non-damage-prone areas to capture the global structural response. Obviously, TSs are more sensitive to the interested fatigue damage than NTSs due to their vicinity to the potential damage locations. Details of the system hardware components will be discussed later.

The software for the US-30 SHM system has two modules: a user friendly graphic user interface (GUI) and an autonomous data mining kernel. The data mining procedure developed for this system includes three major steps. First, a data cleansing step is designed to remove unwanted temperature and dynamic effects; then, the cleaned strain data are reduced to a series of TS-NTS extreme-matchings for each truck event; finally, a unique unsupervised damage detection method is carried out that is loosely based upon the control chart concept. In the initial training period, an adequate number of event samples are collected to construct baseline extreme-matching distributions. A structural response that deviates from the baseline distribution is considered to be an indication of structural damage or degradation.

Recently completed finite element analysis verified the effectiveness of the data mining approach. However, the numerical analysis results also indicated that truck parameters, including truck geometry configuration, truck weight, and transverse position can contribute to the deviation of the baseline extreme-matching distributions. Therefore, including a truck detection function, which determines these factors, into the existing system is postulated to improve damage detection ability. This function has been conceptually developed and implemented at the synthetic data level. Field testing and field data debugging will be performed in the near future. After field verification, the new module will be integrated into the existing SHM data mining system.

HARDWARE SYSTEM

The SHM system developed for and installed on the US-30 bridge is schematically depicted in Figure 1. As shown in the figure, according to the physical location of the equipment, the components can be classified into two portions: the bridge site portion and an office site portion. The bridge site component is composed of 40 strategically deployed FBGs, a Micron Optical Si425 interrogator, a Linksys router, and a desktop computer. The data collected by sensors and the interrogator are relayed through the router to the bridge site computer, where they are temporarily stored and immediately processed. After the data have been processed, extracted information and a periodic structure performance evaluation report are sent to the remote office computer for permanent storage.

Data Acquisition Component

The SHM system utilizes strategically deployed FBG sensors and a si425-500 interrogator to collect the strain data. Sensor installation location and orientation must be carefully designed to capture the global structural response, as well as the localized strain response at damage-prone areas. Figure 2a depicts the six instrumented cross sections for a demonstration application on a bridge located on US-30 near Ames, IA. Details of typical TSs and NTSs installed at Section C are shown in Figure 2b.

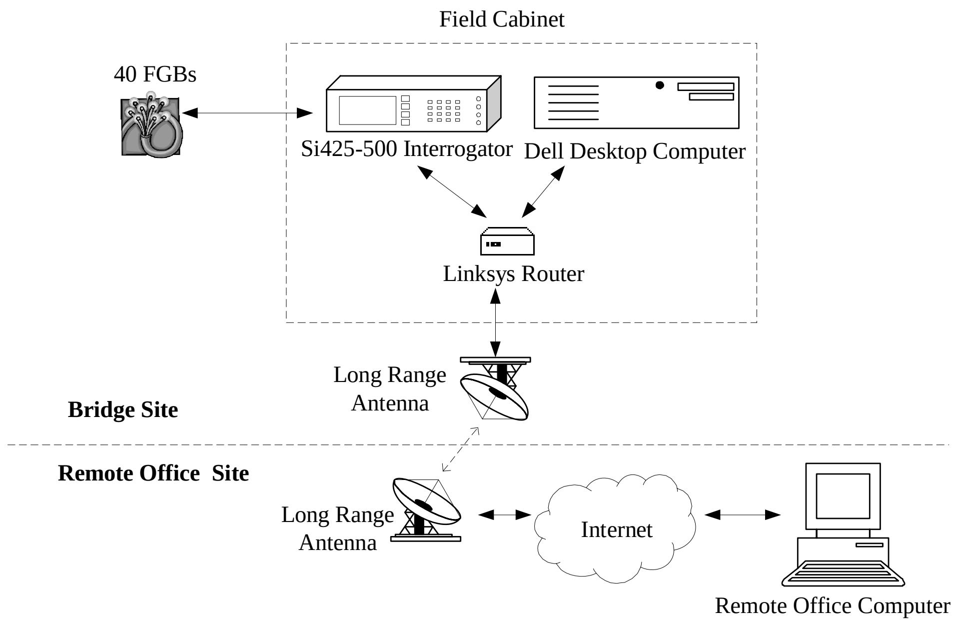
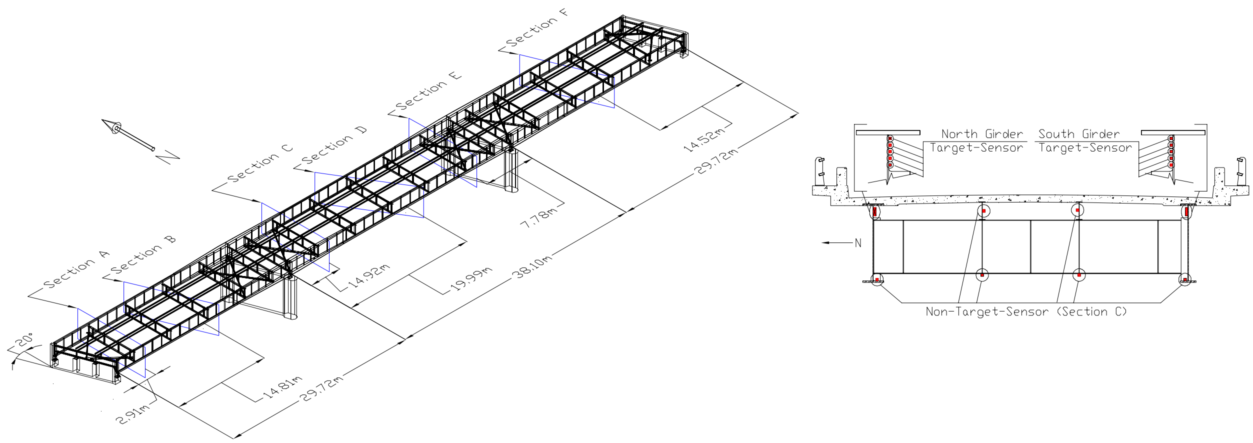


Figure 1. Schematic of the SHM hardware system



a. Instrumented sections **b. TSs and NTSs at Section C**

Figure 2. Sensor deployment

Considering the gradient of the strain field in some of the sensing areas, three different types of FBG strain sensors were used in this system. A 10mm FBG is suitable for global sensing, since the strain fields at these areas are relatively uniform and the strain averaged over the sensor length can accurately represent the strain at a desired point. The 10mm FBG is embedded in a 210 x 20 x 1 mm CFRP packaging, which protects the sensor and provides more surface bonding area to ensure a robust and easy installation. The fiber pigtails exiting from each side of the packaging (entry fiber and exit fiber) consist of SMF simplex cable (3 mm jacketing) and FC/APC mechanical connectors. Due to the complex nonuniform strain field, the 10 mm sensor is not suitable for the strain sensing at cut-back regions. To measure strains in these regions, two different 5 mm FBGs were studied and designed: (1) single 5 mm FBG in a small form factor CFRP packaging and (2) an array of five 5 mm FBGs embedded in a single CFRP package.

The 40 FBGs on the US-30 bridge are distributed among three individual fiber optic leads, and each fiber was connected to one channel of the si425-500 interrogator. The FBGs within any one fiber were designed with approximately 5 nm of separation between adjacent center wavelengths. Several procedures were performed to ensure longevity of the system. The optical fibers were intermittently secured with cable ties to mounting bases that were bonded to the bridge. In addition, all FBGs, fusion splices, and mechanical connectors were covered with a layer of silicone sealant to protect them from moisture. Finally, after the fiber optic network was installed, an optical time domain reflectometer was used to scan the network and check for regions with large optical attenuation. Examples of such optical loss include sharp bends or pinches in the fiber that could lead to extremely low optical levels or even fiber breakage. This would result in the inability to interrogate the FBGs.

Data Processing and Management and Data communication Component

The data processing and management equipment consists of two computers, which are located at the bridge site and office site, respectively. The bridge site computer is responsible for temporary data storage and real-time data processing. The information extracted from the raw data was sent to the remote office computer through a wireless internet connection. The si425-500 interrogator and bridge site computer operate within a private subnet that is managed by the router through wired connections. Data streams are transferred from the interrogator to the computer through a TCP/IP socket. The private network is then connected to internet.

SOFTWARE SYSTEM

The software for the developed system has two modules: a user friendly GUI and an autonomous data processing kernel. Both modules are implemented with LabView, which is a development environment for visual programming. As presented in Figure 3, the GUI allows the user to configure the system operation parameters and easily switch between training and monitoring modes. The kernel is designed to perform automatic data collection and data mining. More than three gigabytes of data are collected by the US-30 SHM system every day. Analyzing such large data sets requires the development of an efficient data mining solution that is well founded in structural performance and feature identification. The designed data mining algorithm analyzes the relationships and patterns of the raw data and evaluates the structural health status using the statistical method known as control chart analysis. Steps of the data mining procedure, including data cleansing, data reduction, and data interpretation, will be described in the following.

Data Cleansing

Static ambient traffic–induced strain was decided to be the desired measurement metric for the damage detection algorithm. However, in normal operation, strains caused by other uncertain factors, such as temperature and dynamic effect, are unavoidable and are recorded together with the desired signal. Fortunately, these types of signals are typically pattern-based and can be removed after the recognition of the pattern. In this system, a zeroing function and a low-pass Chebyshev filter are developed to remove the temperature and dynamic effect, respectively. As can be seen from Figure4a, the temperature-induced strain is constant for a sufficiently short monitoring period, and it can be removed by simple subtraction.

An iterative method is involved to determine the temperature-induced strain. The configuration of the Chebyshev filer for each sensor is determined using Fast Fourier Transform (FFT) and Power Spectra Density (PSD) forms of analysis. Figure4b is an example that shows the effectiveness of the selected Chebyshev filter.

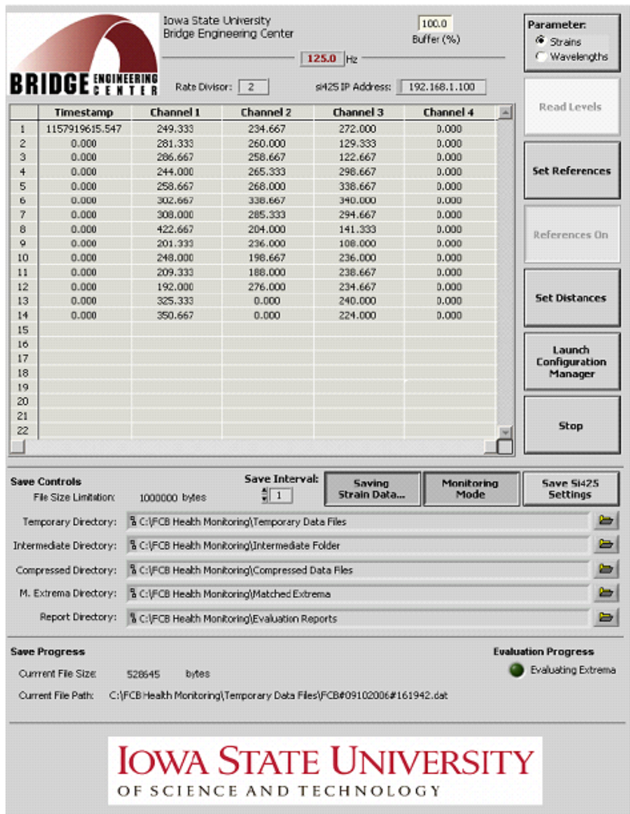
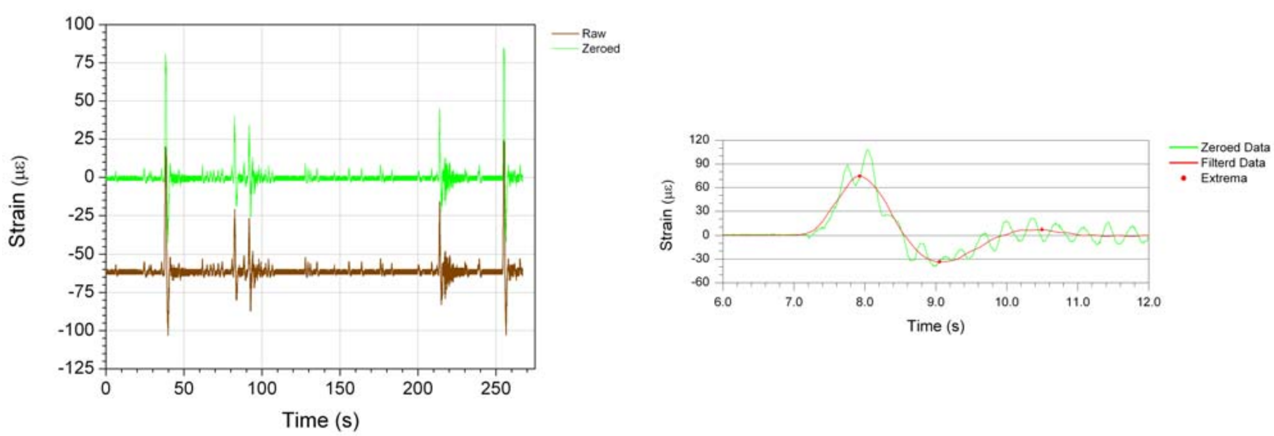


Figure 3. GUI of the SHM software



a. Data zeroing **b. Data filtering**

Figure 4. Data cleansing

Data Reduction

The data acquisition equipments of the US-30 system record strains at 40 points with the frequency of 125Hz. To extract the useful information, the event extremes (both maximum and minimum) were first identified and then matched to create the TS-NTS extreme-matchings. Using the extreme-matchings instead of the entire strain time history increases the data processing speed and reduces the required data storage space dramatically. An example of the extreme-matching distribution is presented in Figure 5.

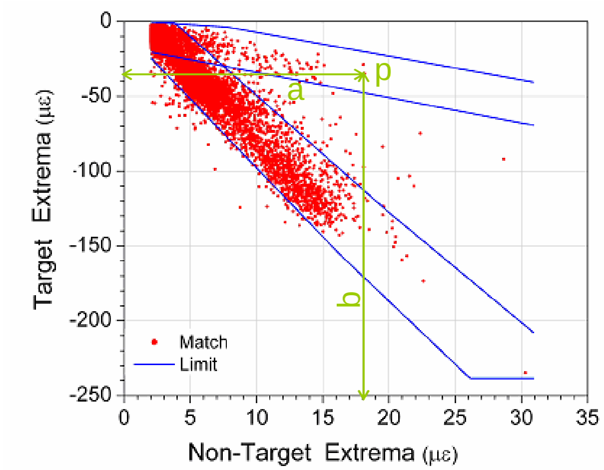


Figure 5. Example of the TS-NTS extreme-matching distribution

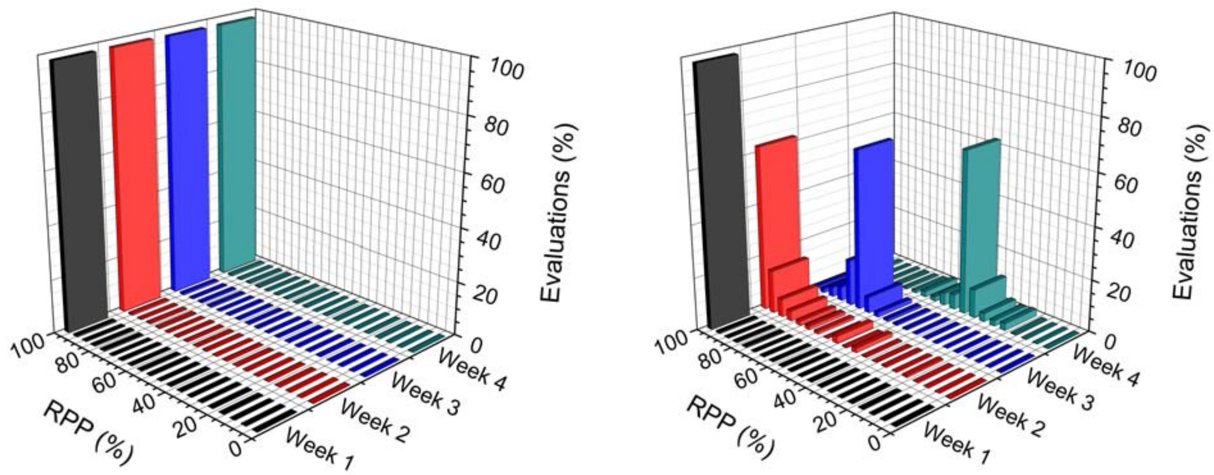
Data Interpretation

Interpretation of the monitoring data in terms of the integrity of the structure is the key function of the SHM system. For this approach, a unique unsupervised learning algorithm was designed based on the concept of control chart analysis. Unsupervised learning means that the data from the damaged structure are not required for system training. Control chart analysis is a statistical data analysis technique that has been widely used in process control in the chemical industry, in manufacturing, and in nuclear power plants. Using this technique, measured quantities are continuously monitored for anomalies. The analysis procedure typically involves two stages. First, the control limits are established based on the distribution of the observations, which are obtained from the normal condition. Then, new data for an unknown condition of the system are compared against the control limits. The system alarm is activated when the new observations fluctuate outside the control limits.

The data interpretation approach designed for this system involves two stages as well. In the initial training stage, the system was exposed to the response pattern of the undamaged structure. After sufficient learning time, the baseline distributions for the TS-NTS extreme-matchings are constructed as shown in Figure 5. In this case, the upper and lower limits used in control chart analysis were set manually. Points outside the limits indicate unusual events and are termed a fail assessment; points within the limits are termed a pass assessment. With this approach, one single fail assessment cannot be used as damage indication, since it could be caused by atypical vehicles or vehicle combinations. However, if the number of fail assessments increases significantly or shows an increasing trend, it is believed that this is a sign of the formation of damage or deterioration. To quantitatively evaluate the pass and fail assessment, a relationship pass percentage (RPP) is computed with Equation (1), and it is used as the structural damage indicator.

$$\text{RPP (\%)} = \frac{\text{Number of "Pass" Assessments}}{\text{Total Number of Assessments}}(100) \quad (1)$$

During autonomous monitoring, weekly RPP histograms are generated automatically. Figure 6a presents a typical one month of weekly RPP histogram for an undamaged structure. The RPP is not expected to be 100% and will fluctuate slightly from week to week due to the existence of atypical events. However, when structural damage or deterioration starts to form, the RPP histogram should show notable changes. As shown in Figure6b, it is predicted that the mean of the weekly RPP will slowly decrease for deteriorated structures. For damaged structures, the changes would be much faster.



a. Undamaged structure **b. Prediction for damaged structure**

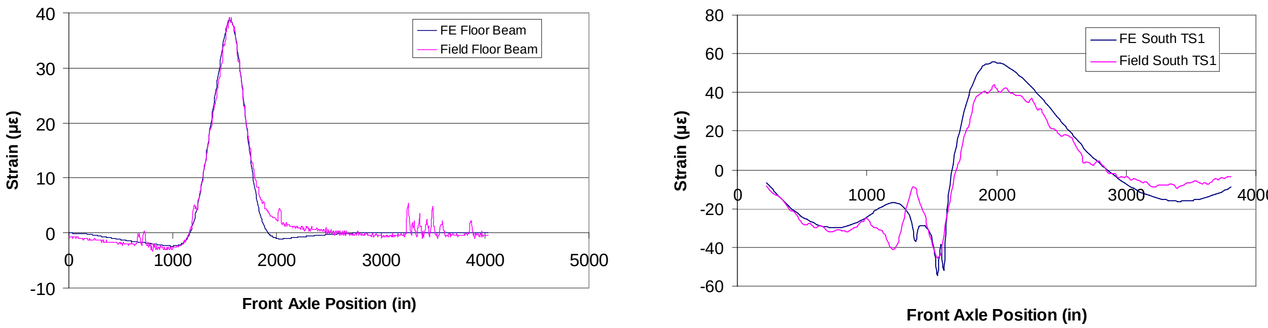
Figure 6. One month of weekly evaluation reports

NUMERICAL STUDY

To understand the performance of the damaged bridge and to verify the effectiveness of the developed system, a finite element analysis (FEA) was carried out with a commercially available software package. In this analytical study, the distributions of TS-NTS extreme-matching for damaged and undamaged structures were compared to see if they were statistically different (i.e., that damage could be detected) and to determine the minimum detectable crack size.

Finite Element Model Description

A 3D finite element (FE) model was constructed for the numerical study. In this model, shell elements were used to create the deck, stringers, floor beams, and girders. Composite action between the deck and girders/stringers was modeled with rigid links. The FE model was verified and tuned using controlled field testing data, and the final model represents the global response of the structure well (see Figure 7a) with only satisfactory representation at the local level (see Figure 7b).

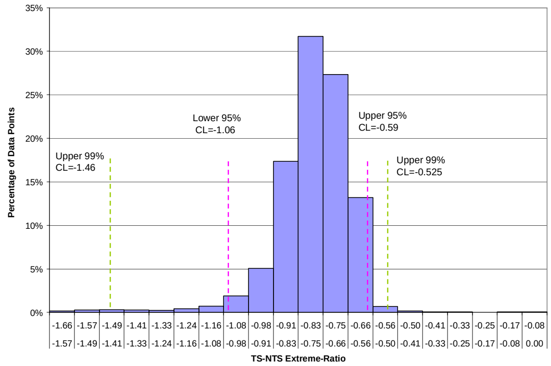


a. Example for NTS **b. Example for TS**

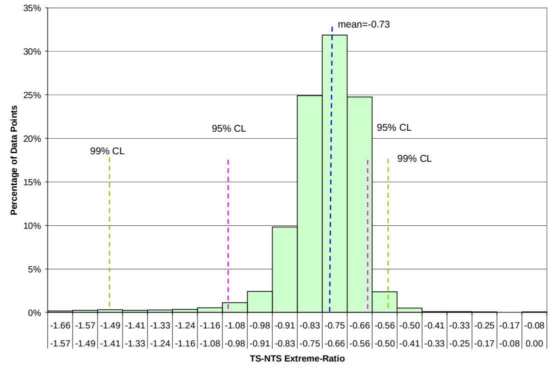
Figure 7. Comparison FEA results to field data

Conceptual Verification of the Implemented Crack Detection Algorithm

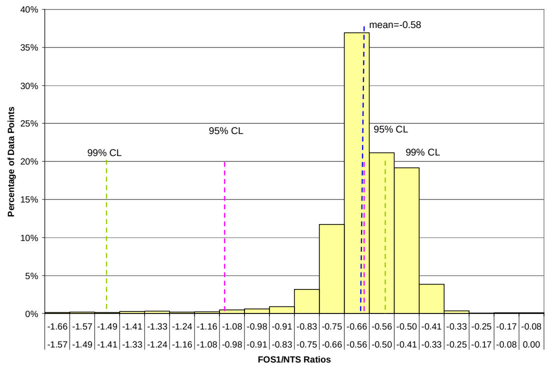
To determine the strain response effects caused by structural damage, a crack with the size of 0.5 in., 0.75 in., and 1in. was introduced to the FE model right above the web of the floor beam and two in. below the top flange of the girder. A histogram of extreme-ratios was constructed for both the damaged and undamaged structure. The extreme-ratio could be calculated easily from predefined extreme-matching. For example, in Figure 5, the extreme-ratio for the selected point *p* is simply the ratio of *a*/*b*. Figure8a presents the histogram of the extreme-ratio for the undamaged structure. This histogram was constructed from actual field monitoring data. The 95% and 99% confidence limits (CLs) for the histogram distribution were created and are also shown in the plot. A histogram for the damaged structure was constructed in the following way. The mean of the distribution was obtained from the FEA results, while the variance and the sample size were assumed to be the same as those used in the undamaged structure distribution. For a particular structural condition, if the mean of the associated extreme-ratio is out of the defined CLs, the damage of the structure is considered to be statistically significant. Figures 8a through 8d suggest that the implemented damage detection algorithm can detect a crack with a size between 0.75 and 1 in. with a confidence level of 99%. It should be noted that the analysis approach is not exactly the same as that used in the monitoring system and that the assumption about variance may not be realistically a good assumption. Nonetheless, the analysis results conceptually verified the correctness of the damage detection method.



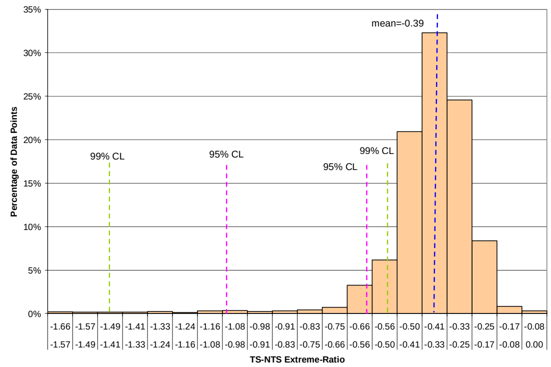
a. No crack



b. 0.5 in. crack



c. 0.75 in. crack

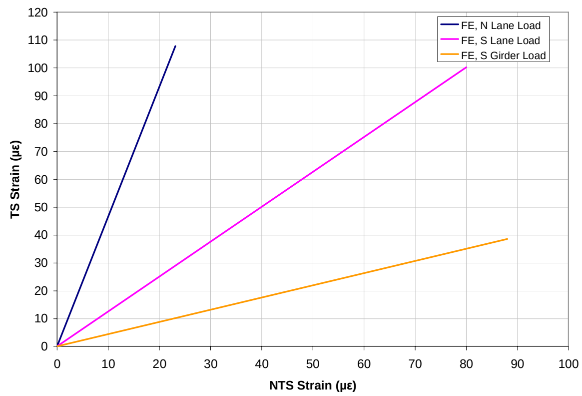


d. 1 in. crack

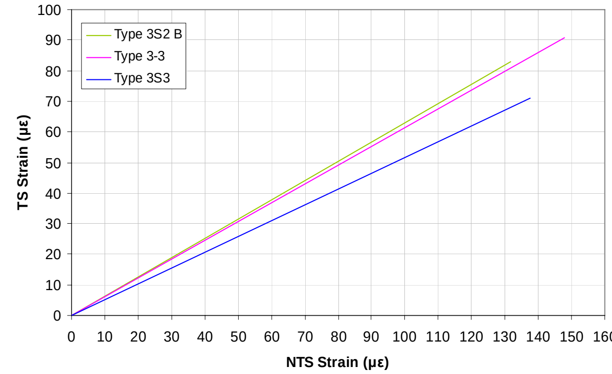
Figure 8. Histogram of the extreme-ratio for damaged and undamaged structures

Identification of other Relevant Parameters

In the early stage of development, it was thought that the TS-NTS extreme-ratio would not be affected by strain variance caused by truck parameters due to the linearity of the structure. However, the analytical results indicated the relevance of these parameters. It can be seen from Figure9a that identical trucks acting at three different locations produced three different extreme-ratio values. Similarly, Figure9b indicates that trucks with the same total weight but different geometry configurations produced different extreme-ratio values too. Ignoring these parameters in the baseline distribution, which represents the undamaged structure, may result in a decrease in the damage detection ability of the system.



a. Effect of truck transverse position



b. Effect of truck geometry

Figure 9. Factors that affect the extreme ratio

PROPOSED DATA MINING IMPROVEMENTS

As discussed previously, the existing damage detection approach may be improved by including more relevant factors into the baseline evaluation model. Further, constructing the limits used in the control chart analysis through an advanced statistical approach instead of the currently used subjective/manual method may also improve the damage detection capabilities.

A unique truck detection function has been designed to quantitatively evaluate truck parameters for this monitoring. A multiple bottom-of-the-deck sensor deployment strategy similar to the one shown in Figure10 allows for the detection of the number of axles, transverse position, speed, and axle spacing. For a particular deck bottom sensor, each truck axle produces a unique peak point in the strain time history data when it is in the vicinity of the sensor. Figure 11 presents synthetic strain time histories for six selected sensors produced by a three-axle vehicular event. The locations of the sensors are shown in Figure10. The three peaks in each sensor data instance (Figure 11) represent the three vehicle axles. As the coordinates of the sensors and the timestamp for each peak are known, computing of the truck speed and truck axle spacings is straightforward. The relative magnitudes of the strain peaks for sensors within the same group can also be used to infer the truck transverse position. For example, as shown in Figure10, S11, S12, and S13 belong to one sensor group (SA1), while S21, S22, and S23 belong to the other sensor group (SA2). In Figure11, the peak value for S11 is greater than that for S12 and S13. This shows that the transverse position of the truck is nearer to S11 than to the other two sensors.

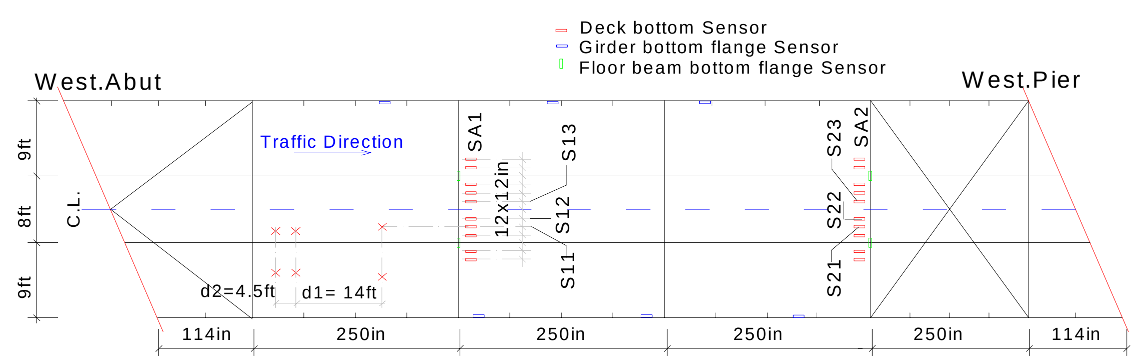


Figure 10. Sensor deployment for truck detection

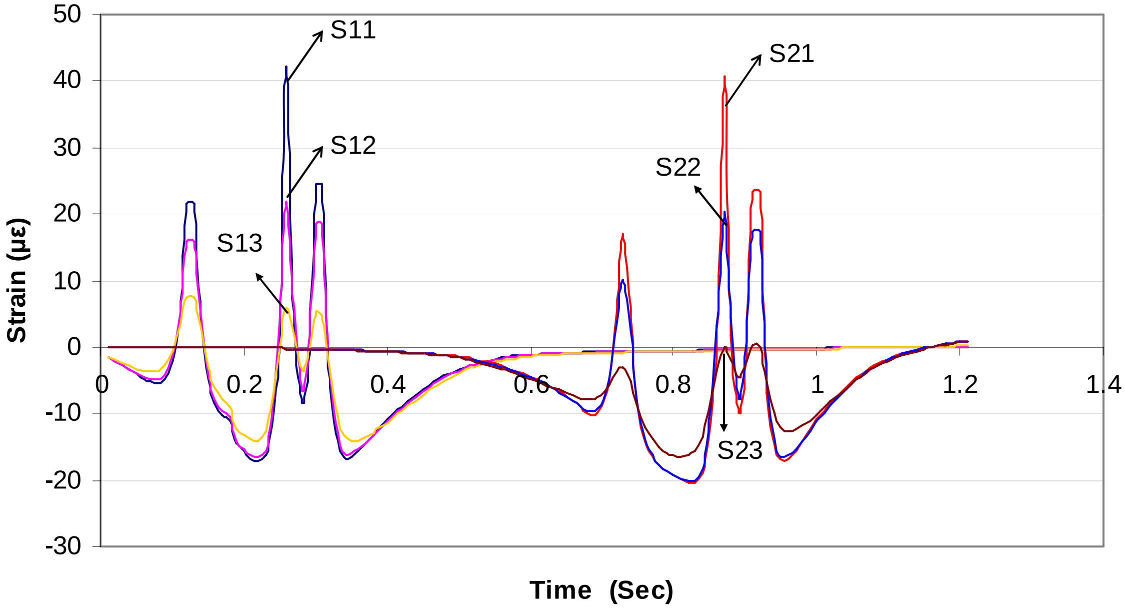


Figure 11. Synthetic strain time history plot for six selected deck bottom sensors

The truck axle weights and total weight can be calculated with a novel concept. Specifically, an axle-based influence line/surface for the girder bottom flange sensor can be constructed by running a control truck on the bridge multiple times and then solving the simultaneous equations shown in (2):

$$\begin{vmatrix} W_{11} & W_{12} & W_{13} \\ W_{21} & W_{22} & W_{23} \\ W_{31} & W_{32} & W_{33} \end{vmatrix} \begin{vmatrix} I(P) \\ I(P-D_1) \\ I(P-D_2) \end{vmatrix} = \begin{vmatrix} R_1(P) \\ R_2(P) \\ R_3(P) \end{vmatrix} \tag{2}$$

where,

$W_{11}, W_{12} \cdots W_{33}$ are axle weights for control trucks

I( ) are values in the influence line for specified locations

$R_1(P), R_2(P)$, and $R_3(P)$ are strain response at the sensing points

P is the longitudinal coordinate for the girder bottom flange sensor

D_1 and D_2 are truck axle spacings

Typically, $W_{12} = W_{13}$ & $W_{22} = W_{23}$, so $I(P) = \dfrac{W_{22}R_1(P) - W_{12}R_2(P)}{W_{22}W_{11} - W_{12}W_{21}}$

Figure12 shows the good agreement between the influence line constructed from synthetic data using the described approach and the theoretical solution. To make the synthetic data more realistic, each value has been randomly deviated up to 2% to simulate random noise.

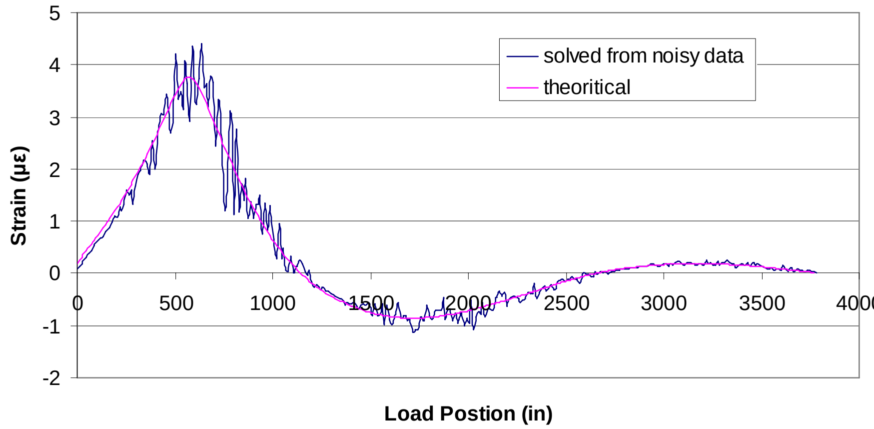


Figure 12. Influence lines for the sensing position

Knowing the influence line/surface, truck speed, and strain time history data, axle weights of trucks could be estimated through a linear least squares optimization. Equation (3) is the objective function of the optimization, in which, R_reconstruct (P,D,W) is a function of the truck first axle position (P), axle spacings (D), and axle weights (W). P and D are known from the monitoring data, and W is the optimization variable. After determining the weight of each axle, the total gross weight is then calculated as the summation of axle weight. As it turns out, the influence line/surface constructed from the control truck can be utilized in the axle weight estimation for any type of truck. Therefore, during the training stage, the system does not have to be exposed to all type of trucks. Table 1 summarizes the weight estimation results for a semi truck using the influence line, which was constructed from a three-axle dump truck.

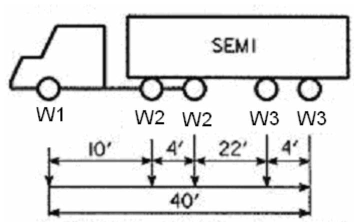
$$SumSquareErr = \sum_{P} (R\_reconstruct(P,D,W) - R\_monitor(P))^2 \qquad (3)$$

where,

P is the positions of first axle of the truck
D represents the truck axle spacings
W represents the axle weights of the truck

Table 1. Axle weight and total weight estimation for a semi truck

	Target (k)	Predicted (k)	Error
W1	6	5.38	-1.03%
W2	6.5	6.74	3.7%
W3	7.75	7.61	-1.7%
W Total	34.5	34.09	-1.19%



Following further verification of this approach, statistical data analysis approaches will be developed to set the control chart limits and set up alarming thresholds for the RPP histogram. It is thought that these analysis approaches will in some way incorporate the vehicle configuration information.

CONCLUSIONS AND FUTURE WORK

The SHM system described herein enables bridge owners to remotely monitor bridges for gradual deterioration or sudden damage formation. The system is trained with performance data to identify the typical bridge response when subjected to ambient traffic loads. During monitoring, strain records in data files are zeroed and filtered, and event extremes are extracted automatically. After the TS-NTS extreme-matching searching is completed, RPPs are calculated using the knowledge learned during the training stage. From generated RPP histograms, bridge owners can evaluate the structure's health without needing to fully understand the data mining procedure. The effectiveness of the system was studied through a numerical analysis. Improvements have been proposed and partially implemented with the simulation data.

In the future, efforts will be made toward improving the damage detection capabilities and reducing the possibility of false positive alarms. As discussed before, the uncertainty with the system may be reduced by incorporating more variables into the baseline model. More sophisticated statistical pattern recognition approaches are also being developed to improve the damage detection ability of the system.

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The 70 mph Speed Limit: Speed Adaptation, Spillover, and Surrogate Measures of Safety

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ABSTRACT

Although numerous publications have studied the safety effects of speed limit policy, the topic continues to be the subject of lively debate. On July 1, 2005, the speed limit on rural interstates in Iowa was increased from 65 mph to 70 mph. This research first conducted a before–and-after study on rural interstates and other facilities to study the effects on safety performance in Iowa due to this speed limit change. The study explored the impact of the speed limit change on two effects known as the “speed adaptation” and “spillover” effects. Research was also conducted on traffic citations issued on the rural interstate because citations may be a surrogate measure for highway safety. Finally, research was conducted on the recent increase in the retail price of gasoline and its effect on driver behavior.

Key words: safety—speed adaptation—speed limit—spillover effect—traffic citations

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

DISCLAIMER

The data analysis included herein is not intended to be an exhaustive before-and-after study of the safety effects due to the increase of the rural interstate speed limit from 65 mph to 70 mph and should therefore be considered preliminary. No conclusions on the actual impact the increased speed limit has had on safety can or should be drawn from these data. An additional study conducted by the Center for Transportation Research and Education will be completed when more data are available.

1. PROBLEM STATEMENT

One of the essential components in providing safe roads is the speed limit. However, speed limit policy continues to be controversial. On July 1, 2005, the speed limit on rural interstates in Iowa was increased from 65 mph to 70 mph. This change had been long considered by policymakers and the Iowa Department of Transportation (Iowa DOT) and was the subject of lively debate. Of concern was the impact the speed limit change on rural interstates had on safety performance and whether this change negatively affected other facilities (spillover) in terms of crashes and speeds. Additionally, the rural interstate speed limit introduced a 15 mph speed limit differential to rural primary highways that intersected and shared mutual access with the interstate. This speed limit differential may have induced or augmented an effect known as the “speed adaptation” effect.

2. RESEARCH OBJECTIVES

This research first examined crash performance on and off the system. It also explored the impact of the speed limit change on an effect known as the “spillover” effect to determine if increasing the speed limit on the rural interstates negatively affected other systems in terms of crashes or speeds. Because traffic citations reflect driver behavior, they may be a surrogate for highway safety. Research was then conducted on traffic citations issued before and after the speed limit change. If the recent increase in the retail price of gasoline did reduce the amount of travel, it may have partially masked any negative effects of the speed limit change. Research was conducted on the retail gasoline price and its corresponding effect on driver behavior in terms of the amount of travel. Finally, this research studied the speed adaptation effect in rural Iowa to determine if this effect exists and over what distance.

3. RESEARCH METHODOLOGY

Crash data were obtained for the period January 1, 2003, to December 31, 2006. This provided up to 30 months for the before period and up to 18 months for the after period. The crash data used for this study were downloaded from the Iowa DOT database on April 2, 2007, and included a variety of information about each crash. Each crash is assigned a geographical coordinate that can be mapped using a geographical information system (GIS) software package. Crashes that were within a specified distance of any facility of interest were assigned to that road type within GIS. A total of 3,261 out of 232,061 crashes (for the years 2003 to 2006), or 1.4%, were not assigned a coordinate, and thus they could not be located on a map and were not used for this research. Once the crash data were assigned to the appropriate road types, they were summarized by the number of crashes within the various severity categories on a monthly basis. The severity categories included in this analysis were fatal, fatal and major injury, and all crashes. The analysis period for crashes included a before period of July 2003 to December 2004 and an after period of July 2005 to December 2006.

In addition to the crash data, speed and volume data for each facility type were collected from the Iowa DOT. During discussions with the Iowa DOT, it was noted that the automatic traffic recorder (ATR) database was changed during the summer of 2004. Because of this change, the analysis period for the speed data was August 2004 to December 2006. The speed data were summarized by the average speed and 85th percentile speed. Volume data were obtained from the monthly ATR reports provided on the Iowa DOT's website. The analysis period for volume was the same as for the crash data.

3.1. Before and After Study

Rural interstates were the primary focus of this research. Rural interstate crashes were further defined as occurring during the day or night based upon official sunrise and sunset times obtained from the United States Naval Observatory. The website of the United States Naval Observatory provides an online product that provides the sunrise and sunset times for a given year at any location within the United States (US Naval Observatory 2006). Because of the longitudinal width of the state of Iowa, the sunrise and sunset times are different for the east and west ends of the state. Therefore, it was assumed that the sunrise and sunset times for a central location in Iowa would approximate the sunrise and sunset times for every location within the state. The central location selected was Ames, Iowa.

3.2. Spillover Effect

To determine if the speed limit change negatively affected other systems, other road types were analyzed to test for any type of spillover effect. The four road types that were analyzed are the following:

- Urban interstates
- Rural expressways
- Rural other primary highways
- Rural non-primary roads

3.3. Rural Interstate Traffic Citations

When the speed limit increase was passed by the Iowa Legislature, the governor committed to stepping up enforcement to mitigate any potential safety concerns. A change in the number of speeding citations could be considered as a surrogate for a change in the number of speed-related crashes. However, information was only available for electronic citations (not paper). Therefore, the number of electronic speeding citations could not be compared before and after the change in the speed limit. Instead, the monthly ratio of rural interstate electronic speeding citations to all rural interstate electronic citations was calculated for the period of January 2004 to December 2006. A database consisting of all electronic traffic citations issued by the Iowa State Patrol was obtained through the Center for Transportation Research and Education (CTRE) at Iowa State University.

3.4. Retail Gasoline Price

The recent increase in the retail price of gasoline may have affected the travel behaviors of drivers in Iowa. Because of the higher prices, drivers may tend to drive less and attempt to conserve fuel. Therefore, it is possible that the higher cost of gasoline may have partially canceled out some effects of the increase of the rural interstate speed limit. To determine if the cost of gasoline did have an effect on driver behavior and therefore offset some of the possible negative impacts of the speed limit change, the price of gasoline for the recent history in Iowa was collected. It was presumed that if drivers altered their behavior due to the higher cost of gasoline, this behavior would be most pronounced on a facility that accommodates longer trips, namely rural interstates.

Data for the retail price of gasoline in Iowa were obtained from the United States Department of Energy (Energy Information Administration 2007). The gasoline formulation selected for this analysis was regular grade gasoline sold through retail outlets. Prices were recorded on a monthly basis. The analysis period was selected as January 2002 to December 2006, coincident with the availability of ATR reports.

3.5. Speed Adaptation

Speed adaptation is thought to occur in locations where drivers exit a high-speed facility and enter a lower speed facility (Casey and Lund 1987; Matthews 1978). Therefore, when studying this effect, it is desirable to find two adjacent facilities (one being a rural interstate) with mutual access and a large speed limit differential. In Iowa, the locations that best support these criteria are rural interstate interchanges. Study sites were selected at locations in which a rural two-lane undivided primary highway intersected with and provided access to a rural interstate that provided a speed limit differential of 15 mph. After reviewing potential study sites, a total of four were selected along the I 35 corridor. For each study site, a test section was located on the east side of the interstate. All four sites were located on flat terrain with no horizontal curves. The fourth site was eventually eliminated from the study because of data corruption that occurred during the data collection process. Figure 1 illustrates the study locations.

Using a series of road tubes, vehicles exiting the interstate were tracked through the study site. Since the counters (as shown in Figure 1) could not communicate with each other, a computer program was written to track experimental vehicles at each counter. The program used the date, time, and class fields to identify target vehicles at each counter, and it used a progressive process to track the vehicles through the system. This progressive process used speeds and times at the first counter as the basis for identifying target vehicles at the second counter. Once all the target vehicles had been tracked to the second counter, they were used as the basis for identifying vehicles at the third counter. This process was repeated until all target vehicles were identified. Control vehicles were those identified as traveling in the westbound lane. Speeds for control vehicles were observed at the last (east-most) counter.

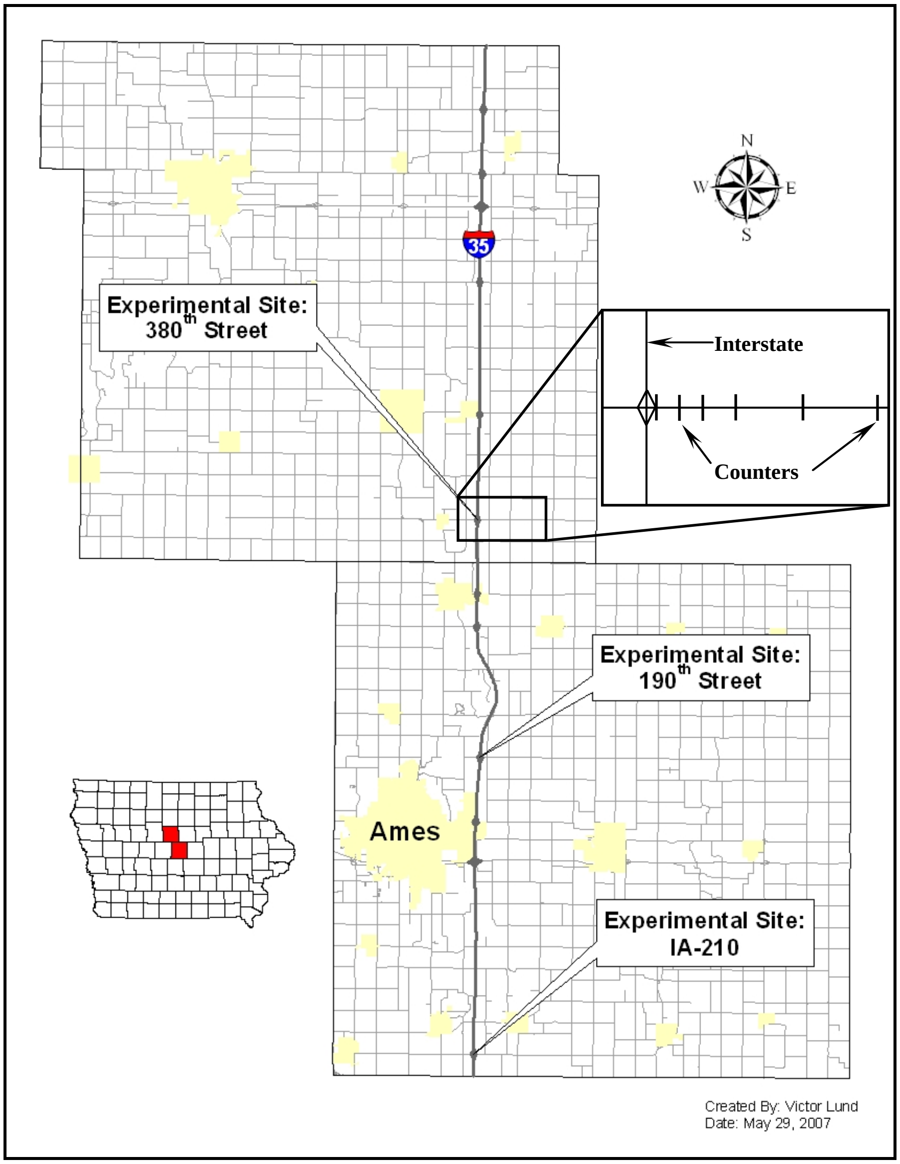

Figure 1. Speed adaptation sites

4. KEY FINDINGS

Table 1 displays a comparison of the before and after period monthly crash frequency means with a before period of July 2003 to December 2004 and an after period of July 2005 to December 2006. For each road type, a two-sample t-test assuming unequal variances was conducted to determine if the change in the crash frequency mean was statistically significant at the 95% confidence level. The results of a similar t-test are also displayed in Table 1 for average and 85th percentile speeds for each road type. A plus (+) sign indicates a statistically significant increase, while a negative (-) sign indicates a statistically significant decrease. The recorded monthly crashes are frequency means, and are not rate-adjusted for traffic volume. However, only small changes were observed in the traffic volume between the before and after periods.

Table 1. Summary of before and after period average monthly crash frequencies

Road Type	Crash Severity	Before Period Crash Frequency	Before Period Monthly Mean	After Period Crash Frequency	After Period Monthly Mean	Percent Change	Crash P-Value (one-tail)	Crash Significance (α = 0.05)	Average Speed Significance (α = 0.05)	85th Percentile Speed Significance (α = 0.05)
Rural Interstate	Fatal	29	1.61	40	2.22	37.9%	0.069	close		
	Fatal and Major Injury	117	6.50	138	7.67	18.0%	0.070	close	+	+
	All	2811	156.17	2940	163.33	4.6%	0.363			
Rural Interstate Daytime	Fatal	19	1.06	21	1.17	10.4%	0.376			
	Fatal and Major Injury	70	3.89	75	4.17	7.2%	0.374		+	+
	All	1299	72.17	1325	73.61	2.0%	0.431			
Rural Interstate Nighttime	Fatal	10	0.56	19	1.06	89.3%	0.087			
	Fatal and Major Injury	47	2.61	63	3.50	34.1%	0.143		+	+
	All	1512	84.00	1615	89.72	6.8%	0.373			
Urban Interstate	Fatal	15	0.83	16	0.89	6.8%	0.428			
	Fatal and Major Injury	93	5.17	89	4.94	-4.4%	0.383			
	All	2685	149.17	2346	130.33	-12.6%	0.106	close		-
Rural Expressway	Fatal	51	2.83	42	2.33	-17.7%	0.156			
	Fatal and Major Injury	183	10.17	178	9.89	-2.8%	0.411		-	-
	All	4365	242.50	4032	224.00	-7.6%	0.277			
Rural Other Primary	Fatal	105	5.83	140	7.78	33.4%	0.037	+		
	Fatal and Major Injury	479	26.61	490	27.22	2.3%	0.412		-	-
	All	8814	489.67	8620	478.89	-2.2%	0.421			
Rural Non-Primary	Fatal	227	12.61	218	12.11	-4.0%	0.365			
	Fatal and Major Injury	1046	58.11	981	54.50	-6.2%	0.162			
	All	15189	843.83	14801	822.28	-2.6%	0.359			
All Rural	Fatal	412	22.89	440	24.44	6.8%	0.233			
	Fatal and Major Injury	1825	101.39	1787	99.28	-2.1%	0.358			
	All	31179	1732.17	30393	1688.50	-2.5%	0.391			

4.1. Before and After Study Results

Overall, rural interstates experienced an increase in the higher severity crashes, such as fatal and fatal and major injury crashes. Table 1 shows that rural interstates experienced an increase in each crash severity, with fatal crashes increasing the most at 38%. Rural interstate nighttime fatal crashes increased by 89%. Although the change in the rural interstate monthly crash frequency mean was not statistically significant, the reported p-values were relatively low.

The diurnal effect of speed limit change was also investigated. Figure 2 displays the average daytime and nighttime speeds on the rural interstate for the period of August 2004 to December 2006. Both the daytime and nighttime speeds are observed to be slowly increasing after the speed limit change.

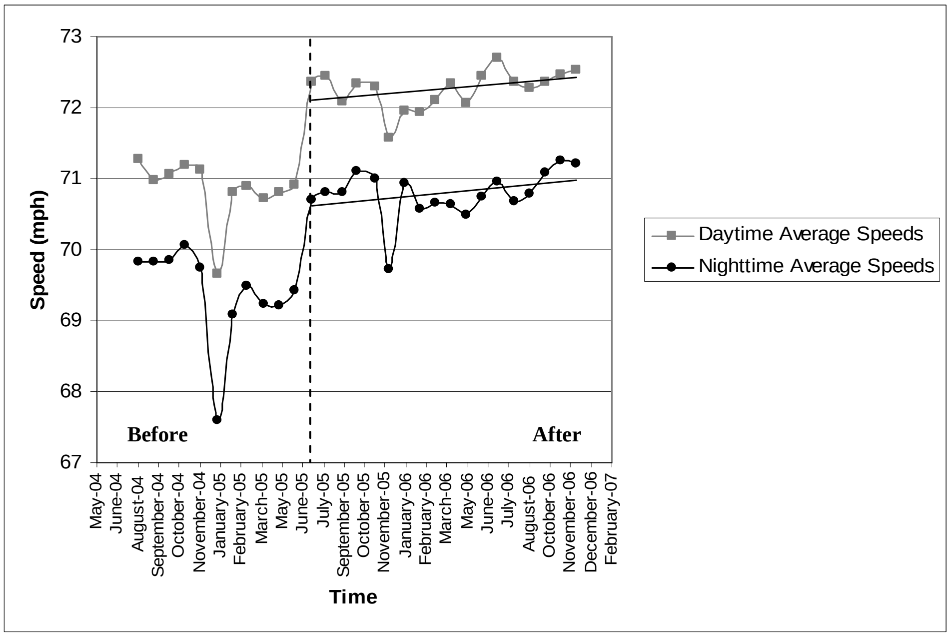

Figure 2. Rural interstate daytime and nighttime average speeds

A crash trend analysis was also completed for the rural interstates. Crash frequencies, adjusted for volume, were plotted for one-year periods. Each data point represents the one-year running average. Trend lines were fit to the before period data and extrapolated through the after period data. The difference between the expected mean crash frequency and the observed mean crash frequency in the after period was calculated and is summarized in Table 2.

Table 2. Rural interstate crash trend analysis

Crash Severity	Period	Observed Average Frequency	Expected Average Frequency	Difference
Fatal	before	24	24	0
	after	30	25	5
Fatal and Major Injury	before	93	93	0
	after	112	107	5
All	before	2217	2210	7
	after	2232	2175	57

4.2. Spillover Effect Results

Rural other primary highways have experienced an increase in fatal crash frequency mean relative to all other road types (33%), which was statistically significant. Both rural other primary highways and urban interstates experienced a similar change in higher severity crashes. However, rural expressways and rural non-primary roads experienced a decrease for all crash severities.

Interestingly, the increase in the monthly crash frequency means for rural other primary highway fatal crashes were statistically significant, while at the same time, a decrease in average and 85th percentile speed means were also reported as statistically significant.

4.3. Traffic Citation Results

Figure 3 displays the calculated ratio of electronic speeding citations to all electronic citations. Immediately following the change in the rural interstate speed limit, the months of July, October, and November of 2005 recorded a relatively higher ratio, which could indicate a short-term increase in the number of speed-related crashes or an increase in enforcement. Overall, there is no discernable change in the ratio of electronic speeding citations to all electronic citations on the rural interstate between the before and after periods. It is interesting to note that, very recently, the Iowa State Patrol has indicated that they will again step up enforcement due to increased numbers of crashes on the Iowa interstates.

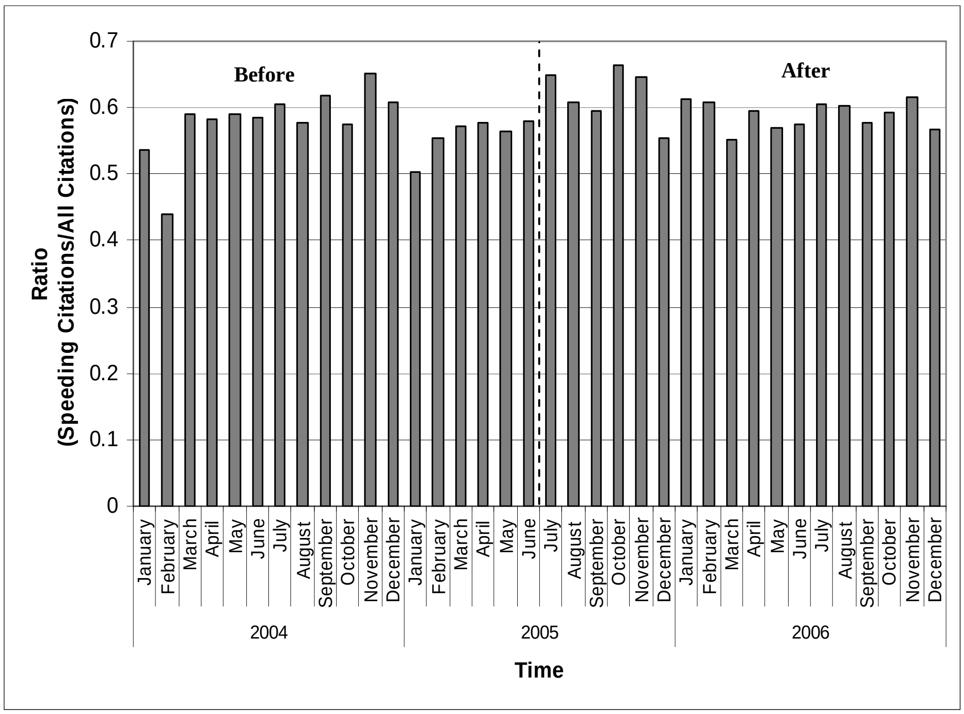

Figure 3. Ratio of rural interstate electronic speeding citations to all rural interstate electronic citations

4.4. Effect of Retail Gasoline Price Change

Because fuel prices both affect and are affected by travel demand, this relationship was examined as part of this study. Figure 4 displays the ratios of observed average daily traffic (ADT) and price in each month in 2006 to their corresponding data for 2005. As shown, the ADT ratio hovered around 1.0 while the gasoline price ratio remained mostly above 1.0. It may be concluded that, at least for Iowa in this two-year period, higher gasoline prices did not have a significant effect on rural interstate ADT.

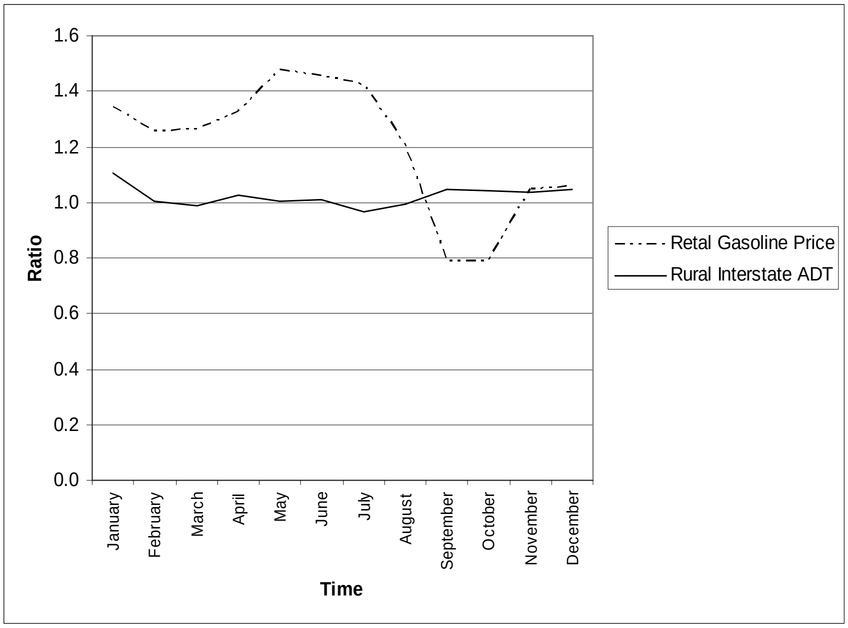

Figure 4. Ratio of rural interstate ADT to retail gasoline price in Iowa

4.5. Speed Adaptation Results

The results for all three speed adaptation study sites indicated that there was no significant difference between experimental vehicles and control vehicles' speeds for all vehicles, passenger vehicles, and those observed during the p.m. period. In fact, most of the observed experimental vehicles' mean speeds were less than the control vehicles' mean speeds. These results indicate there is little or no speed adaptation effect on those drivers who had exited the interstate onto a 55-mph rural primary highway. One of the concerns related to increasing the rural interstate speed limit was whether drivers would continue to drive at higher speeds than they normally would have upon entering non-interstate roads. Because the speed adaptation effect is not observed to exist in drivers who exit the rural interstate onto non-interstate rural primary highways, as the results suggest, a 15-mph speed limit differential is not observed to induce this effect.

5. CONCLUSIONS

Although more data (over time) would be required for statistical significance, safety performance of the rural interstate system in Iowa appears to have declined in the period following the 2005 speed limit increase. As shown in Table 1, the percentage changes in fatal crashes and in fatal and major injury crashes on the rural interstate were greater than for all crashes. The increase in more severe crashes also suggests that crashes may have become more severe as a result of a higher speed limit, especially at night.

Following the rural interstate speed limit change, daytime and nighttime average speeds are observed to be increasing over time. Comparing before and after periods, both average and 85th percentile speeds significantly increased at the 0.05 level.

No speed spillover effect was observed from rural interstates to other roads. Additionally, it was observed that there was no shift in traffic from other primary roads to the rural interstate.

The ratio of rural interstate electronic speeding citations to all rural interstate electronic citations did not seem to increase in the long run in the after period. Also, higher gasoline prices did not appear to have an effect on the amount of travel.

The speed adaptation study found no evidence of any significant differences in speeds of those vehicles exiting the interstate and those on the intersecting highway. Because there was little difference in the experimental and control vehicles' speeds, it is concluded that the speed adaptation effect is not significant on rural primary highways in close proximity to the interstates in Iowa. Although not observed in the present study, the speed adaptation effect may occur in more urbanized environments or in different types of rural facilities or locations.

ACKNOWLEDGMENTS

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Evaluation of Virtual Reality Snowplow Simulator Training

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ABSTRACT

Each winter, Iowa Department of Transportation (Iowa DOT) maintenance operators are responsible for plowing snow off roads in Iowa. Drivers typically work long shifts under treacherous conditions. In addition to properly navigating the vehicle, drivers are required to operate several plowing mechanisms simultaneously, such as plow controls and salt spreaders. However, there is little opportunity for practicing these skills in real-world situations. A virtual reality training program would provide operators with the opportunity to practice these skills under realistic yet safe conditions, as well as provide basic training to novice or less experienced operators.

To provide such training, the Iowa DOT purchased a snowplow simulator and commissioned a study through Iowa State University designed to (1) assess the use of this simulator as a training tool and (2) examine personality and other characteristics associated with being an experienced snowplow operator.

The results of this study suggest that Iowa DOT operators of all ages and levels of experience enjoyed, and seemed to benefit from, virtual reality snowplow simulator training. Simulator sickness ratings were relatively low, implying that the simulator is appropriate for training a wide range of operators. Many participants also reported that simulator training was the most useful aspect of training for them. Finally, performance measures observed in the simulator and operators' scanning behaviors recorded by head and eye tracking equipment suggest that trained operators performed better in the simulator than untrained operators. Overall, virtual reality simulator training appears to be a useful method for instructing snowplow operators.

Key words: driving simulator—snowplow—training—virtual reality

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INTRODUCTION

Each winter, Iowa Department of Transportation (Iowa DOT) maintenance operators are responsible for plowing snow off federal and state roads in Iowa. Drivers typically work long shifts under treacherous conditions. In addition to properly navigating the vehicle, drivers are required to operate several plowing mechanisms simultaneously, such as plow controls and salt spreaders. Furthermore, there is little opportunity for practicing these skills in real-world situations. During snowfalls, when training would be most realistic and effective, all available vehicles, drivers, and potential instructors are required to plow the roadways. Consequently, novice operators often do not undergo as comprehensive a training regimen as desired, and experienced operators do not have opportunities to improve their current practices or test new ones. Additionally, conducting novice snowplow operator training on roadways may present an unnecessary hazard for the trainee as well as other drivers.

Virtual reality training is an option when real-world training would be prohibitively high-priced, inappropriate, or hazardous. For example, because simulators provide a safe yet realistic environment in which students can be taught to operate aircraft, practice in flight simulators is a basic requirement of pilot training programs. A similar training program for snowplow operators would provide experienced operators with a chance to practice their skills under realistic yet safe conditions and would supply basic training to novice or less experienced operators. Training could be conducted during any time of year, but would be especially beneficial during the summer when vehicles and drivers are not engaged elsewhere.

This report is based on a collaborative effort between Iowa State University researchers and the Iowa DOT to evaluate the effectiveness of training operators with a TransSim VS III snowplow and truck driving simulator recently purchased by the Iowa DOT.

Several studies have shown that individuals tend to behave similarly in driving simulators as they do in real life. For instance, people drive at a similar speed without feedback from a speedometer (Panerai et al. 2001; Tornros 1998), select a similar lane position (Tornros 1998), and react similarly to stop signs and road markings (Godley, Triggs, and Fildes 2002). Thus, people seem to react to driving simulators and simulated environments as if they were on a real roadway. Consequently, driving simulators have often been used as a surrogate for real vehicles when using real vehicles would be prohibitively expensive or dangerous. For example, Godley, Triggs and Fildes (2004) examined the driving behaviors of people traveling on alternating lane and lane marking widths in a simulator, such that wide lanes could be paired with narrow and wide lane markings, and narrow lanes could also be paired with narrow and wide lane markings. Greenberg et al. (2003) used a driving simulator to investigate whether cell phone use impaired adults' and teenagers' driving performance. In both of these examples, replicating the experiment on a real roadway would be expensive or potentially dangerous, both for the participants in the study and for other motorists.

Recently, some studies have specifically examined the effectiveness of snowplow simulation training. Strayers, Drews and Burns (2004), in collaboration with the Utah Department of Transportation (UDOT), compared the performance on real highways of a group of operators who received snowplow simulator training to a matched group who did not. The authors found several indications that trained operators performed better than untrained operators. Specifically, trained operators experienced fewer accidents than untrained operators and recorded better fuel efficiency. Additionally, researchers from Arizona State University and the Arizona Department of Transportation (Kihl et al. 2006) administered snowplow simulator training over several years to operators from select districts while getting feedback on what operators thought were the most effective aspects of training.

In the present study, the researchers were interested in whether virtual reality snowplow simulator training, when combined with computer-based and traditional classroom training, would improve the performance of Iowa DOT snowplow operators. The behaviors of a group of both trained and untrained operators were compared in the simulator using various performance measurements, such as number of collisions, average speed, and fuel efficiency. Additionally, head and eye movements were monitored to determine whether simulation training would lead less experienced operators to behave more like highly experienced operators in terms of where they looked while driving. Finally, all operators completed a set of personality questionnaires designed to measure whether there were any personality variables associated with being a highly experienced snowplow operator and a set of simulator realism questionnaires designed to determine whether the simulator successfully mimicked the experience of driving a real snowplow. This approach of analyzing behaviors within the simulator allowed the researchers to avoid problems with collecting field performance data while still empirically investigating snowplow simulator training effectiveness, as reported by Strayer et al. (2004) and Kihl et al. (2006), and the approach made it possible to safely monitor eye movements.

METHOD

Participants

Two hundred full-time Iowa DOT snowplow operators were randomly selected from a list of 1,098 current maintenance workers, with a representative number chosen from the six districts in Iowa, to participate in this study. Half of the operators were randomly chosen to be in the control group, and the other half comprised the experimental group. These groups were matched in terms of age, district, number of accidents in the past five years and approximate amount of snowplowing and truck driving experience. Operators were informed that they would be receiving mandatory training, but that they had the option of participating in the study to assess training effectiveness. Overall, 174 operators, 84 in the experimental and 90 in the control group, participated in the study.

Apparatus

Participants received training in the TranSim VS III truck and snowplow simulator (see Figure 1). The VS III has a 180 degree horizontal and 37 degree vertical viewing area at 34 inches from the center screen, and is comprised of three 1024 x 768 monitors with a refresh rate of 70 Hz. The driving apparatus is similar to what would be found in a typical truck: functional brake, clutch, and accelerator pedals; radio; transmission; and digit gauges, all of which can be tailored to the requirements of the trainer. Although a clutch pedal was present, all simulations used an automatic transmission. The simulator also outputs digital sound designed to mimic normal operating sounds, such as engine and exterior noise. Tactile transducers under the drivers' seat provided simulated road vibration. Finally, two graphical rear view mirrors were displayed on the bottom corners of the central screen (spot mirrors) along with two graphical adjustable side mirrors on the outer half of the left and right screens (wing mirrors).

Three weeks into the study, certain parts of the driving simulator were upgraded: more snow was added to the edges of the windshield, and a few additional cars were added to the test scenario. Also as part of the upgrade, some of the tests for the performance measures were changed or excluded. Unless otherwise stated, the performance statistics reported only include the 136 participants who completed the training after the upgrade.

In order to track the participants' eyes and head movements, cameras were mounted on the very top of the simulator, approximately six inches above the central monitor, so as not to interfere with participants' performance in the simulator. The tracking was accomplished by running FaceLab (version 3.2) software. Head and eye movements were recorded only for the data collection scenarios.

Figure 1. Photograph of the simulator

Procedure

Training was conducted primarily at the main District 1 facility in Ames, Iowa. Typically, participants were trained in groups of two in a session that lasted for approximately four hours. Participants were first introduced to the trainer, who was an experienced Iowa DOT snowplow operator, and shown the simulator. The trainer drove a three-minute scenario down a sparsely populated rural road to acquaint the participants with the visuals and with the auditory feedback. Operators who chose to participate then drove an introductory three-minute scenario through a snowy highway to become acclimated to the controls and handling of the simulator.

Immediately after the orientation drive, participants completed their first set of questionnaires. This set included the NEO Five-Factor Inventory, which is based on the Five-Factor Model developed by Costa and McCrae (1985); a modified version of Zuckerman's (1979) Sensation Seeking Scale (form V); and Witmer and Singer's (1998) immersive tendencies questionnaire.

While participants were completing these questionnaires, they took turns driving the first data collection scenario. The scenario involved merging onto an interstate in snowy conditions and plowing snow for approximately 10 minutes. The simulated vehicle had a right plow and right wing, although participants did not control this equipment. At various times during the scenario, participants had to pass slow moving vehicles in the right lane, while avoiding striking them with the right wing, and then merge back into the right lane to avoid faster moving vehicles approaching from the rear. Participants were instructed to operate the simulator in the same manner that they would operate a snowplow while removing snow on a real highway.

After completing the first set of questionnaires and the first data collection scenario, operators assigned to the control group immediately completed the second data collection scenario. The second scenario was identical to the first. Following completion of the second drive, individuals in the control group began their training. Participants in the experimental group received training before completing their second data collection scenario. Thus, the control group performed both data collection scenarios prior to training, while the experimental group performed one scenario prior to training and one after training.

The training consisted of three parts: a lecture that included PowerPoint slides, a computer exercise, and a simulator exercise. The trainer began by giving a 20- to 25-minute lecture, which included a PowerPoint presentation. The lecture focused on the importance of being aware of the space around the vehicle and included a discussion of the Scan-Identify-Predict-Decide-Execute (S.I.P.D.E.) method. After the PowerPoint lecture, each participant completed a 5- to 10-minute driving scenario in the simulator, during which he or she was encouraged to employ the techniques learned during the lecture. Next, participants completed a self-paced computer exercise in which they watched short video clips that contained information about passing vehicles, speed management, and space management. Finally, participants drove an additional 5- to 10-minute scenario in which they were again instructed to employ the techniques that they learned during the PowerPoint lecture and the computer exercise. Training concluded with the trainer giving a 5- to 10-minute summary of the information covered during training.

Once training ended, participants in the control group completed their second set of questionnaires. Participants in the experimental group drove the second data collection scenario before completing the questionnaires. The questionnaires included a modified version of Witmer and Singer's (1998) presence questionnaire, Kennedy et al.'s (1993) Simulator Sickness Questionnaire (SSQ), and a modified version of the questionnaire used by Strayer et al. (2004) for the UDOT snowplow study.

RESULTS

In addition to contrasting performance between the experimental and control groups, the researchers also took operator expertise into account in examining the results. Specifically, participants were divided into three groups based on their amount of snowplow experience in the field: low experience (0-5 years), medium experience (6-15 years), and high experience (16+ years). These ranges were recommended by the Iowa DOT.

For a more in depth analysis of these results, see Masciocchi, Dark, and Parkhurst (2007).

Pre-Training Questionnaires

The personality questionnaires showed few differences among the experience groups. In terms of the effectiveness of simulator training, the most important result from this set of questionnaires concerns the immersion questionnaire (Witmer and Singer 1998). Immersion is defined as the ability to become enveloped or involved in an environment or task. People with high immersive tendency scores, therefore, would be hypothesized to become particularly involved in simulator training. The scores for operators in the low, medium, and high experience groups were compared to determine whether operators in one group may show lower mean scores than the others, and thus potentially not benefit as much from simulator training. The mean score ($M = 93.5$) indicated a moderate level of immersion, with no differences among the experience groups. Although one must be cautious when interpreting the lack of a difference, the means suggest that older, highly experienced operators will be as likely to become immersed in simulator training as younger, less experienced operators. To the extent that immersion is a necessary prerequisite to successful simulator training, then, experienced operators should have the potential to benefit as much from training as less experienced operators.

Post-Training Questionnaires

This set of questionnaires consisted of the presence questionnaire (Witmer and Singer 1998), the SSQ (Kennedy et al. 1993), and the quality of training questionnaire based on Strayer et al. (2004). Witmer and Singer (1998) defined presence as “the subjective experience of being in one place or environment, even when one is physically situated in another” (p. 225). In this case, participants were physically located in the training room, but they may have subjectively felt like they were driving a snowplow on a road. We had similar concerns regarding presence that we did with immersion, specifically, that some group of operators, based on experience operating a real snowplow, may have felt less presence, and consequently benefited less from simulator training. Once again, however, operators from the low, medium, and high experience groups had very similar presence scores, suggesting that they should feel similar amounts of presence in the simulator, and potentially benefit similarly from training in the simulator. The mean score (M =142.0) indicated a moderate to high level of presence.

Another encouraging result came from responses to the sickness questionnaire. Participants completed Kennedy et al.’s (1993) SSQ, which contains 16 questions concerning various symptoms of simulator sickness that users of virtual reality equipment may experience. Participants were asked to respond on a scale from 0 (“none”) to 3 (“severe”), indicating the extent to which they experienced each symptom. Simulator sickness scores were low overall, with mean scores less than what would correspond to a 1 (“slight”) for the 16 symptoms. Moreover, only 5 out of 174 participants were unable to complete the simulator training due to excessive simulator sickness. Thus, it does not appear that simulator sickness is an obstacle to the use of this simulator in training Iowa DOT operators.

Finally, results from the quality of training questionnaire showed that participants seemed to enjoy training. Questions examined simulator training, computer-based training, the lecture, the trainer, and whether training was valuable. Participants responded to these questions on a scale from 1 to 7. Approximately 80% of participants gave a score of neutral or above for the simulator training portion of training, suggesting that most participants enjoyed the simulator aspect of training. Participants were then asked to complete a series of simulator realism questions, once again using a scale from 1 to 7, which asked about the realism of various aspects of the simulator, as well as several free-response questions concerning the quality of the simulator training. Realism scores were moderately high (average score approximately 5), and operators gave several suggestions in the free-response section about how to better simulate real driving conditions.

Performance in the Simulator

The number of accidents in which participants were involved during the first and second data collection drives in the simulator was examined. Additionally, participants’ data were examined based on their real-world experience level (see Figure 2). Operators in the experimental group had more collisions overall than control group operators, but this difference was present on the first drive and did not interact with time. The most likely explanation is that pre-existing baseline differences produced this effect. More interesting was the finding that more highly experienced snowplow operators had fewer accidents than less experienced snowplow operators. Thus experienced drivers, who presumably had a longer time to refine their skills in the field prior to receiving simulator training, had fewer collisions in the driving simulator than less experienced operators, who had not had as much snowplowing experience. Such data confirm the assumption that simulator performance maps onto real-world performance. Finally, there was also some evidence that low-experience operators in the experimental group showed a greater improvement from their first data collection drive to their second than operators in the control group. This would suggest that training was particularly useful for these operators. However, baseline differences and difficulties in determining the true number of accidents in which some operators were involved complicate the interpretation of this finding. These analyses included data from operators who participated prior to the simulator upgrade.

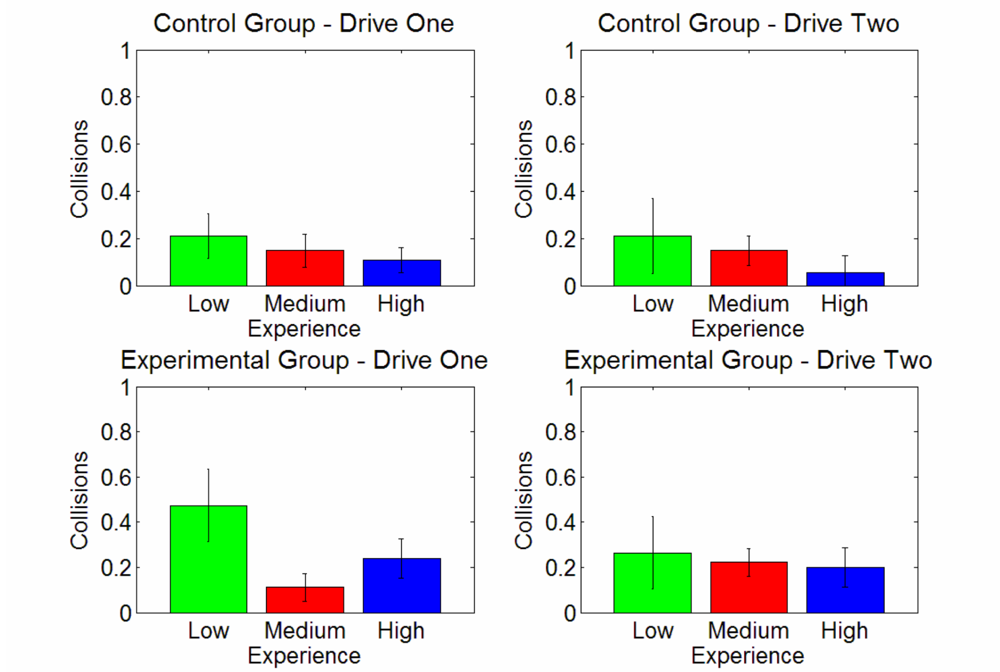

Figure 2. Mean number of collisions (with standard error bars) for the three experience groups during the first and second drive

Differences in average speed and fuel consumption were also examined. For these analyses, however, only those operators who participated after the simulator upgrade were included. Figure 3 shows the differences between the second and first data collection drives for operators in the control and experimental groups as a function of experience level. Overall, both groups drove more quickly in their second data collection drive than in their first, but this difference was approximately three times as large for operators in the experimental group. Moreover, this increase in speed came with only a small, but statistically reliable, decrease in fuel efficiency. In other words, operators who received training between their first and second data collection drives managed to increase their speed on their second drive, relative to their first, while showing only a minor decrease in fuel efficiency. Training that leads to operators covering more ground with a similar amount of fuel consumption may lead to overall lower costs.

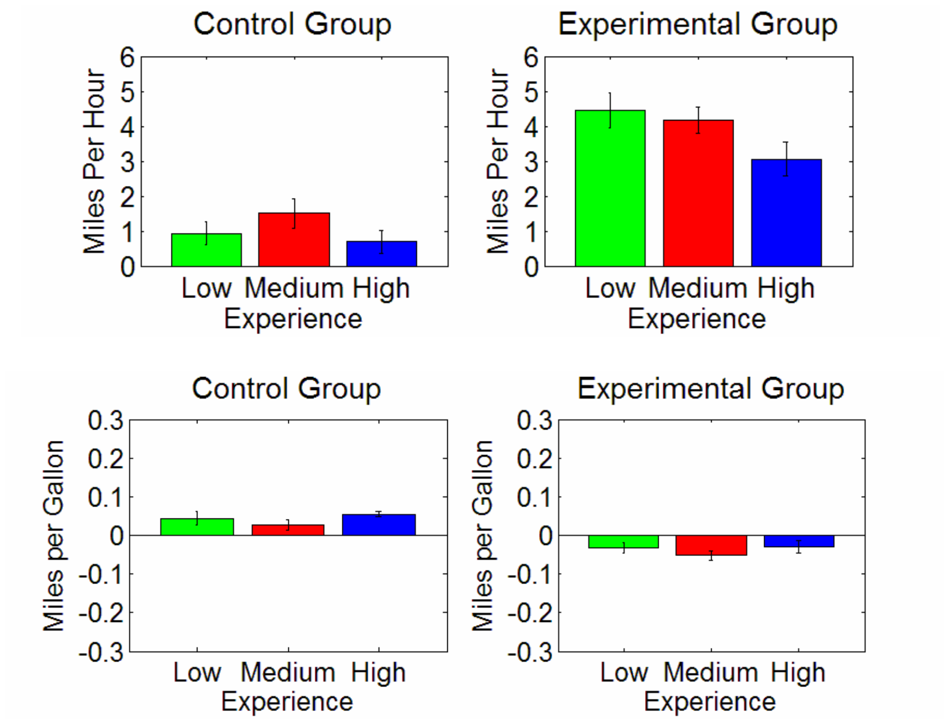

Figure 3. Mean differences with standard error bars for average speed (top) and fuel efficiency (bottom) between first and second drives for the three experience groups and for the control versus experimental groups

Fixation Location

A model of the interior of the simulator was created in order to calculate where participants were looking. Six discrete regions were identified, roughly corresponding to the left and right panels, the center portion of the center panel, the left and right sides of the center panel (i.e., left and right spot mirrors), and the speedometer (located below the center panel). Next, the researchers determined the amount of time that participants spent fixating on each of these six areas, once again comparing across experience, group, and drive number.

Overall, it was found that operators in the low-experience group who received training showed a change in fixation behavior from their first to second drives, which was not seen for operators in the control group. Specifically, the trained, low-experience operators spent more time looking at locations on the left side of the simulator (left panel, left spot mirror) and less time looking at locations on the right side (right panel, right spot mirror). Untrained, low-experience drivers did not show as great a tendency to focus on the left side of the vehicle. We believe this reflects a tendency for trained operators to use the center of their vehicle to judge the alignment of their right wing. In other words, instead of looking out the right side of their vehicle to determine whether their right wing was aligned approximately with the side lane marker, they were relying on the central lane marking to position their vehicle. According to experienced operators at the Iowa DOT, this is consistent with how operators are instructed to operators their snowplow in the field. The obvious advantage to this method is that it allows the operator to concentrate on the road in front of the snowplow, where the majority of potential hazards are likely to be located. Moreover, in the data collection scenarios, a frequent hazard came from fast moving vehicles passing the snowplow in the left lane. Thus, it may have been advantageous for operators to also spend additional time monitoring the left side of their vehicle, reflecting the trend for operators to look out the left side of their vehicle on their second drive. A separate measure showed that highly experienced operators blinked more than less experienced operators, which further suggests that the more experienced drivers did not need to concentrate as hard to drive the simulator.

DISCUSSION

The results of this study suggest that Iowa DOT operators enjoyed and seemed to benefit from virtual reality snowplow simulator training. Operators from all age groups and levels of experience reported having similar immersive tendencies in their everyday life and experienced a similar amount of presence within the simulator. One interpretation is that operators of all ages and levels of experience have a similar potential to benefit from training. The responses to the training questionnaire tend to support this explanation: Operators from all three levels of experience rated all aspects of training similarly. Additionally, although there was a lot of variation in reported simulator sickness scores, mean ratings were relatively low. As might be expected from the simulation literature, there was some evidence that amounts of simulator sickness increased with age (Arms and Cerney 2005); however, the mean ratings for all experience groups were less than what would constitute a response of "slight" for the listed symptoms. In future training sessions, when there is no experimental protocol, more time can be devoted to acclimating operators to the simulator. This likely will lead to even lower simulator sickness scores.

The performance data suggest that training was beneficial. Specifically, low-experience operators who received training before their second data collection drive showed a larger decrease in the number of collisions relative to their first drive than did low-experience operators who did not receive training before their second data collection drive. Difficulties in interpreting the performance files from the simulator software, however, make drawing conclusions from these results problematic. This finding was actually somewhat unexpected, considering that training only lasted for around 1.5 to 2 hours. Training programs for less experienced drivers that are able to devote more time to training, particularly within the driving simulator, should be more likely to produce measurable performance improvements.

Operators in the experimental group also showed a significant increase in their average speed in their second drive compared to their first drive, as well as compared to the second drive for operators in the control group. Importantly, this increase in speed came with a negligible increase in fuel consumption. Thus, this finding also shows that drivers who received training tended to perform better in the simulator than drivers who did not receive training before their second data collection drive.

The eye monitoring data showed that low-experience operators who received training showed a quantitatively different pattern of location fixation in the simulator than untrained operators. Specifically, low-experience operators in the experimental groups spent more time looking at the left panel and left spot mirror and less time looking at the right panel and right spot mirror on their second drive compared to their first. They may have concluded during the first drive that there was a benefit to looking at the left side of the simulator as opposed to the right, or they may have become more comfortable with the virtual reality environment and started behaving as they normally do when driving a snowplow.

The different variables measured in the current study indicate that drivers were behaving in the simulator as one may predict based on their amount of snowplowing experience. Less experienced operators showed a tendency to be involved in more accidents than more experienced operators, while experienced operators seemed not to concentrate as hard on driving in the simulator. Training also seemed to benefit less experienced operators in particular, and there is some evidence to show that all drivers who received training used better driving habits on their second data collection drive.

The major caveat of this study, however, is that the observed benefits of training reflect only the behaviors within the simulator. Although drivers who received training appeared to perform better on their second drive than those who had not had training at that point, it is unclear whether the benefits would transfer to real-world snowplowing behaviors. Previous research has shown that individuals tend to behave similarly in driving simulators as they do in real-world settings (Godley et al. 2002; Panerai et al. 2001; Tornros 1998), but this has never directly been tested for snowplowing. Another potential concern is that the benefits for the experimental group may simply reflect the fact that these operators had additional time to drive the simulator during training. We believe that this explanation is unlikely, however, because operators only drove two 5- to 10-minute scenarios during training, and the training scenarios involved truck driving rather than snowplow operating. Thus, the benefits seen as a result of training were not likely due solely to the fact that participants in the experimental group spent more time in the simulator. Information learned during the lecture and computer-based portions of training undoubtedly also contributed to the improvements in operators' performance.

Different types of follow up research are suggested by the current results. One possibility is that these results be replicated in a field analysis, either as part of a short-term study focused on fuel efficiency, or a longitudinal study investigating the number and severity of accidents across many years. Such a study would help to establish the relationship between snowplow simulator performance and real-world performance that has been established for driving. Although that certainly is a desirable goal, an alternative application of simulator technology may be to train drivers on a set of difficult maneuvers, such as plowing overpasses or highway exit/entrance ramps. This would allow for an easier, but still very useful, evaluation of simulator training effectiveness while focusing on tasks that may be difficult to practice in the real world. Overall, virtual reality simulator training seems to be an effective method of safely and inexpensively training Iowa DOT snowplow operators from all levels of experience.

ACKNOWLEDGMENTS

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Smart Data Collection: Using Custom Forms on a Handheld Smart Phone to Delineate Wetlands

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ABSTRACT

Wetland delineations require time consuming field work. The goal for this project is to automate the administrative documentation associated with routine wetland determinations. Specifically, the aim is to collect the appropriate data in the field, manipulate the data only once, and create reports and forms directly from field data with no additional transcription. Today, some scientists manually complete a draft of the routine wetland determination form in the field and later finalize it in the office prior to submission to the United States Army Corps of Engineers (USACE). The approach presented here automates this process using custom designed computer forms installed on handheld smart phones. Automated data collection tools allow personnel to quickly collect data in the field using software customized for the specific project. With automated data collection forms, scientists reduce the time spent in the office correcting sometimes hard to read field notes, minimize the time required to quality control field data, and eliminate the time spent retyping final reports from field notes. The custom form software uses streamlined data collection tools, auto-complete fields, lookup tables, and pick lists to populate a standard wetland database with the required information needed to complete the USACE forms. Once synchronized with the scientist's computer, the software is automatically integrated with reporting tools and geographic information systems software to create documentation and maps suitable for USACE review. The automated wetland forms are more accurate, reduce administrative project time by 20% (a conservative estimate), and produce a higher quality submittal through real-time field data validation.

Key words: automation—custom data collection forms—smart phone—technology—wetland delineation

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

PROBLEM STATEMENT

Wetland delineations require time-consuming field work. Field scientists must collect several pieces of ecological data to satisfy United States Army Corps of Engineers (USACE) wetland determination requirements. As the national approval authority for wetland determinations in the United States, all field work must be documented on USACE-approved forms derived from the 1987 *Wetlands Delineation Manual*. Today, some scientists manually complete a draft of the routine wetland determination form in the field and later retype the form in the office for submission to the USACE. The paperless approach presented here automates the determination process using custom designed forms installed on handheld smart phones.

OBJECTIVE

The goal for this project is to automate the administrative documentation associated with routine wetland determinations. Specifically, the project's aim is to collect the appropriate data in the field, validate the data onsite, manually enter the data only once, and create USACE-required reports and forms directly from field notes stored on a smart phone with no additional transcription or typing.

METHODOLOGY

Creative developers have designed custom applications to collect housing condition surveys (University of Oregon), data collection tools to generate Environmental Impact Statement documentation (Haas 2005), inspecting wastewater discharges (Zucchero and Sagami 2004) and electric power transmission lines (Southern Company 2004), and Phase I Environmental Site Assessments (Bell 2003). Environmental Systems Research Institute (ESRI) produced Figure 1 below to highlight the many possible applications for mobile data collection solutions. Practically, the number of applications for mobile data collection tools is endless.

Robust, inexpensive mobile data collection tools are not readily available for performing wetland determinations in Iowa. A flexible software application that is easily tailored to the unique soil types, vegetation, and hydrologic characteristics of Iowa is needed to automate the data collection efforts for wetland determinations in Iowa.

The software and hardware tools discussed in this paper are based on off-the-shelf technology, customized to the unique characteristics of Iowa's wetlands. There are two major components to this mobile data collection application: hardware and software.

The platform selected for this application is developed and sold by Hewlett-Packard. The device is the iPaq hw-6495 smart phone. This device is unique because it encapsulates an onboard global positioning system (GPS) receiver, a digital camera, a cellular phone capable of wireless data services, and an operating system compatible with Microsoft Windows.

Task	Industry			
	Government	Utility and Infrastructure	Environment	Public Safety
Field Mapping	• Recording Building Footprints • Right-of-Way Mapping • Based mapping	• Centerline Review and Mapping • Facility Mapping	• Forest Boundary Mapping • Trail Mapping • Geochemical Mapping • Volcanic Deposit Mapping • Wetlands Delineation	• 911 Address Mapping • Minefield Mapping • Military Fieldwork and Mapping
Asset Inventory	• Street Sign Inventory • Municipal Assets Inventory (GASB 34) • Tree Survey • Census Data Collection • Housing Condition Survey • Cemetery Inventory	• Recording Installations • Storm Water Inlet Inventory • Storage Tank Mapping	• Toxic Inventory • Mineral Exploration • Vegetation Survey • Wetland Survey • Archaeological Site Survey	• Aerial Survey • Fire Perimeter Mapping
Asset Maintenance	• Road Condition Survey • Streetlight Survey • Patient Registration	• Power Pole Maintenance • New Equipment Installation • Pavement Condition Assessment	• Crop Management • Vacant Land Condition Management • Timber Harvest Management • Drainage System Management	• Locating Buried Infrastructure • Recording Avalanche Observations • Facility Maintenance Survey
Inspections	• Road Pavement Management • Code Enforcement • Health Inspection • Housing Condition • Water Rights Enforcement	• Meter Reading • Septic System Inspection • Documentation • Compliance Monitoring • Dam Safety Inspection	• Habitat Studies • Weed Abatement • Well Sampling • Wildfire Sightings	• Damage Inspection • Tracking Violations • Street Sign Inspection • Flood Risk Assessment
Incident Reporting	• West Nile Virus Incidents • Public Nuisance Surveys	• Locating Outages • Regulatory Compliance	• Animal Migration Tracking • Oil Spill Assessment • Radioactive Contamination Tracking	• Property Damage Assessment • Accident Reporting

Figure 1. Possible applications for mobile data collection tools

Software installed on this device includes ArcPad 7.1 (ESRI 2007) and Pendragon Forms 5.0. Each software application is customized to reflect the unique soil types, vegetation, and hydrologic characteristics of Iowa.

According to Ishara Kotiah (2004), senior analyst with Spatial Vision, “ESRI’s ArcPad software is an easy-to-use, low-cost mobile mapping tool that can be integrated with a corporate geographic information system (GIS) deployment. ArcPad provides intuitive mapping, basic GIS query tools and GPS functionality for recording locations. ArcPad can make field data collection and verification fast, easy, and accurate. ArcPad can be deployed on a range of handheld PDA devices and tablet PC’s.”

Pendragon describes its Form 5.0 software as “the smart solution for today's mobile worker. Create custom forms that meet your business data collection needs. Take your forms with you on Pocket PC and collect data where you work. Get more accurate data, and get it faster. Data from handhelds is automatically transferred into Microsoft Access or your enterprise database each time you synchronize and without tedious and error-prone retyping.”

The Pendragon Forms 5.0 software works by creating a Microsoft Access database and associated tables for each custom form. The software provides two development scenarios. In the basic development environment, the user does not need to manually create the database relationships or field mappings. The forms software creates the database tables and automatically creates the links between the required data fields in the form and the underlying database. The database synchronization conduit is installed automatically during the installation process. The user simply synchronizes the handheld computer with the desktop or laptop computer. During the synchronization process, new records on the handheld are sent to the PC, and new or changed records on the PC are sent to the handheld. In the advanced development environment, more experienced users have the flexibility to synchronize a custom designed form to an external Microsoft Access database or other open database connectivity data source. The advanced form integration process is illustrated in Figure 2.

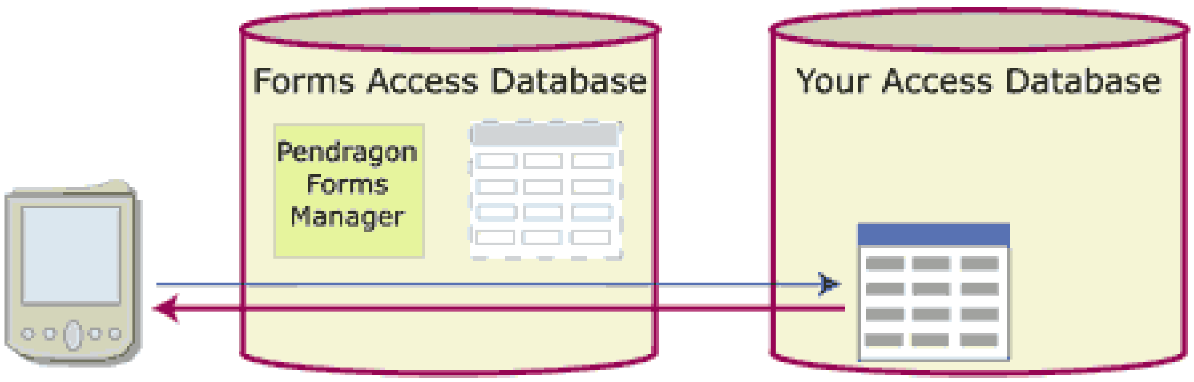

Figure 2. Form software integration with external data sources (http://www.pendragon-software.com/)

The process of converting paper forms into electronic ones is relatively simple using Pendragon Forms software. Each paper form is manually converted into a series of database tables and associated fields. The user has the ability to customize the look and feel of the form application to make it user-friendly and intelligent. The software intelligence is created by integrating a list of meaningful attribute domain choices in a pull-down menu to reduce the time needed for entering field data attributes. These standardized pick lists reduce data entry error and eliminate the need for interpreting muddy or sloppy field notes back at the office. Figure 3 illustrates the custom designed, menu-driven format of the wetland form application.

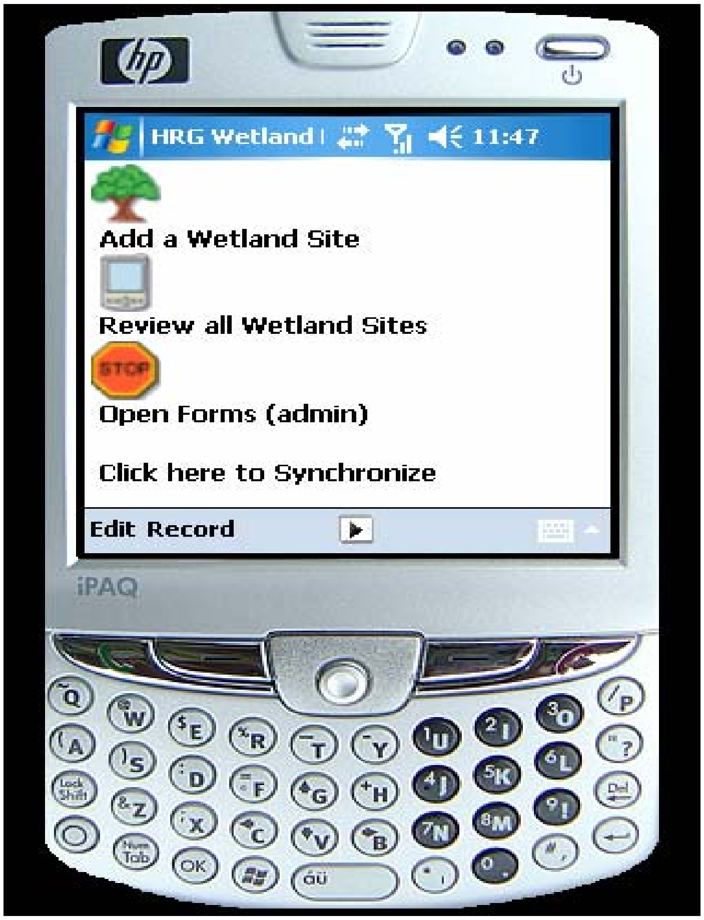

Figure 3. Wetland software screen shot

When the data conversion process is finished, the end result is a database that contains each question or required entry from the paper-based form. This electronic database is the basic building block for more advanced automation tools like custom designed reports or integration with GIS mapping applications that draw their input datasets from the underlying wetland form database. Figure 4 illustrates the final USACE Routine Wetland Determination form as an example of automated reporting. This custom designed reporting tool is built inside the Pendragon Forms database using standard Microsoft Access reporting tools. This reporting tool integrates the data collected on the smart phone's form software (and underlying database) into the appropriate layout and format for USACE submission.

With the appropriate data collected in the field, the user can develop numerous reports to pull information from the form database. The only prerequisite is the form must have a field in the database to store the information. Custom reports could link photos captured in the field of sensitive habitats, unique hydrologic conditions, or rare soil characteristics for later analysis in the office or for inclusion with the USACE wetland form submittal.

DATA FORM
ROUTINE WETLAND DETERMINATION
(1987 COE Wetlands Delineation Manual)

SITE DETAILS

Project/Site: Otter Creek North Bank	Date: 12/13/2006 3:31:26 PM
Applicant/Owner: City of Ankeny	County: Polk
Investigator: Ted McCaslin	State: IA

	Yes	No
Do Normal Circumstances Exist on the site?	☑	☐
Is the site Significantly Distrubed (atypical situation)?	☐	☑
Is the area a Potential Problem Area?	☐	☑
(If needed, explain on reverse.)		

Community ID: PEM
Transect ID: 1
Plot ID: 1

VEGETATION

Dominant Plant Species (and Indicator Status):	Stratum	Indicator	Notes:
Phalaris arundinacea L.	Herb.	FACW+	

Percent of Dominant Species that are OBL, FACW, or Fac (excluding FAC-): 100

Remarks: No visible vegetation problems noted

HYDROLOGY

☑ Recorded Data (Describe in Remarks):
- ☐ Stream, Lake, Or Tide Gauge
- ☑ Aerial Photos
- ☐ Other

☐ No Recorded Data Available

Field Observations

Depth of Surface Water: ______ (in.)
Depth of Free Water in Pit: ______ (in.)
Depth of Saturated Soil: ______ (in.)

Wetland Hydrology Indicators:

Primary Indicators:
- ☐ Inundated
- ☐ Saturated in Upper 12 Inches
- ☐ Water Marks
- ☐ Drift Lines
- ☑ Sediment Deposits
- ☐ Drainage Patterns in Wetlands

Secondary Indicators:
- ☐ Oxidized Root Channels in Upper 12 Inches
- ☐ Water-stained Leaves
- ☑ Local Soil Survey Data
- ☑ FAC-Neutral Test
- ☐ Other (Explain in Remarks)

Remarks: No hydrology problems noted

Form Template approved by HQ USACE 3/92
Forms version 1/02

Figure 4. USACE Routine Wetland Determination form

IMPLEMENTATION OBSTACLES

Although many benefits of customized data collection forms exist for routine wetland determinations, developing an automated data collection software solution is not without obstacles. The software development process requires a significant investment in time and in technology. Complex form development and integration with an existing database design could require between 70 and 100 hours designing, coding, and testing the application prior to deployment. The software and hardware investments could easily cost $2,000 or more depending on the user’s specific database performance and storage requirements.

CONCLUSIONS

The automated wetland forms are more accurate, reduce administrative project time by at least 20% (a conservative estimate), and produce a higher quality submittal through real-time field data validation. Other mobile data collection solutions have created similar results. According to a study completed by the Air Force, automated mobile data collection tools helped create a 90% reduction in the number of man-hours required to monitor solid waste infrastructure on a combat air base (Houston, Sinclair, and Robertson 2005). Similarly, Bell noted other observed strengths of a mobile data collection solution including the following:

- Reduction of time spent in field
- Reduction of time spent performing manual data input
- Reduction of time required to quality control the data
- Field data may be uploaded directly to a PC. This reduces the time necessary to transcribe handwritten field notes and also reduces error in transcription.
- Reduction of time spent to create final reports
- Custom forms may include validation that may allow or disallow data entry.

Mobile data collection using form software deployed to handheld smart phones reduces administrative documentation associated with routine wetland determinations and produces a higher quality submittal through real-time field data validation. With automated data collection forms, scientists reduce the time spent in the office correcting sometimes hard to read field notes, minimize the time required to quality control field data, and eliminate the time spent re-typing final reports from field notes. The flexible software application presented in this paper is easily tailored to local wetland conditions and is needed to automate the data collection efforts for wetland determinations in Iowa.

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Performance Measures for Snow and Ice Control Operations

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ABSTRACT

Under the NCHRP 06-17 project, the research team surveyed snow and ice control organizations in the United States, Canada, Europe, and Asia to determine the current trends in performance measurement. The team also inquired about the methods used in developing these programs in order to determine a practical, user friendly method to assist snow and ice control managers in developing a performance measurement system that uses traditional and nontraditional performance indicators and measurement issues. To achieve the project objectives, the researchers issued a survey to snow and ice control agencies throughout North America, Europe, and Asia to obtain data on the performance indicators and measures used, if any, by these agencies. The identified performance indicators and measures were then categorized, defined, and assessed for their usefulness. A process was then developed to assist snow and ice control operations managers in preparing a customer-focused, environmentally friendly performance measurement program.

Key words: performance measurement—snow and ice control—winter maintenance

INTRODUCTION

The issue of performance measurement for snow and ice control has been a topic of much interest. Developing meaningful data for snow and ice control has produced a variety of responses and differing goals and objectives. However, a rigorous process that the snow and ice control industry can use to determine the most appropriate performance measures and indicators has been lacking.

Research was needed to examine current trends and issues and develop a process that can be used by snow and ice control agencies to prepare a performance measurement system that is sensitive to organizational and public needs as well as environmental concerns.

The research would also analyze the different dimensions along which an agency's performance could be defined, measured, and interpreted based on an agency's goals and objectives.

Under the NCHRP 6-17 project, the research team surveyed snow and ice control organizations in the United States, Canada, Europe, and Asia, to determine the current trends in performance measurement. The team also inquired about the methods used in developing these programs in order to determine a practical, user friendly method to assist snow and ice control managers in developing a performance measurement system that uses traditional and nontraditional performance indicators and measures issues. The plan provides a list of options of performance indicators and measures and explains how to incorporate the indicators and measures in the decision making process to monitor and improve snow and ice control operations.

To achieve the project objectives, the researchers first reviewed pertinent literature and research findings in the area of performance measurement systems. Next, a survey was issued to snow and ice control agencies throughout North America, Asia, and Europe to obtain data on the performance indicators and measures used, if any, by these agencies. These performance indicators and measures were then categorized by functional type and were fully defined. An assessment of the usefulness of each was prepared. The research team then summarized the theory and practice of the performance measurement. The performance measures were then identified by their key aspects and identifying performance indicators and measures that may have applicability in snow and ice control operations. A process was then developed to assist snow and ice control operations managers in preparing a customer-focused environmentally friendly performance measurement program.

The purpose of this research is to provide a synthesis of measures used throughout the world to evaluate the performance of winter maintenance activities (snow and ice removal from roadways) and to make recommendations for further development of the most promising measures. The research was conducted in two parts. The first part entailed a comprehensive review of performance measures that have been and are currently being used by transportation agencies around the world. This was done through a thorough review of the literature and a survey of dozens of agencies with winter maintenance responsibilities. In the second part, the list of performance measures was narrowed to a few that offered the most promise. In other words, these were measures with the most potential to be applied economically to a roadway network and provide reliable, repeatable, and comparable measures of performance. These most promising measures were then recommended for further development.

PERFORMANCE MEASUREMENT

For many transportation agencies, performance measurement has become a critical issue in the last five to ten years, and several significant contributions to the literature have been made on the fundamentals of how transportation agencies should tie strategic direction and agency mission to performance measures.

Performance measurement is one component of a larger "quality in government services" movement. The growing emphasis on performance measurement by transportation agencies has not been addressed sufficiently because there wasn't a need to measure performance in the past but also due to two forces:

1. A culture at transportation agencies that has historically focused on standards and specification for physical conditions or level of service (LOS). Generally, transportation agencies have defined the LOS or conditions of a facility based on static standards.
2. The vast expansion of information technology and the ability to collect information that would have been too costly or impossible to collect in the past has made the collection of performance-related data possible

Measurement of Winter Maintenance Performance

Although winter maintenance is a critical activity, there are no standard methods for measuring performance for either agency programs or programs led by contractors. The lack of standard measures makes it difficult to manage and control winter maintenance activities and subsequently impossible to benchmark and make comparisons both between and within maintenance programs. Measuring the performance of winter maintenance makes it possible to make intelligent management tradeoffs between agency costs and user costs.

Agencies that currently measure winter maintenance performance do so from one or more of three basic perspectives:

- **Inputs.** Input measures represent the resources spent or utilized to perform snow and ice control operations. These include fuel usage, labor hours, machinery or equipment hours, and units of anti-icing materials or abrasives. The level of inputs is directly proportional to agency costs and, therefore, they most easily and most commonly are measured by transportation agencies. Because inputs are applied at the beginning of the winter maintenance process, they are unable to help management assess the efficiency, quality, and effectiveness of winter maintenance.
- **Outputs.** Outputs quantify the resulting physical accomplishment of work put forth in applying resources in winter maintenance. Outputs might include the lane miles plowed or sanded, the number of lane miles to which deicing materials were applied, lane miles of anti-icing brine applied, and other accomplishments of the maintenance process in units of work. Outputs are generally more useful than inputs alone because inputs and output together can help to define how technically efficient winter maintenance operations are performing. In other words, they can tell the winter maintenance manager what level of input was or will be required to achieve a level of output. These measures may also be based on time and storm event.
- **Outcomes.** Performance measures that seek to measure outcomes take into account the relative effectiveness of the winter maintenance activity, very often from the perspective of the user or customer. Outcomes are inherently more difficult to measure. A desired outcome of winter maintenance might include the improvement of safety, mobility, and/or user satisfaction. Safety, mobility, and user satisfaction are abstract concepts and, therefore, are measured through

indicators that are known to be related to the desired outcome. For example, safety might be measured through pavement friction or through the reduction in number of crashes.

Putting Winter Maintenance Performance Measurement into Context

To make comparisons between and among jurisdictions, differences in the severity of storms must also be taken into account. The severity of a storm impacts the performance of winter maintenance. To illustrate the relationship between inputs, outputs, outcomes, and the environment, a fishbone diagram is shown in Figure 1. The top of the figure shows some of the environmental inputs. On the bottom are labor, equipment, and materials inputs for removing snow and ice from the roadway network. In the arrow, the results of the interaction between the environmental variables and the inputs to snow and ice removal are shown.

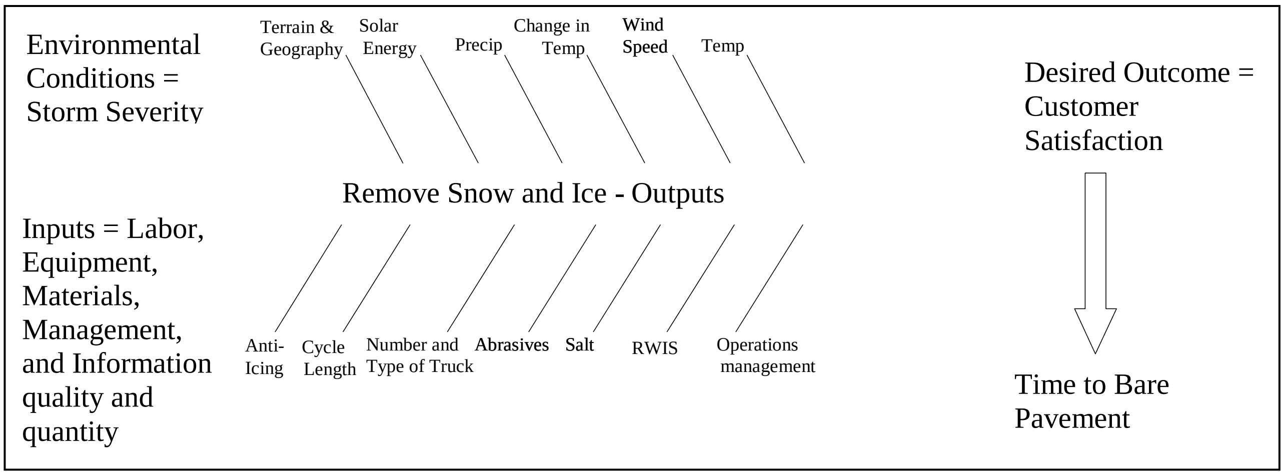

Figure 1. Relationship between inputs, outputs, and outcomes

In this case, we have identified satisfying the customer (the road users) as our desired outcome, and because shorter time to bare pavement is related to higher levels of satisfaction, time to bare pavement is the resulting performance measure. The measurement of time to bare pavement must be supported by a specific data collection methodology.

Summary of Synthesis Findings and Assessment

Various instances of research and testing of proposed performance measures were described in literature but often without implementation or field testing. It appeared that a handful of European countries and Japan are more progressive in terms of developing and implementing winter maintenance performance measures, likely because more snow and ice control operations are contracted to private companies internationally than in the United States.

The survey of winter maintenance personnel was sent to 162 agencies covering the U.S. Snow Belt states, Canadian provinces, northern Europe, and Japan. In all, 39 agencies responded to the survey, with responses covering agencies that did no snow and ice control performance measurement to those that incorporated performance measures into their management plans. Most performance measures cited by the respondents are tied to their accounting and management systems. These measures include lane miles plowed, personnel and/or overtime hours, tons of material used, amounts of equipment deployed, and cost of operations.

Other measures used by some of the respondents include time to bare pavement, time to return to a reasonably near normal condition, length of road closures, and customer satisfaction. The majority of the measures critical to the respondents’ snow and ice control operations focused on public safety and mobility.

In all, the survey analysis identified 4 input measures, 5 output measures, and 11 outcome measures used by public agencies to measure snow and ice control performance. A complete list of the performance measures identified can be found in the final report. To identify measures and approaches that warrant further study, the following criteria were applied to the measures and approaches:

Table 1. Measures and approaches

Measure criteria
Does the measure directly measure safety, mobility, or public satisfaction?
Does the measure improve snow and ice control?
Is the measure mapped to roadway segments?
Is the measure reported for garages or districts?
Is the measure sensitive to storm characteristics?
Approach criteria
Is the approach quantitative?
Is the approach stable across observers?
Is the technology used likely to improve?
Is a major capital or operational investment required?
Can the approach be piggybacked on another system to reduce installation cost?

As a result of the assessment, it was determined that outcome measures should be pursued further, because if the measurement of snow and ice control is to have a role in improving safety and mobility, measures of outcome must be pursued. To help determine the measures and approaches to pursue further, the 11 outcome measures observed in this study were reduced to three basic categories. Two approaches are possible for each measure.

Table 2. Approaches used per measure

Measure: Degree of clear pavement
Approach: Manual observation
Approach: Camera-assisted observation
Measure: Traffic flow
Approach: Detectors—speed, volume, and occupancy
Approach: Road closure
Measure: Crash risk
Approach: Friction (or slipperiness)
Approach: Reported Crashes

Summary of Key Points

The literature review revealed that a significant amount of published materials deal with different types of performance measures, both in use and theoretical. However, there was a limited amount of literature documenting agencies' utilization of performance measures in day-to-day practice. Various instances of research and testing of proposed performance measures were described, but often without implementation or field testing by state or local agencies in the United States. It appeared that a handful of European countries and Japan are more progressive in terms of developing and implementing winter maintenance performance measures, likely because more snow and ice control operations are contracted to private companies internationally than in the United States. From this review, the following was discovered:

- Three scanning review teams of U.S. officials, in 1994, 1998, and 2002, have visited Europe and Japan, focusing on winter maintenance activities and advanced intelligent transportation systems technologies.
- Performance measures can be divided into three general categories: input, output, and outcome measures
- Known input measures include labor hours, equipment hours, various material units, and monies expended.
- Known output measures include cost determined by a unit of accomplishment of work performed (e.g., lane miles plowed or sanded), material application rates, equipping and calibrating trucks, and route characteristics. These measures may also be based on time and storm event.
- Known outcome measures include bare pavement regain time, friction (skid resistance by coefficient of friction), reduction in crashes, duration and frequency of closures, advanced warning time to customers, and customer satisfaction (indicated by customer satisfaction surveys).
- A Pavement Snow and Ice Condition chart, used by some agencies, assists with uniform pavement condition identification by combining traffic flow characteristics and visual observation.
- Various outcome measures can and often are combined to form an overall LOS rating for a roadway.
- Contracts with private sector operators are often written such that reimbursement is based on a combination of input (pay items) and output or outcome measures (expectations).
- Innovative technologies installed on winter maintenance vehicles that aid in the collection of data applied to performance measures include AVL, GPS, friction meters, and various sensors of material, equipment, and temperature.
- Winter weather severity indices have been developed to help quantify the relationship between the severity of winter weather events and roadway condition or safety factors.

The study also provided an inventory and discussion of the winter maintenance performance measures used by states, provinces, cities, and counties. There is a broad range of participation in the uses of performance measurement, as stated by those surveyed. The range is from not measuring performance of snow and ice control at all to establishing sophisticated measures of performance of operations. The landscape of performance measurement is wide ranging. While many agencies stated the need for performance measurement, only a handful of these have established a formal performance measurement process for their operations. In these times of budget challenges, the agencies have focused their efforts on achieving the desired results of effective snow and ice control to meet the demands of the traveling public.

Survey Results

For this project, the study team sent out a survey to 162 winter maintenance operations personnel throughout the world. The targeted survey respondents were from local, state, and federal agencies. The respondents were chosen to provide feedback unique to their areas of expertise.

Of the 162 surveys distributed, 41 were returned, for a response rate of 24%. Within these 41, 20 states responded, as well as 4 Canadian provinces, 1 response from Europe, and 1 from Asia. The remaining respondents included those from cities and counties in the United States and Canada. Figure 3 and Table 1 illustrate the agencies responding to the survey, along with their locations. The surveys were distributed by electronic mail and through the postal service. While this response is not as high as we had hoped it to be, the surveys that were returned provide remarkable insight into the use of performance measures in winter maintenance operations, particularly in the northern hemisphere regions. The respondents were primarily from the United States and Canada.

The responses to the survey covered both ends of the spectrum, from those that did no performance measurement to those that incorporated performance measures into their management plans. Four agencies responded that they do not use performance measures at all, while 34 responded that performance measures were used in some capacity. One agency did not respond. There were also those that indicated that they would like to improve their methods to measure their performance for snow and ice control but weren't able to obtain the proper data. Clearly there is room for improvement in this area.

Most of performance measures cited by the respondents are tied to their accounting and management systems. These measures include lane miles plowed, personnel hours, overtime hours, tons of material used, amount of equipment deployed, and cost of operations. Other measures used by the respondents include time to bare pavement, time to return to a reasonably near normal condition, LOS, and customer satisfaction. Customer satisfaction was cited by 21 respondents as a performance measure. Additionally, 19 respondents indicated that public was surveyed periodically, either by the department or in a citywide survey. The surveys showed that the public was generally satisfied with their performance. Two respondents indicated that they measured customer satisfaction based on telephone calls or complaints.

The majority of the measures critical to the respondents' snow and ice control operations focused on public safety and mobility. Obviously, these subjects are central to the role of all transportation agencies, so it makes sense that the performance measures would focus on these subjects. By maintaining mobility and traffic flow, accidents are reduced and public safety is enhanced.

Both the state and local agencies are generally interested in providing the best service to the public. However, budget and staffing constraints make it difficult for agencies to experiment with new methods or technologies. The agencies want to be able to provide these services at the lowest possible costs. Therefore, the performance measures that are established cannot be too time consuming or costly to measure.

Eventually, more winter maintenance agencies will adopt more performance measurement practices. The public will continue to expect clear roads and less harm to the environment from snow ice control operations. Technologies such as AVL, GPS, friction meters, and RWIS, among others, hold the key to obtaining additional data to enhance measuring performance. Expanded use of these technologies will bring down the prices as production and competition increases. Both field personnel and management would have to train to focus more on outcomes when using these more costly technologies.

The objectives selected by each agency can drive performance measurement by creating targets toward which activities can be directed. In addition to objectives, performance measures need to include a short-term result, an improvement strategy, and accountable entities. In addition, success with performance measurement will rely upon the ability to create responsive data systems that generate timely data.

Performance measurement offers a promise of improved management and improved outcomes. It builds on a long history and extensive experience in techniques to strengthen and improve winter maintenance operations. As the winter maintenance community moves toward a future that includes performance measurement, program successes will follow.

SYNTHESIS AND ASSESSMENT

Based on the review of relevant literature and survey of agencies, more than 20 distinct performance measures were identified. For some of the measures, agencies used a variety of approaches to acquire the data to calculate the measures. Within this data set, more than 40 combinations were identified. Our approach was to categorize the various measures as input-, output-, or outcome-based and summarize their frequency of use.

Generally, the data for input and output measures come from the agencies' accounting systems or maintenance logs. There is not much variation in the approach to acquiring these data. For outcome measures, however, it is more difficult to obtain data, since the majority of outcome measures are based on some form of manual observation. However, some developing technologies in the experimental stages can provide innovative solutions to acquiring outcome measure data.

Additionally, any measure used for time-series analysis would benefit from applying a storm severity index. There is no shortage of options in the literature. The various indices were evaluated based on the availability of data to calculate the index and its usefulness in improving understanding of performance or communicating performance to administrators.

To provide direction for this synthesis and assessment, the study team developed criteria for evaluating measures and the associated approaches to acquiring data. These criteria were applied to screen out measures or approaches that do not exhibit the following characteristics:

- Related to controllable facets of performance
- Reliable
- Understandable
- Timely
- Consistent
- Sensitive to data collection costs

Table 4. Summary of snow and ice control performance measures by category

Input measures
Fuel usage
Overtime hours
Personnel hours
Percent of salt spreaders calibrated
Output measures
Lane miles plowed
Tons of material used
Amount of equipment deployed
Plow-down miles traveled
Cost per lane mile (efficiency)
Outcome measures
Time to bare pavement
Time to wet pavement
Time to return to a reasonably near-normal winter condition
Time for traffic volume to return to "normal" after the storm
Time to provide 1 wheel track
Friction
Level of service
Travel Speed during storm
Customer satisfaction
Crashes per vehicle mile
Traffic volume during storm

Screening of Approaches

The survey analysis identified 4 input measures, 5 output measures, and 11 outcome measures used by public agencies to measure snow and ice control performance. To identify measures and approaches that warrant further study, the following criteria were applied to the measures and approaches:

Measure criteria

- Does the measure directly measure safety, mobility, or public satisfaction?
- Does the measure improve snow and ice control?
- Is the measure mapped to roadway segments?
- Is the measure reported for garages or districts?
- Is the measure sensitive to storm characteristics?

Approach criteria

- Is the approach quantitative?
- Is the approach stable across observers?
- Is the technology likely to improve?
- Is a major capital or operational investment required?
- Can the approach be "piggy backed" on another system to reduce installation cost?

Applying these criteria revealed that input and output measures are valuable management tools because they measure the amount of material, labor, and money consumed, as well as the amount of material applied to roads, lane miles plowed, etc. However, these measures do not directly address the goals of the agencies, which all speak to public safety and maintenance of mobility. As they are, input and output measures help with budgeting and can be used roughly to compare efficiency between garages or districts that experience similar snow and traffic conditions. However, as far as the survey could determine, the measures do not improve snow and ice control but rather track the investment required to do so. The measures are generally not mapped to roadway segments, although they are often reported by garage or by district. Input and output measures are not observed to be sensitive to storm characteristics, although they could be if an index were applied.

Summary of Approaches

In summary, the study team recommends the following:

- Document best practices for manual observation of pavement conditions. Manual observation will clearly be the dominant approach to acquiring winter condition data for a long time, and best practices should, therefore, be shared.
- Document the use of traffic control center cameras or remote cameras to aid manual observation inputs to performance measures.
- Strongly pursue detector-based approaches that use traffic speed, volume, or occupancy as means of acquiring data measuring performance. Also pursue institutional issues regarding data use, technological opportunities, and technological barriers.
- Document measures that are or can be based on friction. (Friction measuring technology will not be evaluated.)
- Document best practices and opportunities for recording and analyzing crash data during winter storms for use as a performance measure.
- From the 15 storm severity indices found in the literature, recommend a reasonable procedure for incorporating an index to normalize input, output, and outcome measures.
- Determine best practices in the measurement of customer satisfaction and link those measures to measures of operational performance.

What Performance Measures Do for an Organization

The accounting firm of Price Waterhouse (Artley and Stroh 2001) has offered three main reasons for establishing metrics in an organization, listed below. These reasons can also be applied to snow and ice control operations.

1. **Measurement clarifies and focuses long-term goals and strategic objectives.** Performance measurement involves comparing actual performance against expectations and setting up targets by which progress toward objectives can be measured.
2. **Measurement provides performance information to stakeholders**. Performance measures are the most effective method for communicating about the success of programs and services. For example, in public education, states and school districts routinely issue report cards highlighting test score outcomes and other key indicators of educational performance. These have become centerpieces of attention among not only educators but many other stakeholders. Snow and ice control agencies can also benefit from report cards regarding their performance.
3. **Measures encourage delegation rather than micromanagement.** Hierarchical structures and extensive oversight requirements can hinder organizational effectiveness. Performance measures free senior executives for more strategic decision making and collective intervention while clarifying the responsibilities and authority of managers down the line.

Benefits of Performance Measurement

Performance measurements offer the following benefits to an organization:

1. **Performance measurement enhances decision making**. The process of developing performance measures allows an agency to determine its mission, set goals for desired results, and identify methods of measuring how well the results are achieved. The data generated through performance measurement can be used to determine program effectiveness, evaluate options for road maintenance, and chart long-term programs and fiscal plans. For upper-level management, performance measures can focus attention on outcomes and can allow for solid evaluation techniques.
2. **Performance measurement improves internal accountability**. Measuring performance gives decision makers a significant tool to achieve accountability. Employees at all levels are accountable to managers for their performance or that of their crew, and upper-level managers are accountable to departmental executives. This relationship becomes much clearer when outcomes and outputs are measured by a commonly accepted standard. Systems such as "management by objectives" or "pay for performance" can be much more effective when teamed with a high-quality measurement system.
3. **Performance measurement supports strategic planning and goal setting**. Without the ability to measure performance and progress, the process of developing strategic plans and goals is less meaningful. While there is clearly some benefit to thinking and planning strategically, the evaluation of such plans and goals cannot be objective without measuring performance and achievement. For example, our literature review found that the Wisconsin Department of Transportation in 1996 implemented its MAP, which used performance measures to achieve its performance-based service levels. These performance measures are based on customer-oriented outcomes or the results of highway winter maintenance operations that highway users are able to identify. The results are collected by field evaluations of highway conditions (Baroga 2004).

Organizational metrics are important for these organizations. Working with employees, management, and affected stakeholders, organizations involved in strategic planning can develop measures of performance in the production of goods and services and in meeting the organization's most important objectives.

There is no single model or process for developing performance objectives and measures, nor is there a process that will guarantee good results. We have attempted to synthesize lessons learned from the literature as well as the insights gained from our surveys and work with agencies in applying performance measurement to the management of snow and ice control operations issues.

Applying a Performance Framework or Toolbox

One method used to develop performance measurements for snow and ice control is to apply a framework or toolbox to the problem. A performance measure toolbox brings structure to performance planning and clarifies the connection between activities, outputs, and results. The toolbox uses the following steps relative to the objectives specified in an agency's strategic plan:

1. **Confirm snow and ice control operations role**. The rationale here is to determine why the agency is measuring performance. The agency should define the role that snow and ice control operations are intended to play with respect to strategic objectives and should provide a basis for establishing overall targets and performance measures.

2. **Identify the key snow and ice control activities and outputs**. The rationale for this step is to ensure that winter maintenance managers and staff focus on key issues that contribute to the achievement of the department's strategy for snow and ice control operations.
3. **Identify stakeholders and issues**. The rationale for this step, in order to formulate a set of snow and ice control objectives, is to identify the customers whom the winter maintenance activities and outputs should serve, influence, or target; the other principal groups affected are; and the ways these groups are affected.
4. **Identify what the snow and ice control operations aim to accomplish**. The rationale for this step is to illustrate that the results are defined in terms of outcomes that then become the focus for determining appropriate objectives, milestone targets, and measures, e.g., that managers receive appropriate feedback.
5. **Identify responses and performance requirements**. The rationale for this step is that performance objectives must be defined in operational terms to be managed effectively.

CONCLUSIONS

Achieving reliable and relevant performance data for a snow and ice control performance measurement program is a large task for any organization. The challenges and problems associated with performance measurement are multiplied by the unpredictable nature of working with winter weather.

Complex factors influence the usefulness of performance measures. First, the performance measures must be perceived as reliable. Straightforward processes are best suited for obtaining reliable data because complexities can cause variations in reporting. Furthermore, each district or garage should have a clear understanding of what to include and exclude from the performance measurement program. The program should also involve key people in the creation of performance target definitions and in the reexamination of existing definitions and measures.

In addition to reliability, relevance is a key ingredient in data use. As discussed, relevance takes many shapes, and managers and jurisdictions each have their own unique needs. Factors influencing relevance include managerial control, timeliness, fruitfulness, organizational capacity, and the organizational philosophy of performance measures. This is not an exhaustive list, yet it is enough to demonstrate that achieving data use is not effortless.

Agencies may be able to improve their snow and ice control services by measuring the effectiveness of services they provide. Measuring performance, or the results of services, provides several benefits. The results can demonstrate value to taxpayers. Knowing the results of the service allows an agency to tell whether it has accomplished its intended objectives, and, if necessary, adjust its procedures or practices. Concentrating on results also helps agencies be more responsive to the needs of their customers and may help agencies communicate more effectively with taxpayers.

The research revealed the organizational objectives associated with snow and ice control performance measures. These objectives relate to the inputs, outputs, and outcomes of snow and ice control operations as follows:

- Accounting for inputs used for snow and ice control
- Accounting for outputs accomplished
- Operational efficiency
- Meeting outcome goals

- Highway safety
- Highway mobility
- Public satisfaction
- Controlling negative environmental impacts

Many snow and ice control agencies have not moved beyond collecting performance data to utilizing these data to proactively manage the agency. A successful snow and ice performance program relies on the ability to obtain meaningful data, use these data to manage the program, and institutionalize these practices so that they become routine. Leadership is important to promote understanding and support for the organizational mission, and leadership demonstrates commitment to managing for results. Staff must buy into the program and feel empowerment and continuity. Finally, the results of performance management must be communicated among relevant stakeholders is crucial to the success of any performance measurement or management system.

While performance measurement is beginning to become more common, very few snow and ice control agencies are actively involved in using that data to proactively manage. In other words, performance measurement has not yet become performance management. Careful planning, consistent implementation, and thorough communication will help shift the snow and ice control agency beyond performance data collection to effective performance management.

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The Utilization of Agriculturally Derived Lignin as an Antioxidant in Asphalt Binder

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ABSTRACT

Asphalt pavements undergo long-term aging due to oxidation of the asphalt binder. As a pavement oxidizes, it stiffens and eventually cracks. The use of an antioxidant in an asphalt binder could retard aging, thus increasing the pavement's service life. Lignin is a known antioxidant and is highly available from timber and many agricultural products. A wet-mill ethanol process produces several co-products, some of which contain lignin. The utilization of lignin from an ethanol plant could provide benefit to asphalt pavements, along with increasing the value of these lignin containing co-products. Three different lignin containing co-products were added to four asphalt binders with varying amounts to find the optimum amount of lignin that would provide the greatest benefit to the asphalt binders. The asphalt-lignin blends were evaluated according to Superpave specifications and performance-graded on a continuous scale. The blends were also tested for separation tendencies. The results illustrate that the addition of lignin has a slight stiffening effect on the binder. The more lignin added, the greater the stiffening. However, separation effects are significant at high lignin levels. Binder stiffening increases the high-temperature properties, while the low-temperature properties are slightly decreased. However, the low-temperature stiffening effects are small and do not change the actual performance grade. The lignin has an overall effect of widening the performance grade range of the asphalt binders.

Key words: antioxidant—asphalt—ethanol—lignin

INTRODUCTION

As the world demand for energy increases, the United States looks to establish a more bio-based economy (Van Dam et al. 2005). A bio-based economy has several advantages to traditional reliance on fossil fuels. First, energy derived from agricultural products is renewable. Crops such as corn, soybeans, switch grass, and sugar cane are common in the United States and are excellent for producing reliable and efficient biofuels. Second, fuels created from agricultural products burn with fewer toxic emissions and a reduction in the amount of green house gases produced (Demirbas and Balat 2006). Carbon neutrality is becoming ever more important as the world's concern for global warming increases. Finally, a bio-based economy provides the United States and other agricultural nations with a large economic opportunity. Crops would have greater value giving rural America an economic boost. New markets will need to develop to utilize co-products derived from biofuel production. However, research of co-products from fuel production needs more attention.

Ethanol is the nation's largest and most popular biofuel. Ethanol has existed for over 100 years, but is increasing in popularity as the United States tries to liberate itself from dependence upon foreign oil (over 62% imported) (Bothast and Schlicher 2005). Ethanol production is an efficient process. However, many co-products are produced as the entire corn kernel cannot be used to make fuel (Gulati et al. 1997). These co-products have uses, mostly acting as livestock feed. Further market opportunities for co-products could greatly benefit ethanol producers.

With wet-mill ethanol production, co-products are produced that contain lignin. Lignin is a well studied biological polymer and is a known antioxidant. The antioxidant effects of lignin are derived from the scavenging action of phenolic structures on oxygen containing free radicals (Dizhbite et al. 2004). Lignin contains large amounts of phenolic structures enabling it to be an effective antioxidant. The use of lignin in asphalt pavements could provide substantial benefit. The primary cause of long-term aging in asphalt pavements is oxidation of the asphalt binder. The complex reaction of binder with atmospheric oxygen causes the binder to age and stiffen. As a binder stiffens, it becomes brittle and eventually cracks, usually by means of thermal and fatigue stresses (Roberts et al. 1996). Fixing distressed pavements is costly, so delaying oxidative aging would be valuable.

Antioxidants have been previously researched in asphalt binders. Chemicals such as zinc dialkyldithiophosphate (ZDDP) and naphthenoid oil have proven successful at retarding asphalt binder oxidation and age hardening (Ouyang et al. 2006). However, practical limitations have prevented their incorporation into the industry. Wood lignin has also been researched and proven beneficial to asphalt. The high- and low-temperature properties of the binder were increased at four and seven percent by weight lignin content (Bishara et al. 2005). Yet, wood lignin is a waste product of the paper industry, so it carries a negative stigma. The idea of using roads as a "horizontal landfill" is unattractive to many in the asphalt industry. However, if a low-cost, environmentally friendly antioxidant could be used as a performance modifier, then that chemical would have a much greater chance of being incorporated into the industry. Lignin derived from ethanol production has positive value, so its potential use in asphalt pavements is high. No published asphalt research has been performed on ethanol-derived lignin.

EXPERIMENTAL PLAN

This study examined four asphalt binders, each combined with three different corn lignin samples at four percentages. The binders used were two local (one polymer modified and one unmodifed) and two well-studied binders from the National Materials Research Laboratory (AAD-1 and AAM-1). The two local binders were obtained from a local supplier in Tama, Iowa. The corn lignin samples were acquired from Grain Processing Corporation in Muscatine, Iowa. The samples vary slightly in their lignin content due to different processing methods. After processing, the samples range from around 10 to 15% lignin. The remainder is a combination of inert fillers consisting of cellulose and hemi-cellulose. Lignin A and B are modified corn hull which contain more lignin than C. The lignin samples were dried to approximately 10% moisture and then ground into a fine powder. Each binder was blended with each lignin sample at 3.0, 6.0, 9.0, and 12.0% by weight. Also, each binder was tested without the addition of lignin. A total of 52 asphalt/lignin blends were produced, each tested in triplicate.

Each asphalt/lignin blend was performance graded on a continuous scale according to Superpave specifications (AI 2003). First, the unaged samples were tested in a dynamic shear rheometer (DSR) to determine the high-temperature visco-elastic properties. Next, a rolling thin film oven (RTFO) was used to short-term age the binders. The RTFO simulates the aging the binder goes through while being mixed and constructed in the field. The RTFO residue was then tested in the DSR to verify the high-temperature visco-elastic properties. Next, the RTFO residue was long-term aged in a pressure aging vessel (PAV). The PAV mimics 8 to 10 years of actual in-service pavement aging (Roberts et al. 1996). The PAV residue was tested in the DSR to determine the intermediate-temperature visco-elastic properties. Low-temperature properties were determined by examining the PAV residue test properties with a bending beam rheometer (BBR).

Separation testing was performed according to ASTM D 7173-05. Asphalt/lignin blends were examined to determine the maximum amount of each lignin that was able to remain suspended in solution after 48 hours.

RESULTS AND DISCUSSION

Initially, the samples were blended using a high-speed shear mixer. Each blend was mixed at 155°C for one hour. We used 155°C because it closely resembles the temperatures the binder will be subjected to while being mixed in the field. Once the asphalt reached temperature, the mixer was initiated and set to 5,000 rotations per minute (RPM). We used 5,000 RPM because it was the lowest speed that produced a sufficient vortex to properly blend the lignin into the binder. If a higher mixing speed was used, sufficient friction was produced by the mixing head to increase the heating of the asphalt beyond 155°C. Using speeds of 6,000 RPM or more increased the heat of the blends well beyond the desired mixing temperature.

First, the asphalt/lignin blends were studied for separation effects in accordance with ASTM D 7173-05. The blends were poured into aluminum cigar tubes and stored vertically in an oven at 155°C for 48 hours. Theoretically, the denser particles settle to the bottom while the lighter particles stay suspended. Immediately after heating, the samples were frozen. The frozen tubes were cut into three equal-size portions. The middle fraction was discarded, while the top and bottom portions were heated and poured into separate containers. The top and bottom portions were tested separately in a rotational viscometer. The viscosities of each sample were tested at 135.0°C, 150.0°C, and 165.0°C. Results showed large viscosity differences at the 12% level. The top and bottom portions differed by an average of 13.3%. This amount of separation is undesirable and would cause problems for future binder handling and construction. At the 9% level, the viscosities differed only by 3.7%.

Performance testing of asphalt/lignin blends showed consistent trends with all four binders. In general, the addition of lignin slightly stiffened the asphalt. Figure 1 illustrates consistent trends with binders AAD-1 and the local polymer modified binder (LPMB). Both graphs show $G^*/\sin(\delta)$ values tested at 64°C on the y-axis with the means represented as solid bars. A larger $G^*/\sin(\delta)$ value represents a stiffer binder, while a smaller G*/sin(δ) value represents a more viscous, flowing binder. It is easily seen that the further addition of lignin continues to stiffen the binder. In the case binder AAD-1, lignin B produces a significantly stiffer blend than lignin A or C based upon a 95% level of confidence. Also in the case of AAD-1, the addition of all three lignins at the 9.0 and 12.0% level increase the high-temperature performance grade from a PG 58 to a PG 64. For the LPMB, the performance grade remained a PG 64 for all blends.

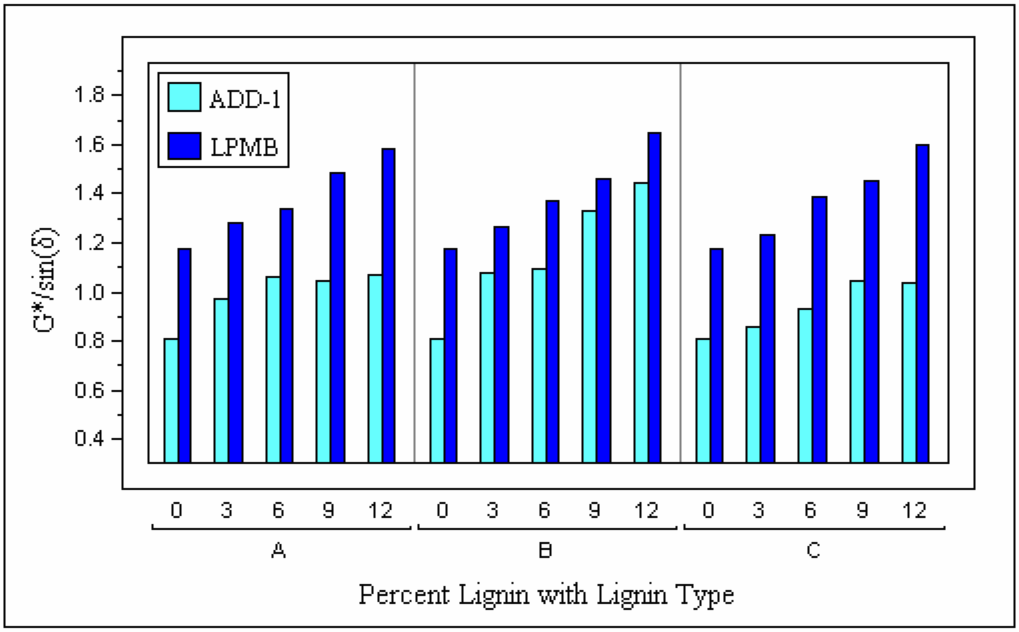

Figure 1. Unaged AAD-1 and LPMB Data at 64°C

After being RTFO aged, the same trends appear. Figure 2 illustrates the effects of the same two previous binders. It is seen from the LPMB that once the addition of lignin reaches the 12.0% level, the stiffness dramatically increases while AAD-1 has a more gradual stiffening effect. The presence of polymer in LPMB indicates some possible interaction effects because of this sharp increase. Since both binders pass at 64°C, the results of the unaged DSR test determine the actual performance grade. Therefore, AAD-1 increased from a PG 58 to a PG 64 with the addition of 9.0 and 12.0% of any of the three lignins. This is a positive effect that adds potential benefit in areas of a warmer climate.

Of most interest is the results of the PAV aged blends. In a PAV, 50.0 gram samples were subjected to high pressures (2.1 MPa) and high temperature (100°C) for 20 hours. After aging, the asphalt looses much of its viscous component and becomes much stiffer then the RTFO aged residue. The goal of this project is to evaluate the antioxidant potential of the lignin. If the lignin acts as an antioxidant, then the PAV aged samples with lignin present should see less age hardening than the samples without lignin. The PAV residue was tested for both intermediate-temperature properties with the DSR and low-temperature properties with the BBR.

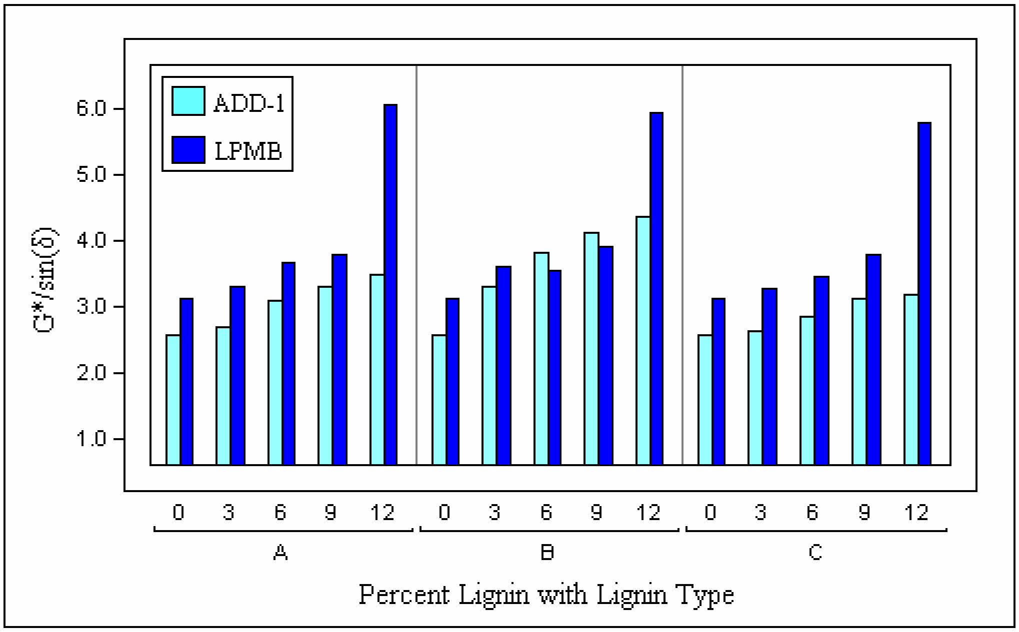

Figure 2. RTFO Aged AAD-1 and LPMB Data at 64°C

The DSR results of the PAV residue show the same pattern as the high-temperature tests. As more lignin was added, the stiffness of the binder generally increased. The intermediate critical temperature does not determine the performance grade, but rather gives an estimation of how the pavement will behave in average operating temperatures. Figure 3 shows the results of ADD-1 and LPMB at 19°C and 25°C, respectively. Both binders show a general increase in stiffness with the further addition of lignin. With the addition of 9.0 and 12.0% lignin B to AAD-1, the binder actually fails at 19° C by having a G*sin(delta) greater than 5,000 kPa. The same happens to the LPMB at 22°C with 6.0 and 12.0% lignin B. The data indicates that lignin B stiffens the binders statistically more than lignins A and C at a 95% confidence level. Lignins A and C produce results that are not statistically different. The results suggest that binders with lignin B would be more susceptible to fatigue cracking than lignins A and C. At the 3 and 6% levels, the binder behaves statistically similar to the neat binder (no lignin added) with lignins A and C.

Finally, the binders were tested for their low-temperature properties in a BBR. A BBR measures two key properties, stiffness and change in stiffness (m-value). To pass at a given temperature, a sample needs to have a stiffness less than 300 MPa and a m-value greater than 0.300. An asphalt with greater stiffness has a higher susceptibility to thermal cracking. The m-value is a measure of the rate of stress relaxation of the binder. Binders that can relieve thermal stresses faster have a greater ability to resist cracking. It can be seen from Figure 4 that the addition of lignin increases the stiffness of the binder at low temperatures. However, there are no statistical differences between the different lignins. The binders all passed the failure criterion of having stiffness less than 300 MPa. What caused the binders to fail was the m-value criterion. It can be seen from Figure 5 that the addition of lignin causes the m-value to slightly decrease. This is a negative effect, but the change is small, so the actual performance grade is not changed. All of the blends passed at -12°C for AAD-1 and failed at -18°C. Every blend for the LPMB passed at -6°C and failed at -12°C. It must be noted that there were no statistical differences between the different lignins with the m-value criterion at a 95% confidence level.

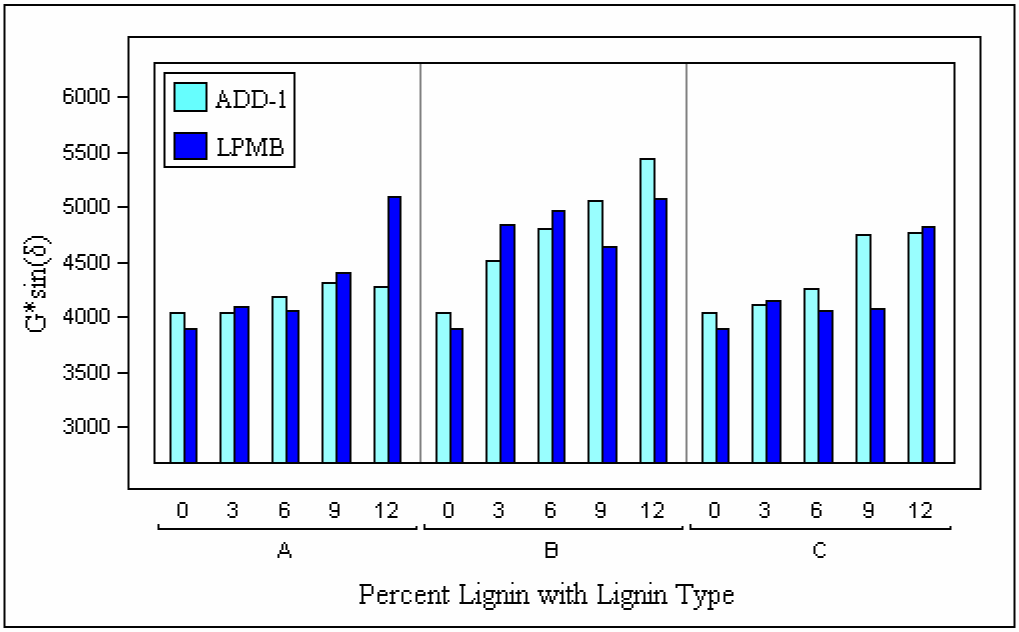

Figure 3. PAV aged AAD-1 and LPMB Data at 19 and 22°C

CONCLUSIONS

The addition of lignin containing co-products to asphalt binders causes a slight stiffening effect depending upon the percentage and type of lignin used. The high-temperature properties were positively affected. At high lignin contents, the high-temperature performance grade was increased. At the intermediate and low temperatures, the increased stiffness had a negative effect, but this effect was small and did not change the actually performance grade. Since the high temperature of the performance grade was increased and the low-temperature grade was not affected, the lignin has an effect of widening the temperature range of the binders. This is opposed to a filler, which would simply shift the temperature range. The lignin provides an overall benefit to the asphalt. Also, there were no negative interactions with the polymer present in the local binder. The LPMB behaved similar to the binders without polymer. The optimum amount of lignin to be added was 6.0 or 9.0%. The 12.0% level was overly susceptible to separation. Lignin B provided the highest increase in the high-temperature properties but also has the greatest susceptibility to fatigue cracking at intermediate temperatures. Therefore, lignins A and C are recommended over lignin B.

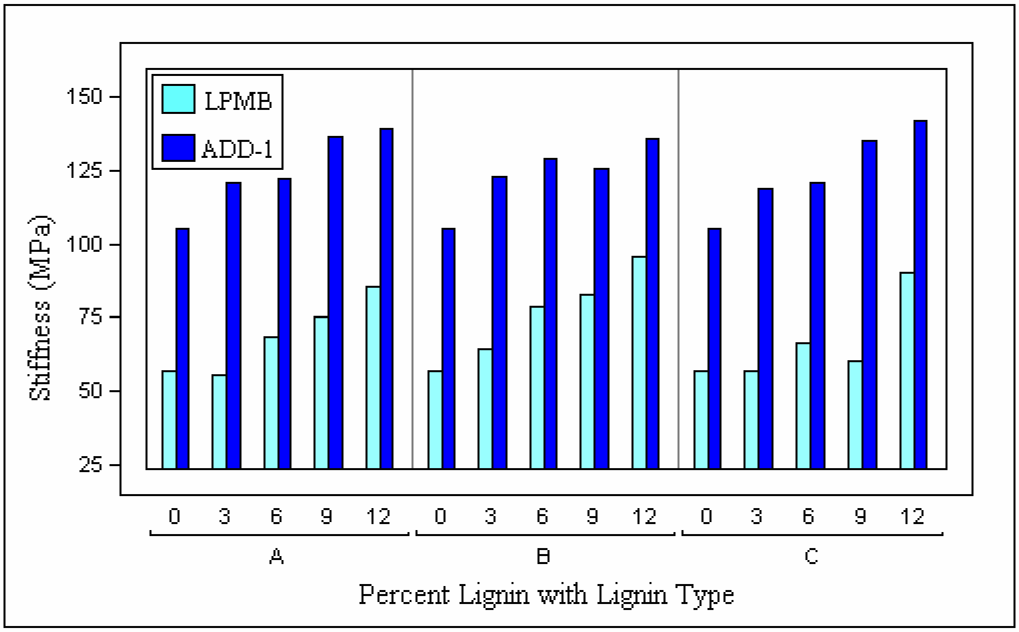

Figure 4. BBR Stiffness for AAD-1 and LPMB data at -12 and -6°C

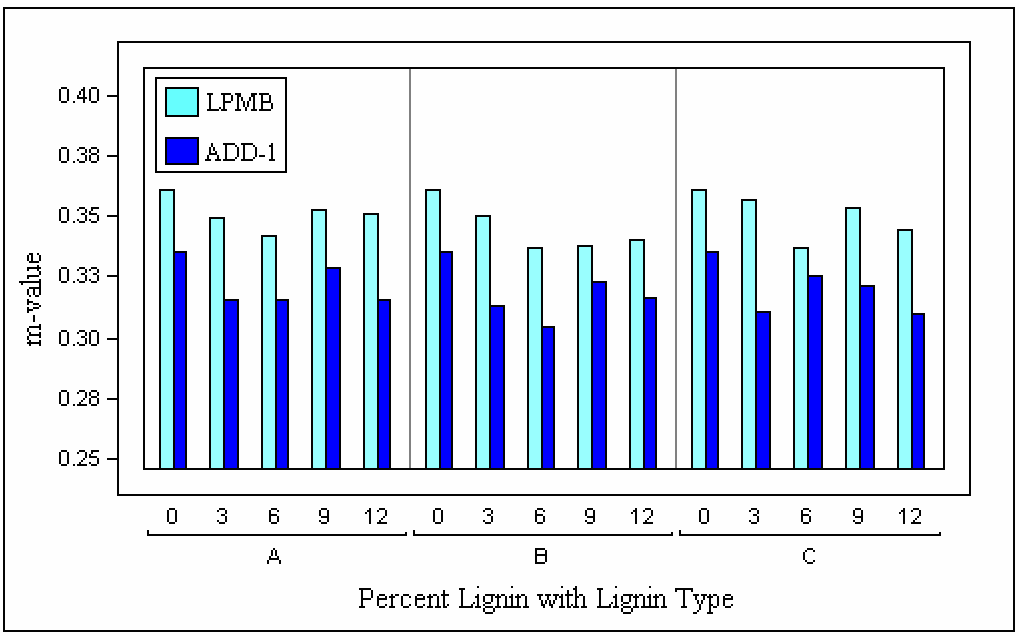

Figure 5. BBR m-value for AAD-1 and LPMB data at -12 and -6°C

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Utilizing Wireless Data Network for AVL and Mobile RWIS

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ABSTRACT

Indiana has a statewide wireless network (SAFE-T) that has been primarily used by the state police. It has enough capacity to accommodate other users. The Indiana Department of Transportation (INDOT) has developed a winter operations tool that utilizes GPS, sensors to produce real-time information on chemical distribution, road temperature and plow position, and road and weather conditions.

The data collected at the maintenance vehicle (snow plow) produces maps of time of chemicals placed, type of chemical, application rate, vehicle speed, road temperature, and plow position. Also, road and weather conditions can be displayed. Another feature of the system is transferring the appropriate data into a maintenance decision support system (MDSS).

In the 2006–2007 winter, ten snow plow vehicles were equipped with the equipment described above. The equipment was placed in vehicles at three locations: Laporte, Monticello, and Columbus. This paper describes the results of system tests for the last two winters. Also, a summer application, paint stripping, will be described.

Keywords: AVL—RWIS—winter operations—wireless

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INTRODUCTION

Indiana Department of Transportation (INDOT) has developed and tested two automated vehicle location (AVL) systems. One is a winter operation application that tracks and manages snow and ice removal activities. The other application is a summer activity that tracks and records roadway painting operation. Both applications will be described.

Snow and ice removal trucks provide an excellent option for collecting data in a mobile environment. There are a couple technical issues that need to be solved with this approach. One is as sensors are added to the mobile collector (snow plow vehicle) the collecting and assembling of the various data strings is an issue. The other is the transferring of this information to the maintenance decision support system and the traveling public road condition system. A current research project at INDOT has utilized the statewide wireless data network, SAFE-T, for data transfer. Other appropriate data transfer options that are being investigated and tested are a wireless hotspot and using a cellular data network.

AVL provides the capability to electronically record the location and activities of winter maintenance vehicles. This data can be transferred electronically and save time, improve data accuracy, and improve the feedback to managers that are responsible for making decisions on winter activities.

Other organizations that have used this technology report quantifiable improvements in their winter activities. These include improved reporting data, better utilization of equipment, and savings in fuel and chemical costs.

AVL OPTIONS

Two AVL options were initially considered and studied. Option 1 uses an AVL service provider where data is transferred via cellular service. There are numerous AVL service providers. Information was collected and a cost comparison done. A summary of the cost comparison is in Table 1. This system consists of proprietary software and a monthly service of $40 to $60 per vehicle.

Option 2 consists of using the Indiana SAFE-T wireless network to transfer data. The Indiana network is used by the state police, and a coverage map is shown in Figure 1. Coverage is available in the northern two-thirds of the state. Installation is proceeding in the southern third of the state. Data is transferred through a 800mhz radio network with a transmission rate of 19.2 Kb that is managed by Motorola. INDOT has approximately 1,100 vehicles that participate in winter operations. Motorola did a data traffic study with this number of vehicles and determined that the data network has sufficient capacity to support this application. All equipment and software would be owned by INDOT.

Option 2 is considerably less expensive due to the monthly service charge required in Option 1.

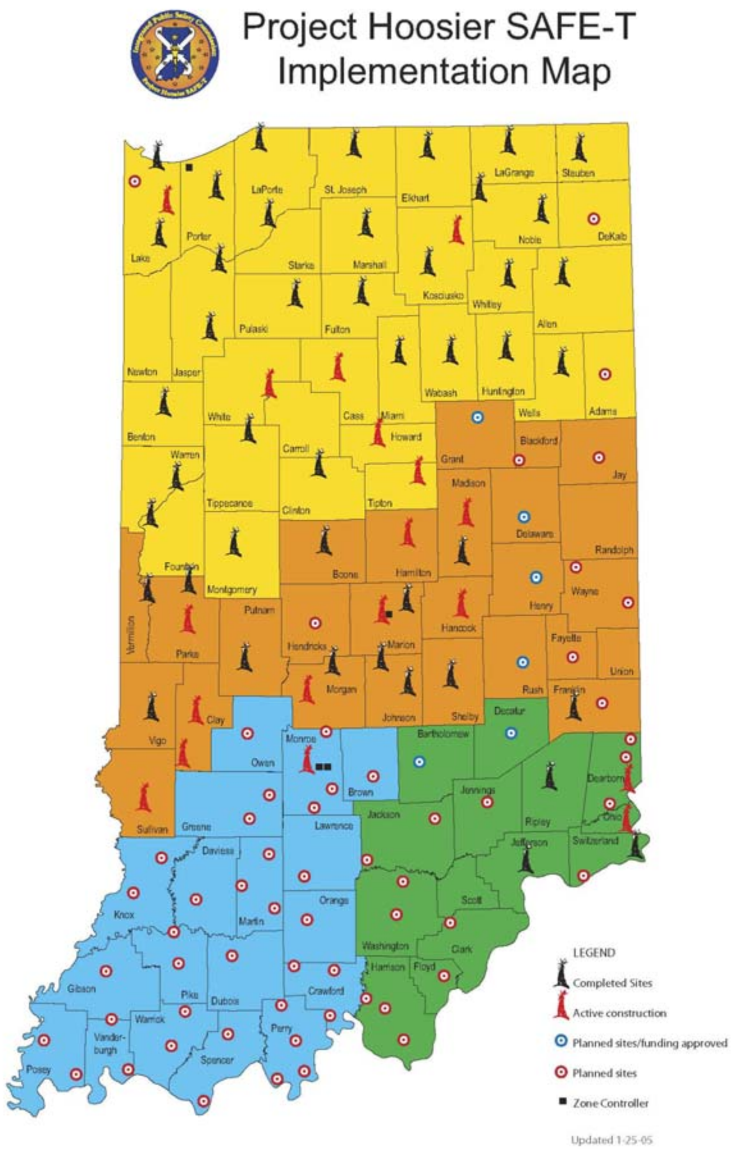

Figure 1. SAFE-T network

Table 1. Cost comparison after year 5 between Option 1 and Option 2

	Option 1		Option 2	
Year	**Additional Cost**	**Total**	**Additional Cost**	**Total**
Year 5		$4,500,000.00		**$3,753,390.00**
Year 6	+ $720,000[1]	$5,220,000.00	+ $32,304.00[2]	**$3,785,694.00**
Year 7	+ $720,000	$5,940,000.00	+ $33,596.00	**$3,819,290.00**

1. Annual Service Fee = $720.00 / vehicle / year x 1,000 trucks = $720,000.00 for Year 6
2. Annual Software Maintenance Cost (10% of Software Cost plus 4% annual escalation) = $31,062.00 (Year 5) x 1.04 = $32,304.00 for Year 6
3. First year that Option 1 costs more than Option 3, assuming the service fee in Option 1 is not changing

INDOT AVL SYSTEM

Since the cost differential between these options is significant, Option 2 was chosen. Figure 2 is a conceptual diagram of Option 2.

The first winter test period, 2005–2006, experienced weather and system problems. With the weather, there were very few winter events at these two sub-locations to test with. There were several system issues. One was driver interface. Inputting the login and password as well as wireless signal verification troubled the drivers. Also, there was a need to do spot treatments and cleanup activities that could not be reported. These modifications were made during the summer of 2006. Also, the system was expanded to three locations and added six more trucks for the 2006–2007 season. The new location was Laporte, and the truck distribution is as follows:

- Laporte: 4
- Monticello: 3
- Columbus: 3

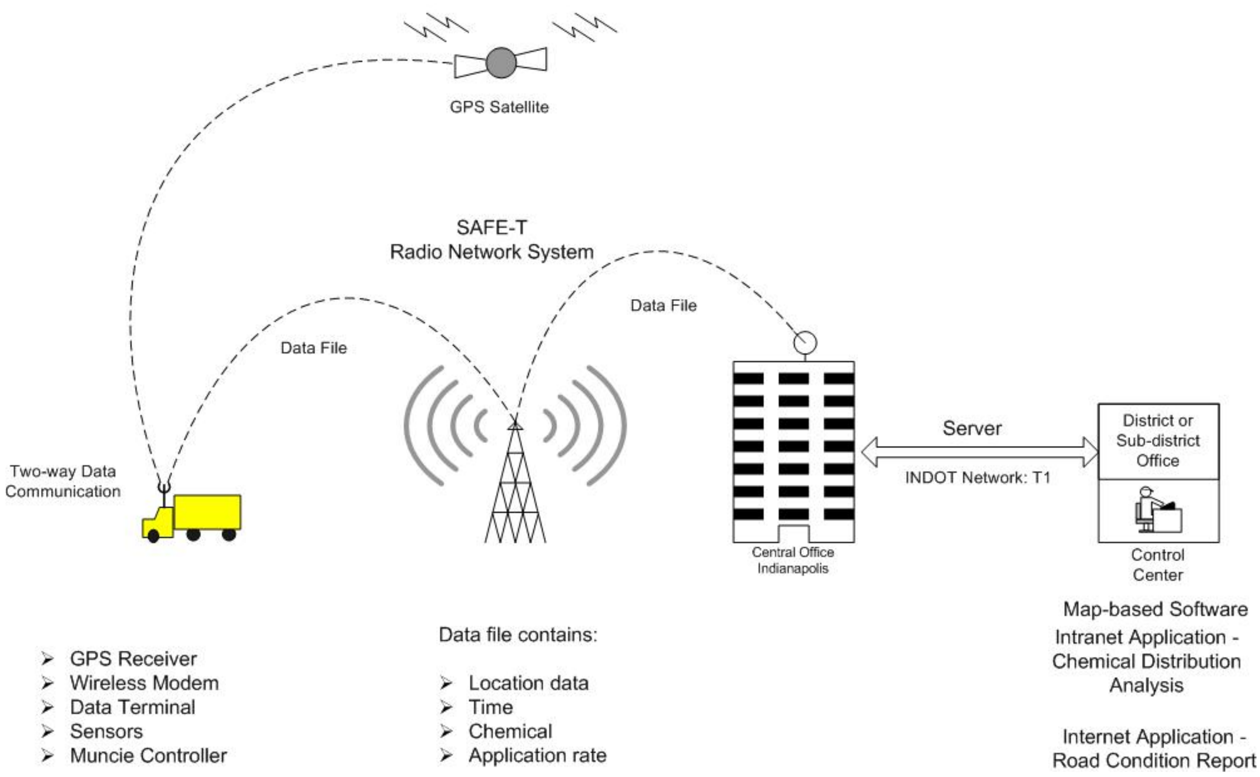

Figure 2. Conceptual diagram of INDOT AVL network using SAFE-T radio network

A more detailed description and explanation of this system is described next.

Vehicle Hardware

A similar conceptual view of the in-vehicle hardware is shown below in Figure 3. There are four main hardware components in the trucks.

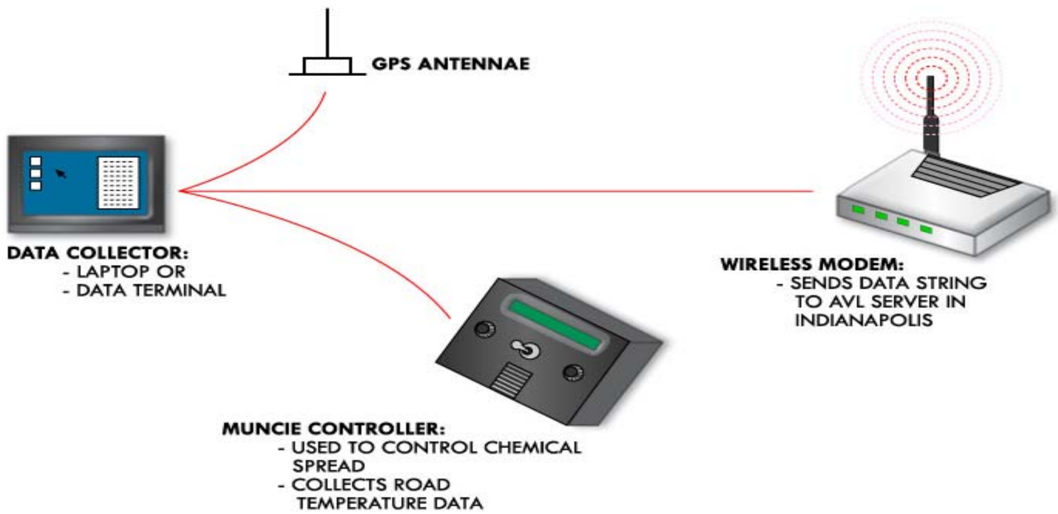

Figure 3. Vehicle hardware diagram

GPS Antennae

Garmin GPS 18 receiver is a GPS OEM unit that receives location data from a satellite. This brand and model was chosen for its low cost and large operating temperature range.

Chemical Distribution Controller

INDOT uses the Muncie Controller (MESP 402E) to control the distribution of chemicals from the winter vehicle. Through the course of the research project a temperature sensor and a plow position sensor were added to the vehicle. These sensor data are collected through the Muncie controller.

Wireless Modem

Motorola modem (VRM 850, 800MHz 35W) transfers the data at regularly defined intervals from the truck to the AVL server in Indianapolis over the SAFE-T network.

Data Collector

Three options for the in-vehicle data collector were tried and tested. A mini PC with touch screen monitor, a touch screen laptop, and an ultra mobile tablet PC were used as the computer data collection alternatives (Figure 4).

(a) Xenarc in-vehicle mini PC | **(b) Samsung Q1 ultra mobile tablet PC** | **(c) Fujitsu B6210 touch screen laptop**

Figure 4. Three options for the in-vehicle data collector

Software was developed to interface the collector with the other three hardware devices: Muncie controller, GPS antennae, and Motorola modem. A description of this software follows.

Vehicle Software

Motorola MWCSII and Autoxfer

This software is a Motorola product, and it transfers data over the SAFE-T network. The MWCSII software connects the modem in the client (truck) to the SAFE-T network. The Autoxfer program is responsible for sending data to the server at defined intervals. The time interval that was used in our application is three minutes.

Garmin Spanner

The Garmin Spanner software writes the GPS data to a selected serial port. The data will then be collected through the Purdue Data Collecting software.

Purdue Data Collection Software

The Purdue Data Collecting Software was developed in Visual Basic 6. The program is activated at computer startup. The first thing the program does is to start the Garmin Spanner program and the Motorola Autoxfer program. Both will run in the background. This software has a series of input screens for the driver and combines the input with data from the Muncie controller and the GPS antennae. The combined data is sent every three minutes to the AVL server through the wireless network. The driver is required to input values for road and weather conditions and report road problem location as well as spot mode location.

Input Screens

There are two operation modes: (1) Chemical Spread and (2) Plowing Only. The Chemical Spread mode is used when the operation involves chemical spread with or without plowing. If no chemical spread occurs, no data is transferred. The Plowing Only mode is used when no chemicals are spread. The Muncie controller should be turned on in both modes.

File Management

This data file is sent every three minutes over the SAFE-T network to the AVL server in Indianapolis. File management software was developed to combine the truck data into a master merged text file. This merged data file is then saved to the Oracle server at a three-minute interval. The map reports use the GISMAP server and ARCIMS software, developed by the INDOT GIS section, to retrieve data from Oracle and display it on state GIS maps.

Map-Based Reports

The map reports display truck data in real time (three-minute delay). Also archived data can be displayed. The map application displays in layers truck speed, application time, application rate, chemical type, road condition, weather condition, and road temperature. As an example, Figure 5 shows additional truck data and how layers can be turned on and off.

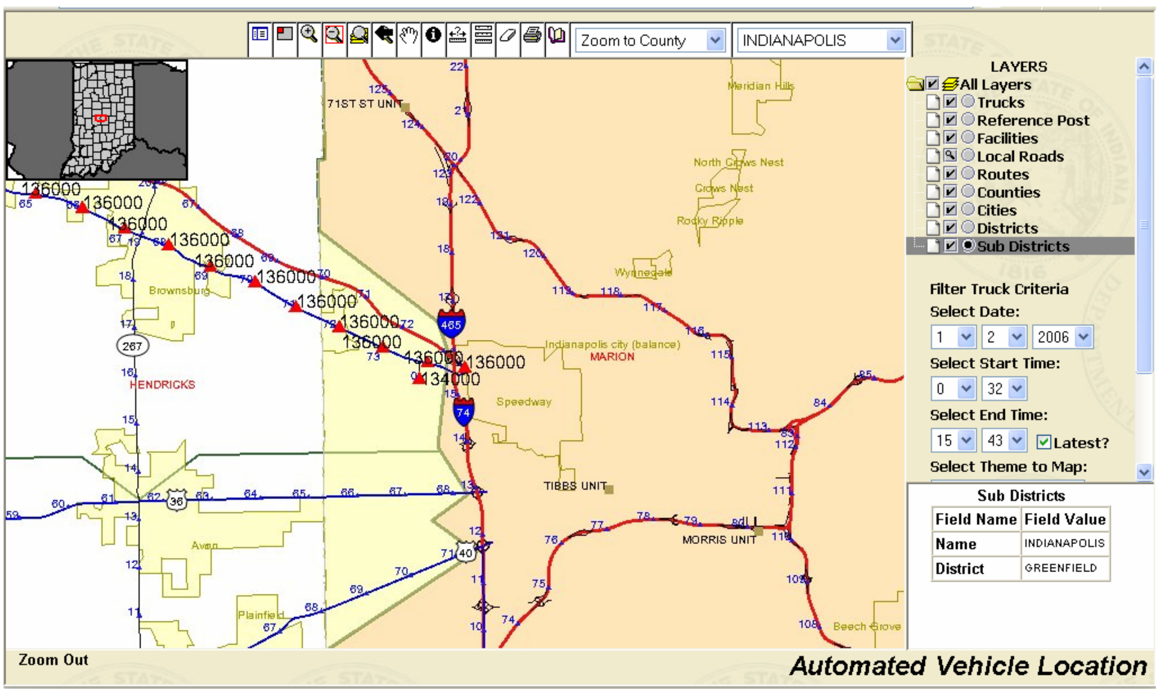

Figure 5. AVL map layers

OTHER WIRELESS DATA TRANSFER OPTIONS

Due to difficulties experienced with the Motorola modems, other data transfer options were investigated. This next section describes two: hotspot and cellular data networks.

Hotspot and VPN

Data is collected in the truck, and when the truck comes within range of a wireless hotspot, a connection is made and data transferred. A hotspot could exist at the unit so when the truck returns the data transfer occurs. Figure 6 illustrates this concept. In the figure, a virtual private network (VPN) is a secured network that requires login and password.

In this case the data is not real-time but delayed in reporting, which is a negative with this approach for winter operations. The positive is a modem or cellular device is not required in the truck for data transfer. This option will work for summer operations where real-time data is not required.

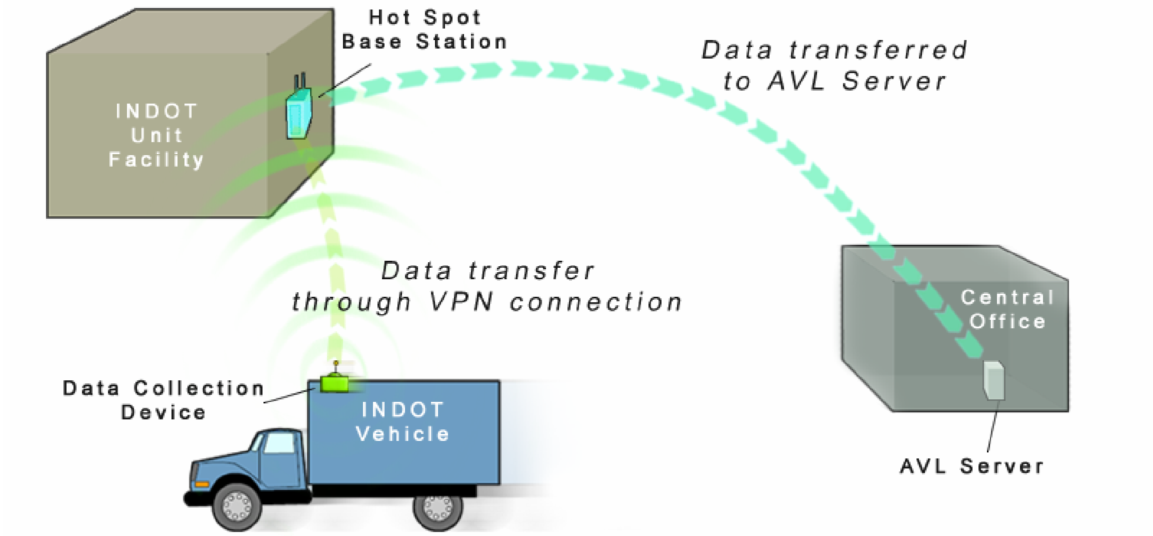

Figure 6. Hotspot option

Using Pocket PC/Smart Phone Devices to Transfer Data

Due to data transmission difficulties experienced with the Motorola modems, another option was explored that utilized a smart phone to transfer data. Smart phone is a mobile phone that was originally designed to have email and basic personal organizer functionalities, while a pocket PC is a personal computer like handheld device with condensed functionality.

Operating Systems

Both smart phone and pocket PC devices have different platforms, but similar functionalities. The different operating systems are Symbian, Linux, Windows Mobile, RIM, and Palm OS.

The newest Windows operating system for such device is Windows Mobile 6.0 that was released in February 2007 with separate versions for each device. The smart phone version does not have touch screen capability; the pocket PC phone version is the PDA's version with phone functionality, and the PDA version is the plain PDA without cellular radios. Photon is the operating system that combines the pocket PC phone and the smart phone version of Windows Mobile 6. This operating system will be launched in 2008.

Serial Communications

With a serial RS232 interface in these mobile devices, data transfer can be established through serial communication. This data transfer application can be developed using visual basic.net.

Wireless Data Transfer to Server

Data can be transferred from a pocket PC/smart phone to another computer through General Packet Radio Service (GPRS). For corporate networks, a VPN connection is necessary. Figure 7 describes this method of transferring data.

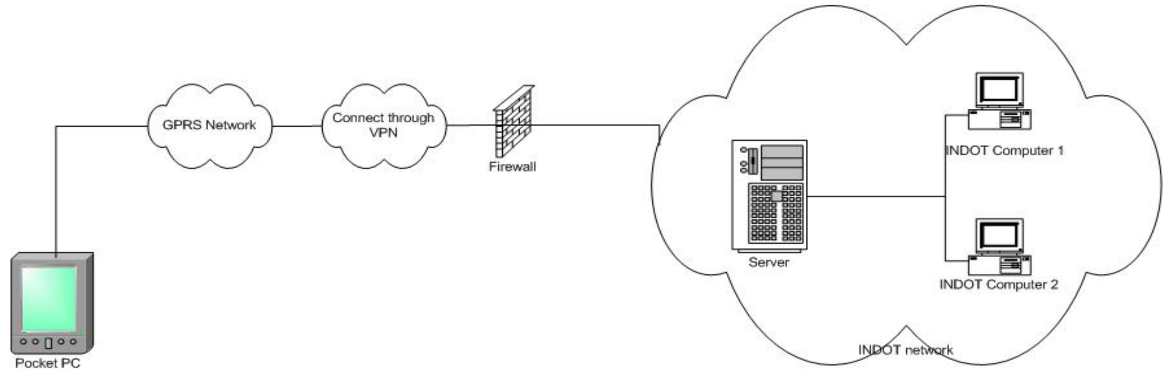

Figure 7. Data transfer with mobile device

System Requirements for Pocket PC/ Smart Phone

- **Operating system.** Windows Mobile 5.0 or higher
- **Interface.** Serial RS-232 for serial communication, Bluetooth
- **Data plan.** GPRS data plan with capability for VPN connections
- **Device cost.** Pocket PC phone cost is usually under $500. This varies with service provider; the cost may be lower with service contract.

Summary

In conclusion, the application can be implemented through a pocket PC phone. By eliminating the computer and the wireless modem, the overall system equipment cost can be reduced dramatically. Since there are fewer hardware components involved, the installation of the equipment will be easier. Table 7 compares overall costs between this option (Option 1) and the Motorola option (Option 2) in 1,000 trucks.

Table 7. Cost comparison for 1,000 trucks over a five-year period

Options Hardware	Parts	Model / vendor	Cost per part	Subtotal	Total max. cost
Option 1					
In-vehicle equipment	GPS receiver, modem, data terminal	Multiple vendors	$500.00 / vehicle	$500,000	$3,500,000.00
Service fee	N/A	Multiple vendors	$3,000/ 5 years/ vehicle ($50 / month / vehicle)	$3,000,000	
Option 2					
In-vehicle equipment	GPS receiver	GPS 18 / Garmin	$130.00	$3,355.00 per vehicle	$3,753,390.00
	Radio modem, antenna	VRM 850 / Motorola	$1,900.00		
	Rugged laptop	ML 850 / Motorola	$1,200.00		
	IP setting software	MWCS2 / Motorola	$125.00		
Base station	N/A	AVL server	$ 5,000	$398,390.00	
Map-based control software	N/A	PU/INDOT	$ 0		
File transfer module	N/A	PMDC application software / Motorola	$393,390.00		

SUMMER AVL APPLICATION

The summer of 2007, a paint stripping application was developed and is being tested in the Laporte District. The hardware consists of a laptop and a GPS receiver in the trailing escort vehicle. The laptop has software that collects painting information. The software requires manually selecting the stripe type. Figure 8 is the input screen.

Painting data is accumulated in a data file. Periodically paint data is transferred to the painting supervisor desktop. This data is viewed in a desktop map-object software program developed in VB.net. Stripping info can be viewed by date or date range on the map with corresponding quantities. Figures 9 and 10 are views of the report information.

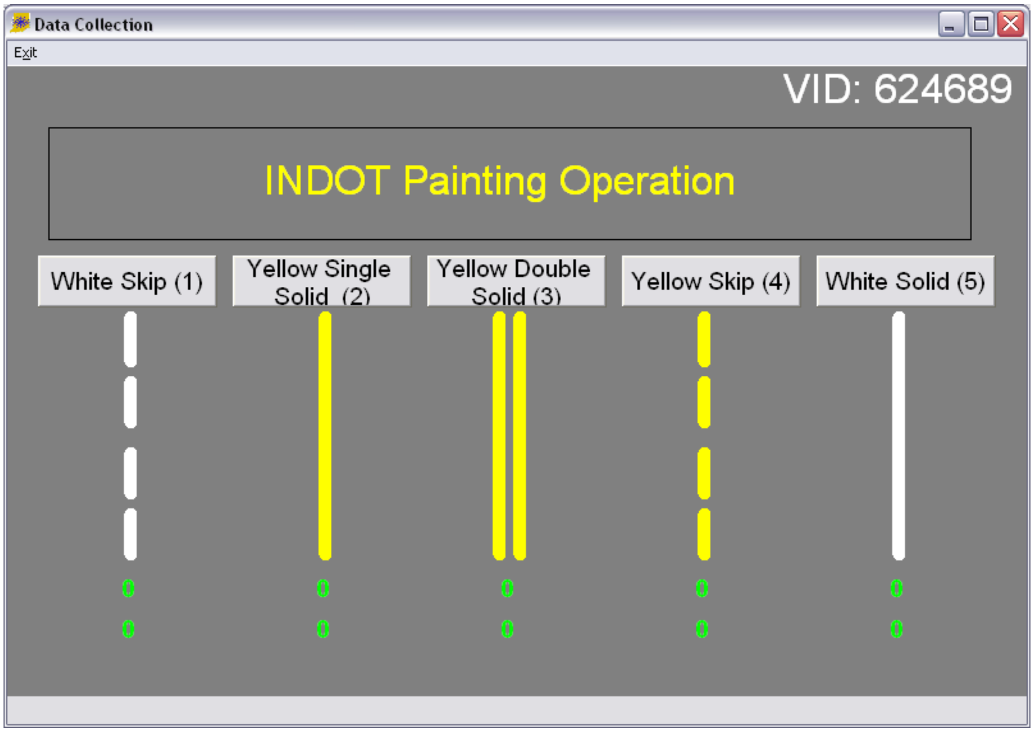

Figure 8. Paint software

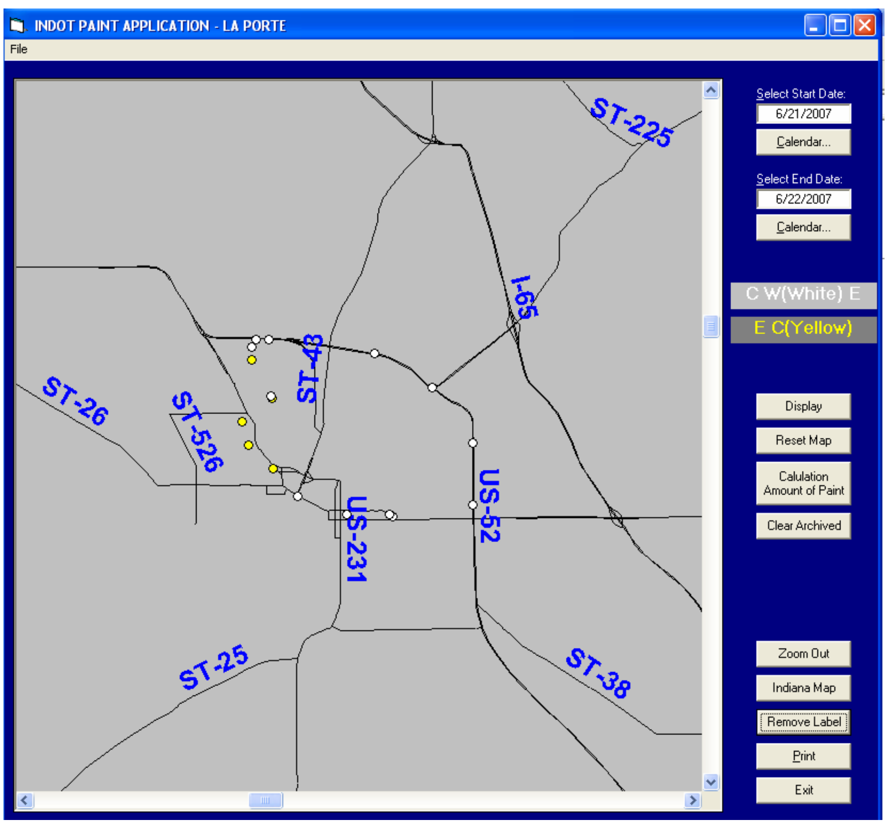

Figure 9. Map-based report

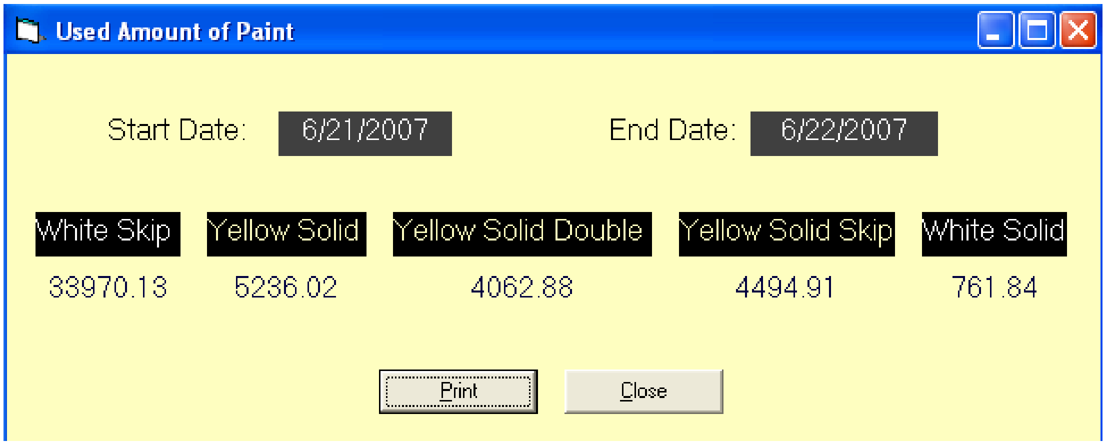

Figure 10. Paint quantities report

CONCLUSIONS

This project's main objective was to develop and test a winter operations system that utilized the statewide wireless network called SAFE-T. The system was tested over a two-year winter period. Other objectives were to develop a maintenance decision support system (MDSS) interface, a hotspot batch data transfer capability, and, in order to utilize the equipment more, a summer application developed.

The first winter season, 2005–2006, four trucks at two locations: two in Monticello and two in Columbus, were equipped and tested. This winter had very few winter weather events to test the system by. Also, when the system was tested, several bugs and software issues were discovered. The drivers had problems logging in and determining if the truck was communicating over the wireless network. Also, they suggested modifications to the software and input screens. As a result of this testing period, changes were made to the software during the summer of 2006.. These changes were

- Revised screen look
- Added plow position – up or down
- Added road temperature
- Added spot application and trouble spot recording
- Removed login
- Eliminated Vehicle ID input

Before the winter of 2006–2007, the application was expanded to three locations and ten trucks. Also in this expansion, three different hardware data collection devices (computers) were to be tested. The testing program looked like this:

- LaPorte: 1 Mini PC, 1 touch screen laptop, 2 ultra mobile tablets
- Monticello: 1 Mini PC, 1 touch screen laptop, 1 ultra mobile tablet
- Columbus: 1 Mini PC, 1 touch screen laptop, 1 ultra mobile tablet

These systems were used for the whole winter season. Screen resolution, icon size, and fonts were adjusted to the optimum to improve the usability of the program in the vehicle.

Winter activities in 2006–2007 were again below normal and most did not occur until February 2007. During this limited time field testing occurred with the following results:

1. Several problems occurred with showing data on the GIS maps. These were resolved and fixed by placing an executable program on the GIS server that archives data to Oracle and places the data on the maps.
2. Drivers experienced fewer problems with the software because it was easier to understand and operate.
3. The modems experienced numerous problems at Monticello and Columbus due to frequency shifts which caused loss of data transfer.
4. It is difficult for drivers to monitor connection status while driving.
5. Drivers preferred laptops.
6. Data transferred successfully into MDSS.
7. The power source of the system was connected to the cigarette lighter of the vehicle for easier disassembly. Based on the evaluation, it is recommended to connect the power cables directly to the vehicle battery because of the adverse road conditions.
8. Change the GIS maps to indicate the latest position of the trucks, and revise some of the legend symbols and colors.
9. Features that managers liked:
    a. Ability to track trucks and retrieve this information at a later date.
    b. Know how much chemical was placed and at what time.
    c. Combining the AVL info with MDSS provides better information to base decisions.
    d. Helps in updating the weather info.
    e. Provides a way that law enforcement & private citizen could see that trucks were out on routes.
    f. It could be a tool to show how much time is spent on keeping roadway safe for motoring public.
    g. Help save on material use by having current weather info.
    h. Tracking employees and trucks to be able to answer calls when a truck or where a truck is on a route and better customer service.

In the end, the project developed many solutions; AVL system, MDSS interface, map-based reports, hotspot data transfer. But it also created many questions. The Study Advisory Committee decided to extend the project to seek solutions to some of the following issues:

- Investigate the alternative of collecting and transferring data through a PC/smart phone device.
- Investigate using the system for summer activities.
- MDSS and INDOT AVL both have a map. Which map best fits INDOT needs, or are both needed, and which one is the most economical option?
- Can other programs (WMS, 511) also use the equipment in the trucks?
- Develop data interface into CARS.
- What are the effects on operations?
- Is it cost beneficial?
- What would be the cost of implementation?
- What would be needed to implement and over what time period?

ACKNOWLEDGEMENTS

The authors express their appreciation to the Indiana Department of Transportation and the Aurora organization for providing funding to perform the research described herein.

Use of Video Feedback in Rural Teen Driving: An Intervention Study

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ABSTRACT

Teen drivers are at high risk for car crashes, especially during their first years of licensure. Providing novice teen drivers and their parents with a means of identifying their risky driving maneuvers may help them learn from their mistakes, thereby reducing their crash propensity. During the initial phase of learning, adult or parental supervision often provides such guidance. However, once teens obtain their license, adult supervision is no longer mandated, and teens are left to themselves to continue the learning process. This study is the first of its type to enhance this continued learning process using an event-triggered video device. By pairing this new technology with parental feedback in the form of a weekly video review and graphical report card, we extend parents' ability to teach their teens even after they begin driving independently.

Twenty-six 16- to 17-year-old drivers were recruited from a small rural high school in Tiffin, Iowa. The district covers a 162 square mile radius, in which nearly all driving occurs on rural highways and gravel roads. Prior to induction into the study, the subjects' driving experience ranged from three months to one year.

Each participant's vehicle was equipped with an event-triggered video recording system made by DriveCam. This system is a palm-sized device that integrates two video cameras (forward and interior view), a two-axis accelerometer, and a wireless transmitter. Video data is continuously buffered 24 hours/day, but the system only writes to internal memory when a 0.55 g lateral or 0.50 g longitudinal threshold is exceeded. All data are automatically downloaded from the device via secure wireless network whenever the participant parks in the high school parking lot. Once downloaded, encrypted data are sent to the laboratory for coding.

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The first nine weeks established a within-subject baseline; no parental or system trigger feedback was given during this time. After the nine-week baseline, feedback was provided to the participant in the form of a blinking LED light whenever the acceleration threshold was exceeded. In addition, teens and parents were sent a weekly graphical summary of events relative to the study peer group that included video of safety-relevant events. The feedback intervention lasted for nine months. After ten months and over 30,000 miles of driving, preliminary findings suggest that combining this emerging technology with parental weekly review of safety-relevant incidents resulted in a significant decrease in events for the more at-risk teen drivers. The baseline data revealed that the participants were divided into two groups: one with a low frequency of events and the other with a high frequency of events. The intervention resulted in a significant reduction in the number of safety-relevant events. In the first nine weeks of the intervention, the drivers reduced their rate of safety-relevant events from an average of 8.6 events per 1,000 miles during baseline to 3.6 events per 1,000 miles. As a group, they cut their safety-relevant events by a little over half in the first nine weeks (58% reduction). The group further reduced its rate of events to 2.1 per 1,000 miles in the following nine weeks (weeks 10 through 18), achieving a 76% reduction rate from the baseline. This drop from 8.6 to 2.1 events per 1,000 miles driven was statistically significant ($t=4.15$, $p<.0007$). The participants averaged 2.0 to 2.5 safety-relevant events per 1,000 miles for the final two nine-week periods. A similar reduction pattern emerged for the incidents, including near-crashes and crashes ($t=4.34$, $p<.0003$).

Of interest is whether the reduction in safety-relevant events during the first nine weeks of intervention was the same for all drivers. The two driver groups reacted differently to the intervention. The 18 low-frequency drivers did not change their behavior significantly, essentially demonstrating a floor effect, maintaining an average of approximately 2.0 safety-relevant events per 1,000 miles driven throughout the baseline and the entire intervention phase. However, the seven high-frequency drivers showed a dramatic 72% reduction, dropping from an average of 23.4 to 6.4 safety-relevant events per 1,000 miles in the first nine weeks of the intervention. After an additional nine weeks of the feedback intervention, the seven high-frequency drivers further dropped their safety-relevant events by 89% from the baseline, averaging 2.6 events per 1,000 miles. They have maintained an average of 3.0 events per 1,000 miles throughout the remaining weeks of the intervention, slightly above the other group. The interaction between driver group and phase was significant ($F(4,92)=37$, $p <.0001$).

A similar pattern emerges for the incidents (including near-crashes and crashes), where the seven high-frequency drivers benefited the most from the intervention, dropping their higher incident rates to almost the level of their low-frequency peers after 18 weeks of intervention. The interaction between driver group and phase was significant, $F(4,92)=24.09$, $p <.0001$. The two most frequent incident types were improper turning or curve negotiation and abrupt braking.

Key words: driving instruction—teen driving—video feedback

Effect of Stiffness Ratio on Slippage Cracking Due to Interlayer Bonding Failure in Hot Mix Asphalt Pavement

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ABSTRACT

Some state agencies, such as the Wisconsin Department of Transportation (WisDOT), have experienced pavement failures that have been attributed to poor bonding at the interlayer. The interlayer nearest the surface of the pavement is where loss of bonding typically occurs. This loss of bonding causes poor load transfer to the lower layers and causes slippage where the surface layer is shoved horizontally. The effect of slippage can be minimized by making the surface layer sufficiently thick or stiff.

In this study, two roads in Wisconsin that experienced varying degrees of slippage distress were analyzed to understand the factors critical to slippage cracking. A total of 17 no-distress and 14 high-distress stations from both roads were studied. This study determined stiffness by backcalculating from falling weight deflectometer (FWD) data and recorded field-observed distresses to identify the factors critical to slippage cracking.

It was observed that the stiffness ratio between the top two layers was higher for no-distress sections than for high-distress sections. Pavements with higher E_1/E_2 values ($E_1/E_2>10$) consistently showed better interlayer bonding performance. Normalized percentage differences in stiffness between full-bond and full-slip pavements appeared to correlate very well with observed distresses. This study provides state agencies with tools to use during pavement design that can help minimize slippage cracking due to interlayer bonding failure.

Key words: asphalt—backcalculation—falling weight deflectometer—interlayer bonding—slippage cracking

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SUMMARY OF THE RESEARCH

Background

Slippage cracks are caused by insufficient pavement stiffness and thickness or a weak bond between the surface course and the layer below, and the complex interaction between these factors makes it difficult to control slippage cracking. This study utilizes backcalculated stiffness values and field-observed distresses to understand the factors critical to slippage cracking and to develop guidelines that can be used during pavement design to minimize slippage cracking that results from interlayer bonding failure.

Objective

The objective of this study was to provide pavement design guidelines regarding the stiffness ratio between the top two layers of a pavement system that can help minimize slippage cracking due to interlayer bonding failure.

Research Approach

FWD data can be used to estimate the pavement layer stiffness values for both no-distress and high-distress sections. This estimation of layer stiffness can be performed through a method called backcalculation. Using measured surface deflections, backcalculation programs determine the pavement layer stiffness. Figure 2 shows that, for the same FWD load, the deflections of high-distress sections (fully slipped, FS) are higher than those of no-distress sections (fully bonded, FB). The phenomenon of slip is thus manifested in FWD data.

Theoretically, the deflection basin from the FWD data of a fully bonded pavement structure will be much lower than the deflection basin of a fully slipped section. A fully bonded pavement structure will transfer load better through the pavement system and hence will utilize the structural capacity of all layers effectively. On the other hand, a poorly bonded pavement system will be relatively more flexible due to poor load transfer. As explained above, the higher stiffness ratio minimizes the impact of slip. The difference in the deflection basins of fully slipped and fully bonded sections will be greater for pavement structures with low stiffness ratios, as shown in Figure 1. The researchers have used this concept in this study to provide the appropriate stiffness ratio needed to minimize effect of slip.

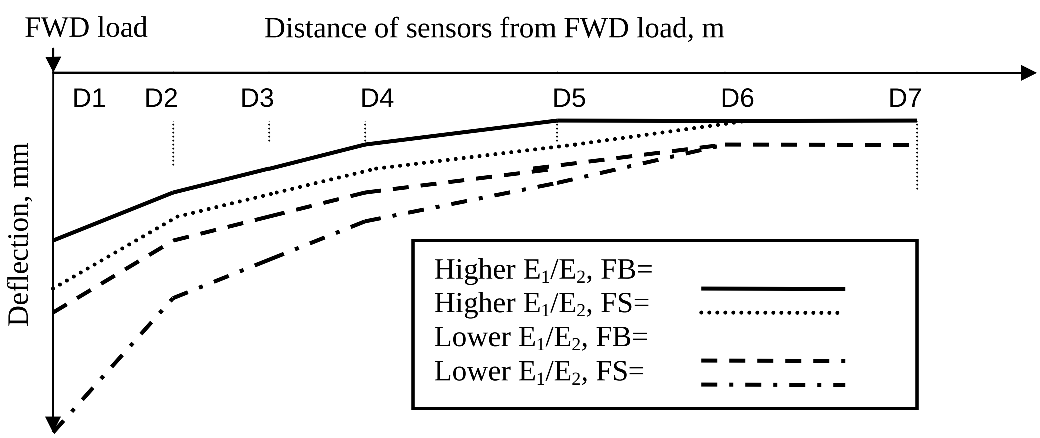

Figure 1. Measured deflection due to FWD load for full-bond and full-slip interface conditions

The following tasks were conducted to achieve the objective stated above:

1. The stiffness values of the different layers were backcalculated for the no-distress pavement sections in full-bond condition, which is the actual condition for no-distress sections. Then, the stiffness values of the HMA layer were calculated by assuming a full-slip condition for no-distress sections, keeping stiffness values in a full-bond condition for all other layers.
2. The stiffness values of the different layers were backcalculated for the high-distress sections in full-slip condition, which is the actual condition for high-distress sections. Then, the stiffness values of the HMA layer were calculated by assuming a full-bond condition for high-distress sections, keeping stiffness values in a full-slip condition for all other layers.
3. The percentage differences of stiffness values between full-bond and full-slip conditions for no-distress sections were correlated to the ratios of stiffness between the top two layers.
4. The percentage differences of stiffness values between full-bond and full-slip conditions were verified with the strain differences between full-bond and full-slip for a specific stiffness ratio.
5. The normalized percentage differences of stiffness values between full-bond and full-slip conditions (for both no-distress and high-distress sections) were correlated to the ratios of stiffness between the top two layers.

Summary of Findings

A summary of findings based on the analysis is presented below:

1. The stiffness ratios between the top two layers for no-distress sections were between 5 and 65, which were higher values than those of high-distress sections (between 1 and 7); this was observed for all sections where the second layer's stiffness was greater than 0.138 GPa.
2. The percentage differences of stiffness between full-bond and full-slip conditions may not be an accurate indicator of the effect of slippage.
3. The normalized percentage differences of stiffness ($P.D./E_1$) between full-bond and full-slip conditions appeared to correlate very well with observed distresses.
4. A very strong inverse correlation was observed between $P.D./E_1$ vs. E_1/E_2, with a root mean square value of the curve ($P.D./E_1$ vs. E_1/E_2) of 0.94.
5. The stiffness ratio appeared to correlate inversely with observed distresses. Higher E_1/E_2 ($E_1/E_2>10$) consistently showed better interlayer bonding performance.
6. When the stiffness ratio was greater than 10, the differences in the slopes of the curves ($P.D./E_1$ vs. E_1/E_2) were almost zero. Since $P.D./E_1$ is directly related to the effect of slip, when E_1/E_2 were greater than 10, the pavement was not as adversely impacted from poor interlayer bonding.

Conclusion

If the stiffness ratio between the top HMA layer and the second layer is greater than 10 during design, and if the second layer stiffness value is greater than 0.138 GPa, the pavement will be less affected by slippage than when the stiffness ratio is less than 10.

Stabilizing Granular Shoulders with Soft Foundation Soils: Iowa Case Histories

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ABSTRACT

Granular shoulders underlain by soft foundation soils can undergo severe rutting when subjected to traffic loads. Traffic loads induce normal and shear stresses, which cause bearing capacity failure of the foundation soil and, consequently, surface rutting. Considerable rutting can be a threat to drivers and is difficult to maintain. Recently, a field reconnaissance study was carried out to evaluate the causes of granular shoulder problems in Iowa. The study revealed that out of 21 problematic granular shoulders, 50% suffered from a soft subgrade layer where the California Bearing Ratio (CBR) was 10 or less. Granular shoulders with soft foundation soils are currently maintained by adding granular material. This, however, is a temporary solution, since it does not address the soft foundation condition. To evaluate alternative rehabilitation procedures, two test sections overlying soft subgrade soils were constructed and monitored. The first section was stabilized by mixing the top 300 mm of the subgrade layer with about 20% class C fly ash. The second test section was stabilized by placing three different biaxial geogrids at the interface of the granular and subgrade layers. To monitor the performance of both sections, field tests including dynamic cone penetration, Clegg impact, and plate load tests were performed. Field results indicate the success of both stabilization techniques in alleviating shoulder rutting. In this paper, observations from a field reconnaissance study, the construction and monitoring of two shoulder test sections, and recommendations for repairing shoulders with similar problems are discussed.

Key words: granular shoulders—rutting—shoulder maintenance—subgrade stabilization—unpaved shoulders

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City of Ames, Iowa, Quiet Zone Assessment

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ABSTRACT

The City of Ames, Iowa, is investigating options to minimize the impacts of train horn noise throughout the community. The Quiet Zone Final Rule, issued by the Federal Railroad Administration (FRA) in June 2005, offers an opportunity for cities to accomplish this objective. The Ames Quiet Zone Assessment analyzed the current safety risk levels and the necessary safety measures required to compensate for horn cessation if a federally approved quiet zone were established.

The City of Ames has two separate rail lines within the city, requiring the 12 at-grade crossings to be evaluated as two separate quiet zones (east/west and north/south). There are six crossings in each potential quiet zone. Based on the analysis performed, the following recommendations are made for City Council's consideration.

East/West Quiet Zone

The east/west railway line runs through downtown Ames and carries approximately 66 trains per day at speeds of up to 60 miles per hour. All of the crossings along the east/west rail line are currently equipped with the minimum quiet zone requirements, thus qualifying the entire corridor as quiet zone eligible. As part of a separate Iowa Department of Transportation project, four-quadrant vehicle gates will be installed at the Duff Avenue crossing in 2007. In addition to the Duff Avenue improvement, by extending both the north and south medians at Scholl Road six ft., to within one ft. of the existing two-quadrant vehicle gate arms, the quiet zone risk index (QZRI) for the east/west line falls below the risk index with horns (RIWH). By lowering the QZRI below the RIWH, the east/west quiet zone would be deemed to be as safe as if the train horns were still sounding.

North/South Quiet Zone

Before determining a course of action for establishing a north/south quiet zone, three questions needed to be answered by the City Council. (1) What is the desired risk level that the city would like to achieve along this rail line? (2) How much of a financial commitment is the city willing to make along this rail line? (3) Are crossing closures an acceptable method to improve safety and reduce cost? After answering these questions, SRF Consulting Group, Inc. created six scenarios for establishing a quiet zone along this rail line.

City Council Meeting Results

Staff from SRF Consulting Group, Inc. presented the Quiet Zone Assessment Report to the Ames City Council on December 12, 2006, highlighting the improvements necessary to obtain a quiet zone on both

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the east/west rail line and the north/south rail line. At the following City Council meeting on December 19, 2006, the City Council unanimously decided to move forward with the implementation of an east/west quiet zone, which includes the following improvements:

- Installation of four-quadrant vehicle gates at the Duff Avenue crossing
- Extension of the existing medians at the Scholl Avenue crossing to within one ft. of the existing gate arms
- Extension of the south median at the North Hazel Avenue crossing to within one ft. of the existing gate arm, removal of the abandoned railroad track, and reconstruction of that section of roadway
- Extension of both existing medians to within one ft. of the existing gate arms at the North Dakota Avenue crossing

At the same meeting, the City Council also directed staff to move forward on the north/south quiet zone, which includes the planned construction of nontraversable medians and constant warning time at Bloomington Road, as well as the creation of a plan to fund the minimum quiet zone requirements (i.e., two-quadrant vehicle gates and constant warning time detection) needed at the 9th and 16th Street crossings.

For the City Council to establish a quiet zone (or zones), a number of implementation activities would be required. The first step is preparing the Quiet Zone Notice of Intent and distributing it to the necessary stakeholders. Following implementation of improvements, a Notice of Quiet Zone Establishment must be filed with the FRA to finalize the quiet zone process. This filing process will be relatively quick and easy, since the assessment process has produced most of the data and information required. Along with these requirements, the City of Ames will also need to install Manual on Uniform Traffic Control Devices (MUTCD)–compliant advanced warning signs, advising motorists that “train horns are not sounded” at the designated crossings.

For copies of the Ames Quiet Zone Assessment Study, please contact the City of Ames Public Works Department.

Key words: Iowa—quiet zones—railroads—safety

Measurement of Visual Attention and Useful Field of View during Driving Tasks Using a Driving Simulator

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ABSTRACT

The main aim of this work was to obtain a better understanding of drivers' perceptions of hazard information in selected traffic situations. To contribute to this aim, the relationship between visual attention and the range of the useful field of view was considered by measuring eye movements during simulated driving. Histories (scan paths) of the eye movements were drawn from eye position data obtained by an eye tracking system, superimposed on driving road scenes viewed from the front side seat in an automobile. This procedure enabled one to evaluate the relationship between visual attention and eye movement, such as saccades and fixations, of the driver under various traffic events and road conditions. Results showed that visual stimuli in the peripheral vision affected eye movements during driving. In situations where there are few objects to be attended to (e.g., driving on a straight road), eye movement frequencies were smaller, and fixation durations were longer, than in situations where there are many objects (e.g., driving through an intersection). Based on these results, it is estimated in this paper, the range of the useful field of view during driving. The method proposed in this paper would be useful for simulating safe driving practices and analyzing human factors for drivers' awareness.

Key words: driving simulator—eye movements—fixation duration—useful field of view—visual attention

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INTRODUCTION

To drive an automobile safely, it is important for a driver to perceive a hazard quickly (Brown and Groger 1988; Lalley 1982). A hazardous situation includes several of the factors responsible for accidents in a traffic situation. Therefore, hazard perception is needed in the early stages of human information processing (Renge 1998; Soliday 1974 and 1975). Although most information needed for automobile driving is obtained through visual input (Hills 1980), early studies showed no correlation between driving performance and visual functions, such as a dynamic and a static visual acuity (Hills 1980). On the other hand, recent studies (Miura 1992; Owsley et al. 1998) suggest that driving performance depends on visual attention, which is also associated closely with the range of the useful field of view. Useful field of view is defined as the visual field in which an observer is able to perceive visual stimuli quickly and accurately during complicated perceptual-motor tasks, such as reading and driving. To our knowledge, however, there have been few studies examining visual attention during driving with conditions controlled experimentally (e.g., speeds of driving).

The main aim of this work was to examine visual attention during driving using a driving simulator. Since eye movements depend on visual attention, it is useful to analyze the relationship between eye movements and road scenes. Another aim of this work was to obtain a better understanding of hazard information for forecasting a particular human behavior, such as visual attention during driving. Further in this study, the range of the useful field of view was estimated, based on a new hypothesis suggested by present and preliminary experiments (Miyoshi, Nakayasu, and Suzuki 2007), that the range of the useful field of view may be inversely proportional to the fixation duration on the driving scene. Although it is difficult to measure the range of the useful field of view directly during automobile driving because of the difficulty of constraining eye movements, it is easy to record fixations and their durations. Therefore, it may be possible to formulate the range of the useful field of view during driving from eye movements, such as the distance between two successive fixation points (i.e., saccades) and those durations, by superimposing the history of the eye movements (scan paths) onto driving road scenes. Since the method proposed in this study can estimate the range of the useful field of view from eye movements, we would be able to investigate it under various traffic conditions, such as road geometry, weather, and driver experience.

EXPERIMENTAL SETUP

Participants

Participants included 14 males, consisting of 11 males from 20 to 31 years of age (mean age of 22.6 ± 3.0) and 3 males from 60 to 65 years of age (mean age of 62.7 ± 2.5). All participants had regular-class automobile licenses in Japan with normal or corrected-to-normal vision. All participants received an explanation of the experiment and gave informed consent before participation.

Driving Simulator

The automobile driving simulator (DA-DOO: HONDA Motor Co. Ltd.) was used to simulate driving scenarios. Figure 1 shows the driving simulator used in this study. This simulator is typically used for driver trainees or novices to learn safe driving techniques. Specifications are shown in Table 1. The simulator is implemented on an advanced six-axis motion base (sway-motion device) that can simulate closely automobile dynamics in real traffic situations. The visual scenes of driving on a road from the viewpoint of a driver are presented on a frontal screen and three mirror CRTs. The distance from the front screen to the driver’s seat is 1,511 mm. The simulator simulates driving courses with a variety of conditions, such as weather and car and human actions (e.g., dashing out in front of the automobile). In this study, both urban road and highway courses (Figures 2 and 3) were selected. These courses consisted of several parts of the road: a straight road, bending road, and cross section, into which dangerous situations could be inserted. The participants drove the simulator in automatic transmission (AT) mode.

Figure 1. Driving simulator and eye tracking system

Table 1. Specification of Honda DS-D00

Front view	Wide field (138 deg.) screen projection type
Rear view	3-mirror independent LCD display
CG	Redraw speed: 30 to 60 frame's
Mechanism motion base	Six axis motion base system using G cylinders Control
Frame	Light weight space frame structure with aluminum extended mechanism
Body	Rear open structure fixed with FRP mold
Operation system	Steering device with reactive force control, Accelerator, clutch, brake, simulation, mechanism
Mission	AT/MT switch mechanism
CG computer	Cuamum3D Alchemy(1.5 million polygons)
Control system	6-axis servo amp
Dimensions	2,440mm(D)×2,280mm(W)×1,855mm(H)

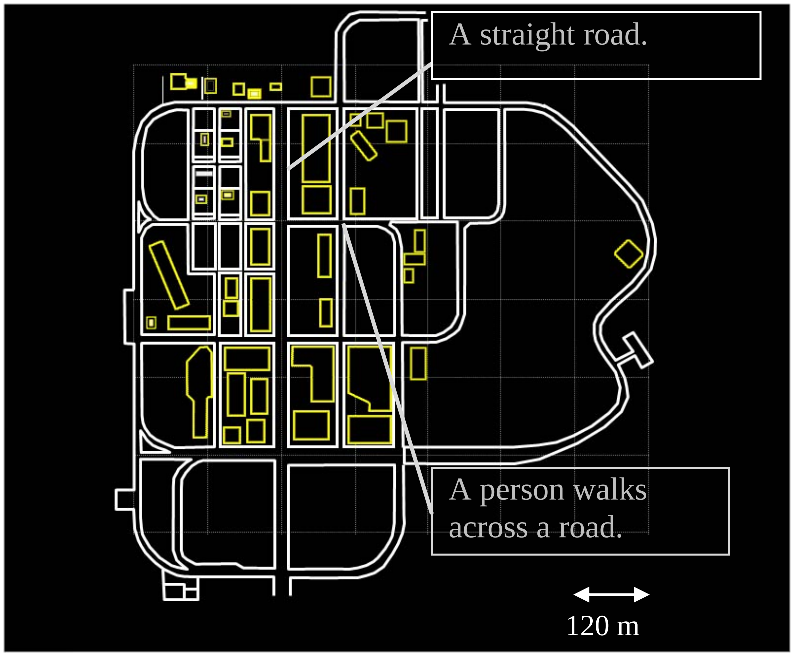


Figure 2. Urban road course in simulator

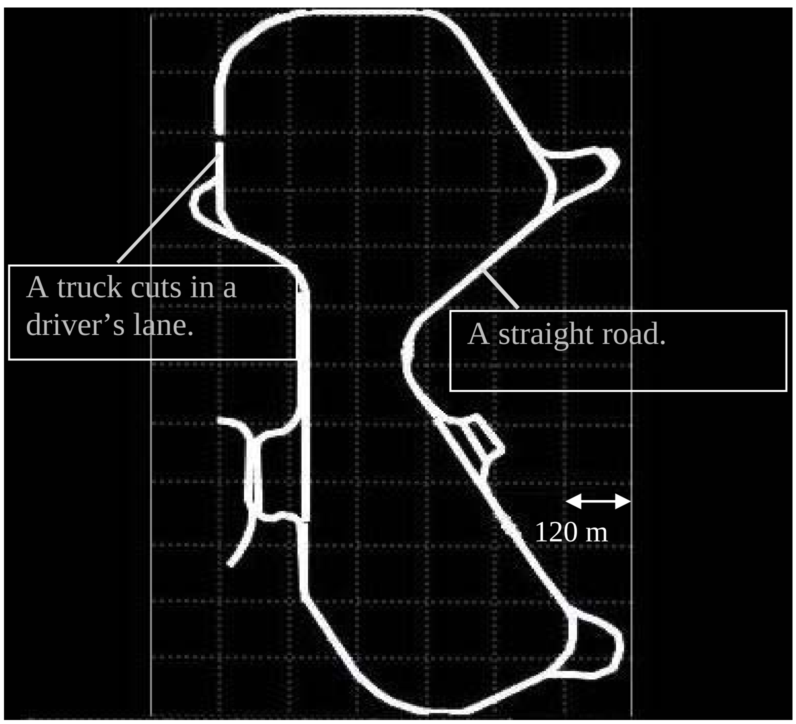


Figure 3. Highway course in simulator

Eye Tracking System

During simulated driving, eye movements in the driving road scene were recorded by a head-mounted eye tracking system (EyeLink II: SR Research Ltd.; see also Figure 1). Tables 2 and 3 show the specifications and functions of the system. The system consists mainly of several components: EyeLink Host PC, EyeLink Control PC, scene camera, overlay box, and DV converter, whose relationships are shown in the block diagram in Figure 4. Two eye cameras track the right and left eyes separately. A scene camera on a head band records the visual scene that the participant views. In the scene camera, either a color or high-resolution (720 by 480 pixels) black and white scene camera is used to generate 30 frames of a scene image per second. The scene image is also streamed to the overlay box, in which the participants' gaze position (indicated by a filled circle) is superimposed onto the visual scene. The host PC converts gaze position data, obtained by the eye cameras, into position data onto the visual scene image captured by the head-mounted scene camera. The host PC also analyzes eye movement events in real-time. These data are sent, through an Ethernet link, to the control PC, where they are stored. The control PC is also used for calibrating the eye tracking system.

Table 2. Specifications of EyeLink II

Sampling Rate	500Hz (Pupil One) 250Hz (Pupil and Corneal Reflection)
Error of Fixation	less 0.5°
Accuracy of Pupil Size	for diameter 0.1%
Range of Trace	Horizontal ±30° Vertical ±20°
Data File	EDF
Rate of data transfer	Pupil: Filter Off: 3 ms Nomal:5 ms Filter High: 7 ms Pupil+CR: FilterOff: 6 ms Normal: 10 ms Filter High: 14 ms
Marker of infrared rays	900nm
Eye Camera	925nm 1.2W/cm2
Weight	420g

Table 3. Functions of EyeLink II

Image Processing	Fully Digital
Pupil Tracking	Hyper acuity
Corneal Reflection Tracking	Hyper acuity, ultra low noise
Resolution(Gaze)	<0.005°(pupil and CR)
Velocity Noise	<0.5°average
Pupil Size Resolution	0.1% of diameter
Eye Tracking Range	±30°horizontal, ±20°vertical in pupil only mode.
Gaze Tracking Range	±20°horizontal, ±18°vertical
Head Tracking Distance	40-140cm (standard), -300cm (Special markers)
EDF File and Link Data Types	Eye position, HREF position, gaze position, Pupil size, buttons, messages, digital inputs
Online Eye Movement Analysis	Saccades, fixations, blinks, fixation updates

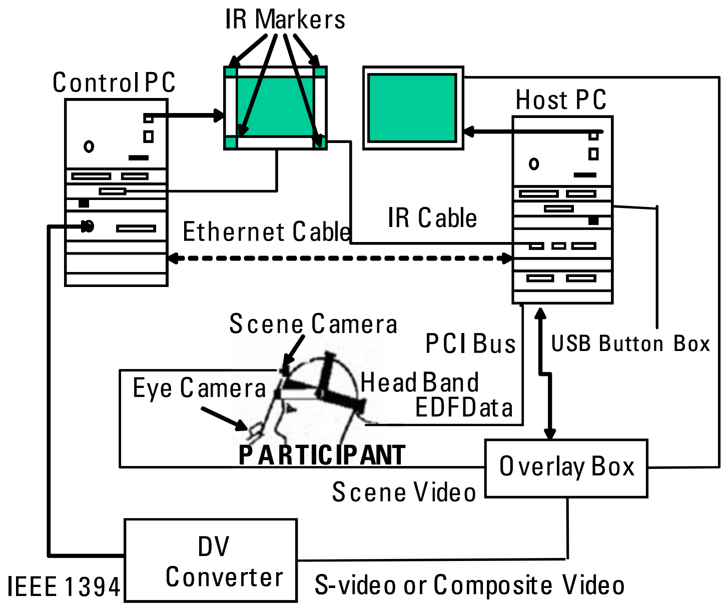


Figure 4. Block diagram of eye tracking system

Procedure

Participants were seated on the seat of the driving simulator. After the participants practiced driving on the urban road and the highway courses (Figures 2 and 3), an experimental session was conducted. In the experiment, the participants were instructed to drive on the driving courses safely.

Eye Movement Recordings and Analyses

In this study, horizontal and vertical eye movements of the participants were recorded from both the right and left eyes. Before beginning the experiment, the participants were asked to fixate on nine points presented on the control PC display to calibrate the eye tracking system. All data were stored on the control PC and analyzed offline by a computer program that drew histories or scan paths of eye movements and then calculated the total distances of the eye movements and the fixation durations.

From the eye position data, histories of the eye movements for events involving the car or human actions were drawn. Eye position data 2 seconds before and after the participants made saccades toward a car or human event to be perceived were used. In addition, the histories of eye movements during driving on a straight road on which there were few objects to be viewed were calculated as a comparison condition (see Figures 2 and 3). The total distance of eye movements was calculated as the differences between the eye positions. A fixation was defined as eye movements of less than 30°/sec. velocity for more than 100 ms. A fixation duration was calculated by the difference in time between two successive saccades, before and after the fixation, defined as eye movement of more than 30°/sec. velocity.

EXPERIMENTAL RESULTS

Histories of Eye Movements

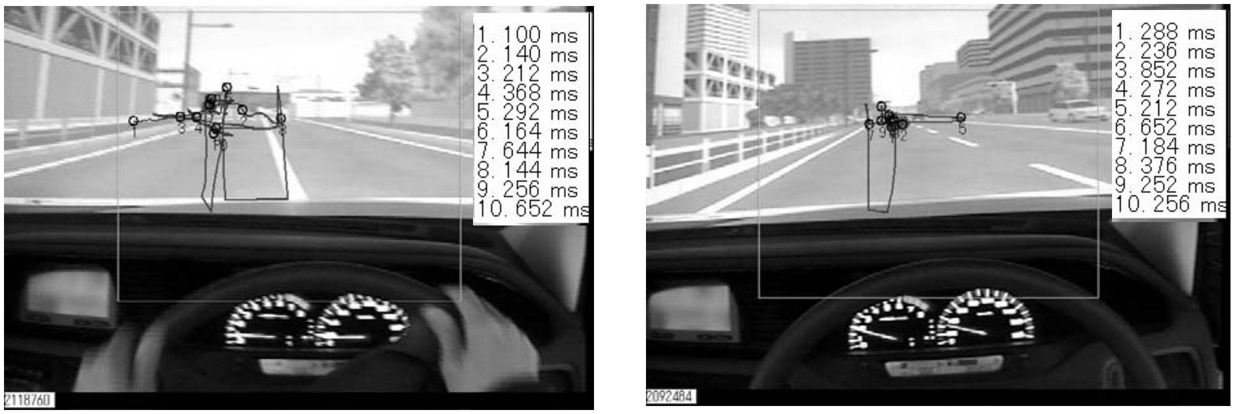
Figure 5 shows representative eye movement histories for one participant approaching a crosswalk in which there was a person on the left side of the crosswalk and driving on a straight road in the urban road course. For the crosswalk, Figure 5(a), the participant's eye moved frequently in various directions, even when the person did not start walking. For the straight road, Figure 5(b), the participant's eye positions were less variable than those at a crosswalk, shown in Figure 5(a), although the number of fixations was the same. Figure 6 shows representative histories of the eye movements of the same participant on the highway course. Similar tendencies to those in the urban road course were observed, although the number of the fixations was different between two scenes. At a merging section, Figure 6(a), in which a truck cut into the participant's lane, the participant changed his fixation points frequently around the truck. On the other hand, at a straight road, Figure 6(b), the participant kept his fixation point forward on the road.

Total Distance of Eye Movements

Figure 7 shows mean total distance of eye movements (in degrees) during events such as the crosswalk and the merging section and the straight section of the urban road and highway courses. As shown in the figure, in both the urban road and highway courses the mean distances of the eye movements were longer for the event than for the straight section, although there were individual differences (indicated by error bars). It is seen from the experimental results that the total distance of the eye movements is not dependent on the differences of the driving course (i.e., the urban road and highway courses).

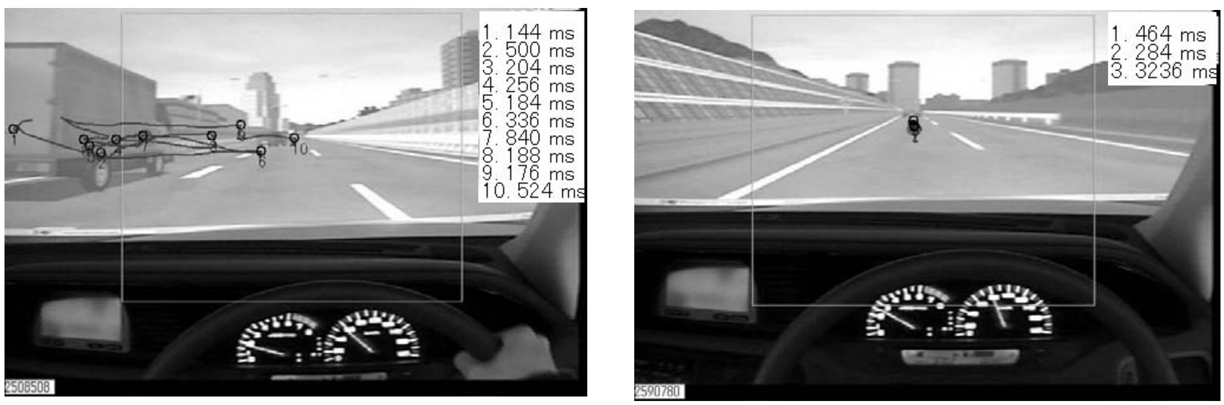
Fixation Duration

Figure 8 shows mean time of fixation durations for the event and the straight section in the urban road and highway courses. There were no differences between duration times for the event and the straight section on the urban road. On the other hand, in the highway course the mean time of fixation durations was longer for the straight section than for the event section, although it is noted that there were relatively large individual differences.



(a) The urban road: the crosswalk **(b) The urban road: the straight road**

Figure 5. Representative histories of the eye movements of one participant (N.Y.) in the urban road course



(a) The highway: the merging section **(b) The highway: the straight road**

Figure 6. Representative histories of the eye movements of one participant (N.Y.) in the highway course

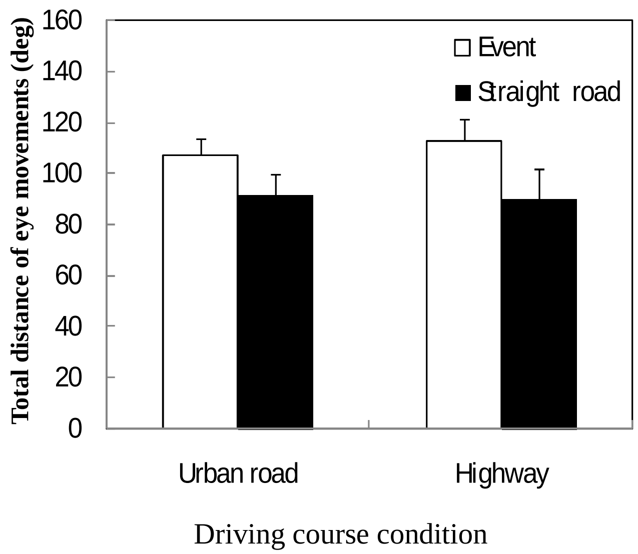


Figure 7. Mean length of the eye movements (error bars indicate standard errors of the mean)

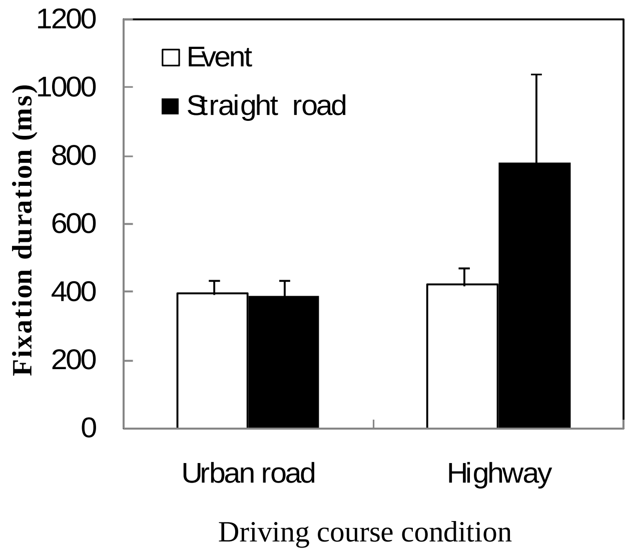


Figure 8. Mean fixation durations (error bars indicate standard errors of the mean)

DISCUSSION

Eye Movements and Visual Attention

In this study, eye movements during automobile driving were measured and analyzed in the context of selected driving road scenes in order to examine visual attention during driving. The present results of the eye movement histories showed that the changes of the fixation point were more frequent for the events section, where there were many objects to be attended to (e.g., at a crossroad section), than for a road where there were few objects (e.g., at a straight section). The total distances of the eye movements and the fixation durations at the event section were longer than those at the straight section in both road conditions. These results were consistent with preliminary experiments (Miyoshi et al. 2007) showing that the frequencies of saccades were higher, and the fixation durations longer, at a section where the amount of changes in visual stimuli of the peripheral vision was relatively larger (e.g., a winding section) than at another road section where the amount of the changes was small (e.g., a straight section). Since research suggests that visual attention moves along with the eye movements (Kowler 1990), it is possible that the drivers may have adopted different attentional strategies for visual search during driving, depending on the visual scenes encountered.

Formulation Estimating the Range of the Useful Field of View

As mentioned in the introduction of this paper, it is quite difficult in driving situations to measure the range of the useful field of view, since usual method measuring useful field of view requires an observer to keep his eye stationary on a fixed fixation point. In the present study, the range of the useful field of view was estimated from eye movements. The present results suggest that the attentional strategies for visual search may change, depending on the visual scene. The changes in the attentional strategies may be because of the changes in the range of the useful field of view, which became narrower with increasing numbers of objects (Miura 1992). Many resources for visual attention are required for visual information processing when the visual objects to be processed are presented in the visual field. Therefore, the range of the useful field of view may be treated as a tradeoff with the depth of the information processing of the visual objects (Miura 1992). Based on the results of the present and the preliminary studies (Miyoshi et al. 2007), we hypothesize that the saccadic amplitude reflects the ranges of two useful fields of view, not a single one (Miura 1992) before and after the saccade. Furthermore, the range of the useful field of view would be inversely proportional to fixation durations; as the fixation duration increases, the information processing increases, but its range would decrease. Following these hypotheses, the range of the useful field can be defined as follows:

$$r_A = \theta_{AB} \times \frac{t_B}{t_A + t_B} \tag{1}$$

$$r_B = \theta_{AB} - r_A \tag{2}$$

In the equations, r_A and r_B indicates the radius of the range of the useful field of view for two successive fixation points, A and B, as shown in Figure 9, where θ_{AB} indicates saccadic amplitude, t_A and t_B indicate the duration times of the successive fixations at points A and B. According to this equation, the range of the useful field depends on the saccadic amplitude. However, since the size of the visual stimuli decreases with the spatial depth of objects, the saccadic amplitudes, particularly during driving, may be affected by the depth of the objects; the amplitude of saccades decreases with the increasing spatial depth of the objects. To correct the effects of the spatial depth of the objects, we calculated the distance from the driver's eye to the objects to which the participants made saccades by calculating the difference in the vertical positions in pixels. Using approximate expression, estimated for a straight road course with distance measures (in meters), we converted the differences in the vertical positions into the depth distance. Based on the estimated distance of the objects, the ranges of the useful field of view were corrected in a way that the range of the field of view could be estimated on the same depth from the eye of the participants.

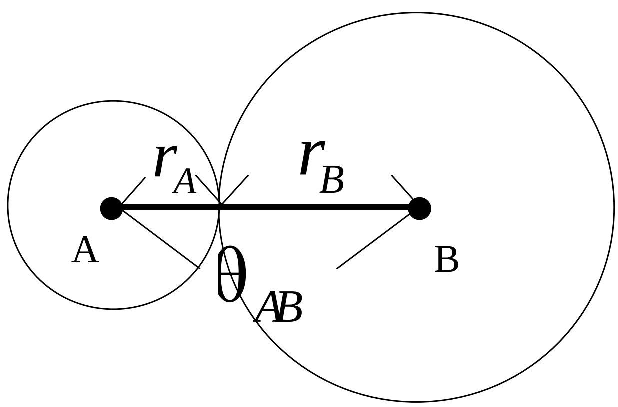
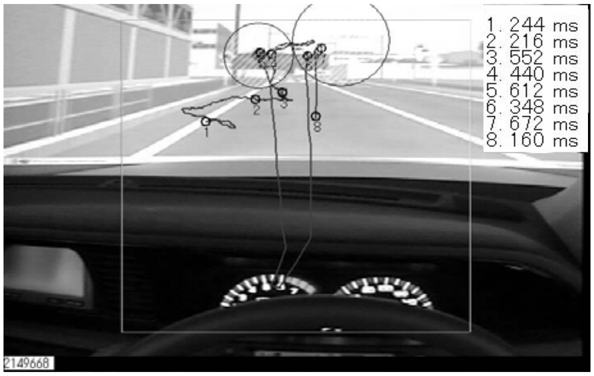
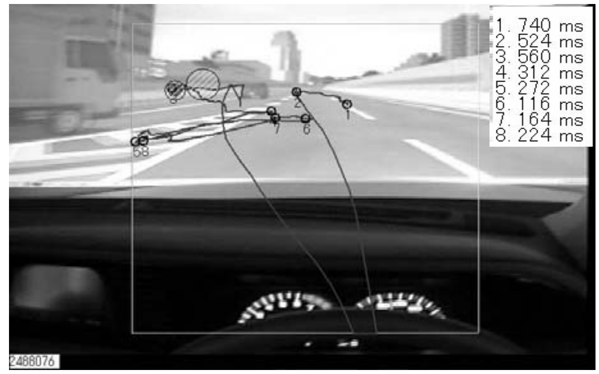


Figure 9. Relations of the parameters used in this study

Following the procedures mentioned above, the range of the field of view was estimated. Figure 10 shows the results for two different participants at the crosswalk in the urban road and at the merging section in the highway course. As shown in the figure, the range of the useful field of view was narrower for the highway than for the urban road courses. Figure 11 shows the mean radius of the field of view at the event in the urban and in the highway courses. Similar tendencies to those in Figure 10 can be observed in Figure 11. Again, the radius of the field of view was narrower in the highway than in the urban road courses.



(a) The urban road course: the crosswalk



(b) The highway course: the merging section

(c) The urban road course: the crosswalk

(d) The highway course: the merging section

Figure 10. Range of useful field of view before and after the saccades at the crosswalk on the urban road and at the merging section on the highway course, for participants T.Y (a) and (b) and S.M. (c) and (d)

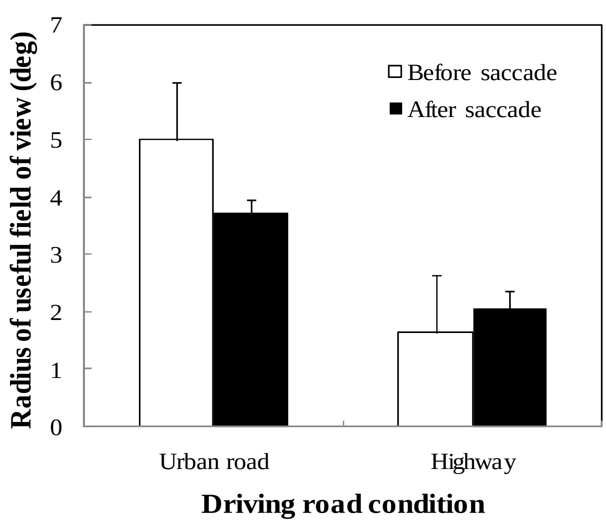


Figure 11. Mean radius of useful field of view before and after the saccade at the event on the urban road and highway courses (error bars indicate standard errors of the mean)

The results of the useful field of view were not consistent with those in Miura (1992), suggesting that the useful field of view is not affected by the speed of driving. The reason for the discrepancies between the present study and Miura's (1992) study is not clear. One possibility may be due to differences in participants. Miura (1992) used highly trained drivers, while this study used experienced or inexperienced drivers. Therefore, it is possible that in Miura's (1992) study the participants' perceptual and cognitive skills, such as anticipation for a risk, may have cancelled out the negative effect of speed on the useful field of view. Indeed, research suggests that the visual search of drivers depends on expectancy or anticipation skills based on their experiences (Hills 1980). In further studies, examination of the possible effects of the participant's skill and experience on the useful field of view will be important.

CONCLUSION

The present study examined the relationship between eye movements and driving road scenes. The results indicated that the eye movements depended on the stimuli in the peripheral vision. This finding suggests that visual attention during driving may scatter in a situation where there are many objects to be processed. In this study, we also attempted to formulate the range of the useful field of view from eye movements. The results showed that the useful field of view was narrower for the highway course than for the urban road course. The method of estimating the range of the useful field of view, proposed in this study is tentative, and further studies will be needed to explore its applicability and validity in various traffic conditions, such as road geometry, weather conditions, and driver experience or skills.

ACKNOWLEDGMENTS

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Pavement Marking Management: Local Agency Practices

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ABSTRACT

Providing good pavement markings is an essential component of safe and efficient travel on Iowa's public roadways. Based on a recent Iowa Department of Transportation project, which has focused on pavement marking performance, local agencies are cautioned in choosing marking materials without field verification of performance in terms of durability and retroreflectivity.

Local agencies rely heavily on contractors to apply pavement markings and currently lack the tools to clearly identify marking conditions systemwide, to select the appropriate combinations of markings to apply based on these needs, and then to track performance and budget over the lifetime of the marking material.

The Iowa Highway Research Board is funding a study to look at local agency pavement marking practices and investigate the feasibility of developing retroreflectivity guidelines, a pavement marking application matrix, and quality control issues (contractor- or agency-applied material) for local agencies in Iowa.

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

As part of this project, the research team collected pavement marking inventories from three local agencies in the fall of 2006 (two counties and one city). Two more agencies will be started this spring. Pavement marking retroreflectivity data were collected on Dallas County, Marion County, and City of Ames roads in the fall of 2006. Measurements were taken approximately every half-mile on the county roads and every 500 to 1,000 ft. on the city roads. White edgelines, white skip lines, and various types of yellow centerlines were collected. The pavement marking retroreflectivity was measured with a Delta Retrometer LTL-X to assess the quality of the pavement markings. The data was stored in a GIS environment to allow for easy viewing and analysis of the data. This abstract will address how the retroreflectivity data will be used to help local agencies in developing a marking program to address roads with the most needs and to contrast the performance of different materials used in the agency's pavement marking program. The abstract will also cover new materials that will be used in demonstration sites this summer. The goal is to test different pavement marking paint, bead, and application method (surface-applied or grooved) combinations to assess performance and durability and provide local agencies with options in terms of an application matrix to be used for determining future needs.

Key words: pavement marking—retroreflectivity

Analysis of Run-Off-Road Crashes in Relation to Roadway Features and Driver Behavior

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ABSTRACT

The state of Wisconsin has been collecting crash information for the entire state trunk highway system that covers over 11,000 miles of roadways. This study examined crashes that occurred between 1998 and 2002, located based on a state-developed linear referencing system. A method (PRÈCIS), originally developed to systematically identify crashes on undivided state trunk highways (STH) and compute crash rates, crash densities (crashes/mile), and other safety statistics at any given point along a STH using a floating highway segment, was utilized. The study expanded the PRÈCIS application to divided highways and established relationships between driver actions that lead to a run-off-road crash and roadway information collected from the State Trunk Network log database. Information such as shoulder width, pavement condition, and roadside features indexed by mile point was examined. The crash database houses over 250,000 crashes that occurred over the analyzed five years, allowing a meaningful analysis of rural low-frequency crash types such as run-off-road crashes. The study merged crash and roadway databases and provided results on a linear highway- and mile point-indexed tabulation system. The results were also transferable to Geographical Information Systems for presentation.

The present analysis describes three tasks that demonstrate applications of the PRÈCIS algorithm and database: (1) development of average and 95th percentile crash rates for targeted crash subsets (e.g., rural, undivided, two-lane highways with 10 ft. shoulders); (2) identification of highway sections for safety improvements where crash rates exceed a set threshold value; (3) identification of the point of diminishing safety returns for highway improvements.

Key words: crash rate—safety improvement—shoulder width—state-wide safety

PROBLEM STATEMENT

The Wisconsin state trunk highway (STH) system covers approximately 12,000 highway miles, of which approximately 10,000 miles are classified as rural highways. The extent of the rural highway system and the relatively lower traffic volumes on these highways contribute to a wide scatter of crashes. It is, thus, a challenging job to identify “unsafe” highway segments using manual or even simple database searches, if a comprehensive state-wide safety evaluation based on crash rates is desired.

The Wisconsin Department of Transportation addressed this challenge and sponsored the development of an automated procedure (PRÈCIS) to analyze widely scattered crashes, based on a floating highway segment analysis method: crash rates are established for the entire STH system, using a one-mile highway length that is “floated” or moved over every highway in the system in small, even, increments at a time—establishing a crash rate for every 1/100th of a mile (point), based on the number of crashes and traffic volumes on short segments of the highway on either side of each point.

Because a number of highway features (such as shoulder widths, number of lanes, lane widths, median information, light posts and bridges) are indexed using a linear referencing system that can be correlated with PRÈCIS, it is possible to develop crash rates just for highway segments where a specific feature is present, for example segments with guardrail, or even contrast crash rates between segments with guardrail versus those using cable guard.

RESEARCH OBJECTIVES

The present effort demonstrates three types of statewide tasks that can be performed using the extensive PRÈCIS database. The first task presents crash rates developed for a targeted subset of crashes (driver intent of negotiating a curve was used as an example) and their relation to a specific geometric feature (right shoulder paved width was chosen here) for a variety of conditions (divided and undivided urban and rural highways with various lane configurations). The same principle can be applied to quantify crash risks posed by light posts or bridges, various types of barriers (concrete, guardrail, cable guard, curb and gutter, etc.), or median types (paved, rumble strip, barrier curb, etc.) Any number of crash rate statistics can be calculated (mean, median, standard deviation, etc.); the 99th percentile crash rates were used for this task in order to facilitate the second task, described below.

The second task demonstrates how the 99th percentile crash rate for a targeted subset of crashes developed under the first task can be used to identify the total length of highways that exceed this criterion and are therefore in need of safety upgrades. Furthermore, the termini of specific highway segments exceeding the 99th percentile crash rate are produced in tabular form (PRÈCIS capability to display results on color-coded geographic information systems [GIS] maps was presented in a 2006 Mid-Continent Symposium paper).

If the criterion for safety upgrades is set at a lower percentile, a larger number of highway miles will meet this new criterion. Safety program administrators can adjust the statewide length of high-risk highway segments targeted for safety upgrades to match available funding by choosing an appropriate crash rate percentile.

The third task demonstrates how benefit-cost analysis data can be developed for a targeted safety upgrade measure, for example provision of a shoulder (or shoulder widening). Paved and unpaved right shoulder widths are analyzed in relation to statewide run-off-road (ROR) crash rates on two-lane highways. The point of diminishing (safety) returns for additional paved and/or unpaved shoulder width can be determined.

RESEARCH METHODOLOGY

The PRÈCIS database, consisting of records containing roadway feature, crash, and traffic volume information for each 1/100th of a mile along the entire Wisconsin STH system was used to develop crash rates. Separate crash rates were produced for all and ROR crashes for urban and rural, divided and undivided highways, segregated into cells representing combinations of number of lanes and right shoulder width. For the first task, mean and 99th percentile crash rates were calculated for each above-described cell; this information was used in the second task to identify the total length of highway segments meeting or exceeding a threshold value; specific highway segments were identified for targeted safety improvements. The third task required the development of regression models, using average crash rates as the dependent variables and paved, unpaved, and total right shoulder width as the dependent variable. A variety of regression models (linear, quadratic, polynomial) were fit to the data; the best-fitting models are presented herein.

KEY FINDINGS

Lengths of STH segments with right shoulder paved width within a given range are summarized in Table 1. The table provides a breakdown of information by highway classification (divided or undivided highway) and population density (urban or rural) for the most common numbers of lanes in each category. It should be noted that this information is provided as a demonstration of PRÈCIS capabilities. It is not intended to provide definitive crash rates for each presented category, given that about a third of the categories do not have adequate mileage.

Table 1. Right shoulder length (miles) for STH

		Rural population density				**Urban population density**			
		Divided hwy		**Undivided hwy**		**Divided hwy**		**Undivided hwy**	
		# of Lanes		**# of Lanes**		**# of Lanes**		**# of Lanes**	
		4	**6**	**2**	**4**	**4**	**6**	**2**	**4**
R Shoulder	1–3 ft.	118.42	4.45	6078.20	24.98	115.86	9.83	565.69	81.09
	4–8 ft.	790.53	12.20	1520.22	21.78	286.68	54.40	279.15	59.17
	9–12 ft.	650.49	87.81	223.88	1.62	183.36	101.93	100.87	14.38

Note: Cells shaded in light grey have very limited mileage—their information may not reliable

Task 1. Statewide Crash Rate Statistics for Targeted Crash Subsets

The extensive crash rate database created by PRÈCIS was used to provide crash rate statistics corresponding to each category (cell) of Table 1. Table 2 provides average crash rates for Table 1 cells. In general, as expected, for identical numbers of lanes, crash rates for undivided highways are higher than for divided highways, and urban crash rates are higher than rural crash rates.

Table 2. Average crash rates, crashes per 100 MVMT, all crashes

		Rural population density				Urban population density			
		Divided hwy		Undivided hwy		Divided hwy		Undivided hwy	
		# of Lanes		# of Lanes		# of Lanes		# of Lanes	
		4	**6**	**4**	**6**	**4**	**6**	2	4
R Shoulder	1–3 ft.	99.30	195.02	143.19	121.50	125.27	191.83	137.38	155.05
	4–8 ft.	72.88	59.46	151.16	129.50	93.76	134.32	138.39	190.33
	9–12 ft.	56.81	78.98	193.52	126.55	79.56	94.34	136.31	176.38

A variety of statistics (median, standard deviation, different percentiles etc.) can also be produced for Table 1 cells. The 99th percentile crash rates in crashes per 100 million vehicle miles of travel (MVMT) for all types of crashes are displayed in sample Tables 3 through 6, in order to demonstrate possible cells for which statewide crash rate statistics can be calculated using PRÈCIS. The 99th percentile was chosen because it is used to demonstrate Task 2, presented later. It should be noted in Table 3 that the extreme values for the 99th percentile for six-lane divided and two-lane undivided rural highways (1533.73 and 1291.71 crashes per 100 MVMT, respectively) are due to large crash rate standard deviations within these two categories, multiples of which are added to the average crash rates in order to find the given percentiles.

Table 3. 99th percentile crash rates, crashes per 100 MVMT, all crashes

		Rural population density				Urban population density			
		Divided hwy		Undivided hwy		Divided hwy		Undivided hwy	
		# of Lanes		# of Lanes		# of Lanes		# of Lanes	
		4	**6**	**4**	**6**	**4**	**6**	**4**	**6**
R Shoulder	1–3 ft.	329.04	450.91	719.67	272.09	513.57	508.52	558.13	471.03
	4–8 ft.	245.65	267.50	701.77	258.00	318.05	449.17	659.24	474.92
	9–12 ft.	226.29	1533.73	1291.71	215.49	301.25	286.67	551.17	487.33

Table 4 displays run-off-road 99th percentile crash rates for Table 1 cells.

Table 4. 99th percentile crash rates, crashes per 100 MVMT, ROR crashes

		Rural population density				Urban population density			
		Divided hwy		Undivided hwy		Divided hwy		Undivided hwy	
		# of Lanes		# of Lanes		# of Lanes		# of Lanes	
		4	**6**	**2**	**4**	**4**	**6**	**2**	**4**
R Shoulder	1–3 ft.	146.46	65.88	388.52	152.53	119.04	72.84	247.70	137.85
	4–8 ft.	96.77	68.91	440.19	124.04	104.15	94.06	239.27	160.84
	9–12 ft.	98.41	842.05	468.18	57.04	93.52	106.05	179.40	142.00

Table 5 displays the 99th percentile crash rates for crashes where the driver was negotiating a curve at the time of the accident.

Table 5. 99th percentile crash rates, crashes per 100 MVMT, driver negotiating curve

		Rural population density				**Urban population density**			
		Divided hwy		**Undivided hwy**		**Divided hwy**		**Undivided hwy**	
		# of Lanes		**# of Lanes**		**# of Lanes**		**# of Lanes**	
		4	**6**	**4**	**6**	**4**	**6**	**4**	**6**
R Shoulder	1–3 ft.	72.83	15.59	252.33	24.04	24.68	15.23	115.37	23.47
	4–8 ft.	34.41	10.97	265.29	16.25	24.09	17.05	91.15	30.34
	9–12 ft.	16.65	90.22	328.80	4.99	23.72	23.72	106.74	24.99

Table 6 displays the 99th percentile crash rates by driver age for four-lane divided highways.

Table 6. 99th percentile crash rates, crashes per 100 million vehicle miles of travel, driver age group

Hwy classification: divided; # of Lanes: 4.00		**Rural population density**				**Urban population density**			
		16–24	**25–34**	**35–44**	**45–64**	**16–24**	**25–34**	**35–44**	**45–64**
R Shoulder	1–3 ft.	65.62	44.16	27.92	30.75	46.69	36.40	30.93	24.94
	4–8 ft.	36.61	29.03	26.88	30.11	47.83	28.90	25.33	20.09
	9–12 ft.	33.65	26.75	20.62	25.58	31.13	23.18	21.51	21.34

Task 2. Crash Rate Percentiles Used to Identify Highway Segments for Safety Upgrades

Once the 99th crash rate percentile values for a particular combination of highway type, right shoulder width range and crash category have been identified, the crash rate database can be queried again in order to identify the total length of highway segments that meet or exceed this crash rate. The query also provides the specific highway and highway segment(s) that meet or exceed this criterion.

Findings in Table 3 will then be used to identify segments meeting or exceeding the calculated 99th percentile crash rate. For example, a data base query indicated that there were a total of 9.38 STH miles exceeding a crash rate of 226.29 crashes per 100 MVMT (identified as the 99th percentile value for four-lane divided highways with shoulders 9–12 ft. in Table 3). A partial listing of identified segments is shown in Table 7 below, organized by right shoulder width, the sum of daily volumes in the analyzed five years, a highway reference point that provides highway, direction and milepost and the crash rate for the segment, based on five years of crashes and traffic information.

Table 7. STH rural divided four-lane highway segments targeted for treatment

R. shoulder (ft.)	5 YR ADT	Reference	Rate
9.00	2070	078N124 115	262.09
		078N124 121	262.09
	31700	002W022D000	273.83
		002W023D040	273.83
	160500	164N133K000	231.91
		164N134D020	228.33
10.00	2070	078N124 146	262.09
		078N124 150	284.63

Table 7 indicates, for example, that crash rates in the northbound direction of State Trunk Highway 78 between reference point 078N124 115 and reference point 078N124 121 (these codes provide the exact termini of a specific segment, approximately 320 ft. long) exceeded the 99th percentile value for similar divided four-lane rural highways with 9–12 ft. right shoulders since the segment had a crash rate of 262.09 crashes per 100 MVMT. The sum of ADTs during the five-year analysis period was 2070 vpd; the segment had 9 ft. shoulders.

If sufficient funds are available to treat more highway miles, the criterion for safety upgrades can be set at a lower percentile, so that a larger number of highway miles will meet this new criterion. For example, the 95th percentile crash rate value could have been used in the place of the 99th percentile to increase the number of miles identified for treatment. Safety program administrators can adjust the statewide length of high-risk highway segments targeted for safety upgrades to match available funding by choosing an appropriate crash rate percentile.

Task3. Identifying the Point of Diminishing Returns for a Targeted Safety Upgrade Measure

This section will demonstrate how PRÈCIS can be used to support a decision for a statewide policy on appropriate paved and unpaved right shoulder widths if it is desired to lower ROR crash rates on two-lane, two-way rural highways. Average crash rates are used throughout the section.

Between 1998 and 2002, 61,787 ROR crashes occurred on the Wisconsin STH system. The average ROR crash rate on rural roadways was approximately 30% higher than on urban roadways.

Table 8. ROR avg. crash rates, crashes per 100 MVMT, population density

	Rate	Length (miles)
Rural	51.0	10222
Urban	39.1	2195

ROR crash rates on rural highways were much higher for undivided roadways (Table 9), mainly due to two-lane pavements that comprised approximately 80% of the rural mileage (Table 10).

Table 9. Rural ROR avg. crash rates, crashes per 100 MVMT, highway classification

	Rate	Length (miles)
Divided	26.83	1997.85
Undivided	56.82	8215.93

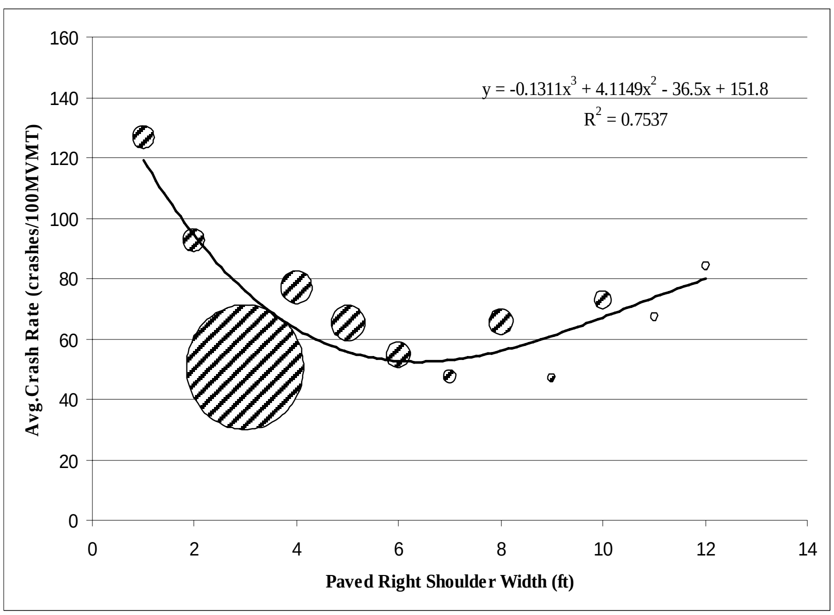
Table 10. Rural ROR avg. crash rates, crashes per 100 MVMT, highway classification

Divided/Undivided	No. of Lanes	Crash rate	Length (miles)
D	4	25.0	1784.98
D	6	31.8	135.47
U	1	21.5	75.79
U	2	57.2	8046.62
U	4	36.8	62.60

Average ROR crash rates for two-lane rural highways were used as the dependent variable and paved right shoulder width as the independent variable in regression models (linear, quadratic, other polynomial) calibrated on the PRÈCIS database. Figure 1 is a graphical representation of the best fit, a third-degree polynomial model, with an R^2 value of 0.753. The model is based on a total of 8,038 miles of rural two-lane undivided highways. The diameters of the cross-hatched circles indicate the mileage for each given shoulder width value (exact mileage shown on the table accompanying Figure 1).

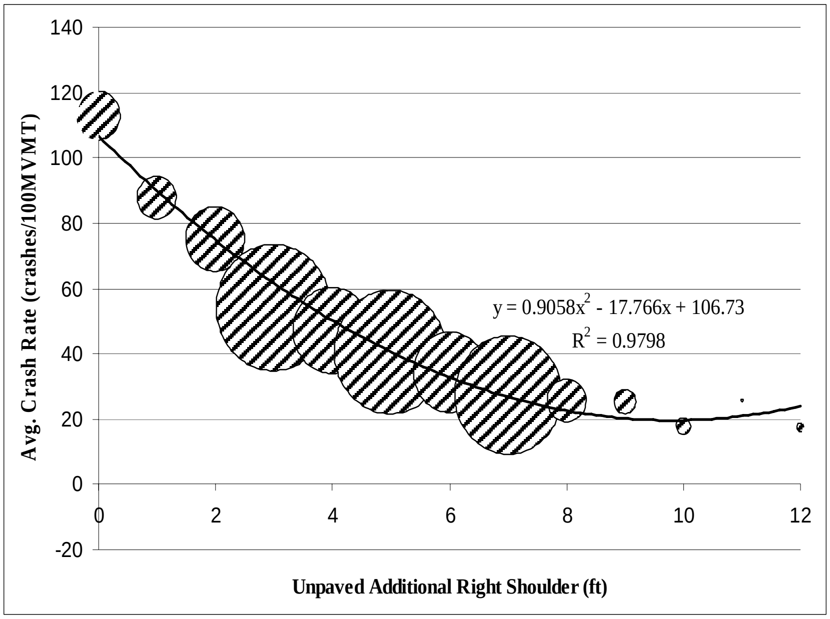
ROR crash rates drop rapidly as right paved shoulder width increases from zero to three ft.; this effect seems to taper off as the width of the paved shoulder increases to values greater than three ft.

The majority of rural two-lane mileage (5,792 miles) in the Wisconsin STH system has three ft. paved shoulders. These sections had a ROR crash rate of 50.3 crashes/100MVMT. A quadratic regression model (ROR crash rate independent variable; additional unpaved shoulder width dependent variable) was calibrated on this population of two-lane rural highways with paved three ft. shoulders. A graphical representation of the model is shown in Figure 2.



Paved R. shoulder width (ft.)	Crash rate	Length (miles)
1	126.8	224.5
2	92.5	238.6
3	50.3	5793.0
4	77.0	418.1
5	65.1	505.8
6	54.8	285.5
7	47.8	76.2
8	65.8	268.8
9	47.1	28.4
10	72.8	136.3
11	67.5	31.1
12	84.4	31.8

Figure 1. Effect of paved right shoulder width on average ROR crash rate



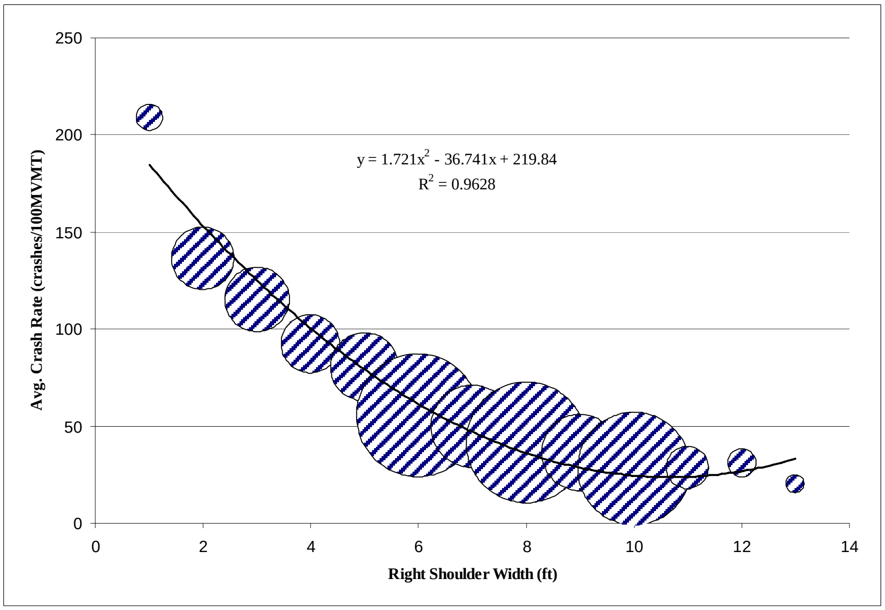
Unpaved R. shoulder width (ft.)	Crash rate	Length (miles)
0	112.9	184.8
1	87.8	149.7
2	74.9	339.7
3	54.2	1284.9
4	47	589.2
5	40.3	1207.1
6	34.3	507.0
7	27.5	1092.1
8	25.6	152.5
9	25.1	53.7

Figure 2. Effect of additional unpaved right shoulder width on average ROR crash rate

Figure 2 shows ROR crash rate reductions for two-lane rural highways with three ft. shoulders when additional unpaved shoulder width is provided. Crash rate reductions taper off for additional unpaved shoulder widths in excess of seven ft.; however, very limited mileage is available with wider unpaved shoulders, thus this width limit is a tentative finding.

When a quadratic regression model is calibrated using total right shoulder width (paved plus unpaved width) as the independent variable, it is shown that a crash rates become lower as the width increases from one to ten ft. (Figure 3); additional shoulder width does not reduce crash rates any farther. Here, again, there is limited mileage with a total right shoulder width greater than ten ft. for statistically sound findings.

PRÈCIS helped determine that, when it comes to ROR crashes on rural two-lane highways, crash rates decrease in direct relation to the available right shoulder width, up to a width of 10 ft. An optimal paved shoulder width is three ft.; additional safety benefits correlate well with the width of any available additional unpaved shoulder.



Unpaved R. shoulder width (ft.)	Crash rate	Length (miles)
1	208.8	78.8
2	136.5	426.5
3	115.2	437.6
4	92.5	380.0
5	80.3	494.8
6	55.7	1598.2
7	49.8	740.2
8	41.4	1542.5
9	36.2	625.0
10	27.9	1358.8
11	28.3	197.7
12	31.0	94.3

Figure 3. Effect of total paved right shoulder width on average ROR crash rate

CONCLUSIONS

PRÈCIS was shown to be suitable to provide statewide crash rate statistics for a variety of targeted crash types (e.g., driver negotiating a curve) or crashes associated with specific driver characteristics (e.g., driver age) on targeted highway subsets (e.g., rural four-lane undivided highways) that exhibit specific characteristics (e.g., a given shoulder width).

Furthermore, PRÈCIS was used to identify specific highway segments that exceed a certain crash rate threshold (based on statewide information) for a targeted type of safety problem. Adjusting the crash rate threshold upward (or downward) allows safety administrators to target a smaller (or larger) number of highway miles for safety treatment.

The PRÈCIS database provides the opportunity to identify the point of diminishing safety returns for various roadway features. For rural two-lane undivided highways, three ft. paved shoulders and seven ft unpaved shoulders were found to be associated with the lowest ROR crash rates, based on available statewide information.

PRÈCIS is a flexible method to analyze statewide safety information in a consistent and labor-efficient manner.

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The Effects of Headcut and Knickpoint Propagation on Bridges in Iowa

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ABSTRACT

Headcuts (known also as primary knickpoints) and knickpoints (known also as secondary knickpoints) have been found to contribute to the accelerated riverbed degradation problem in the midwestern United States. Step-changes that occur at the head of channel networks are referred to as headcuts, and those that occur within the confines of channel banks are referred to as knickpoints. The formation of headcuts and knickpoints and their upstream migration have been linked to the over-steepening of stream reaches when the flow plunges to the bed and creates a plunge pool. Secondary flow currents and seepage are believed to be some other parameters contributing to the formation and evolution of headcuts and knickpoints. Ongoing research suggests that headcuts and knickpoints, where they form and migrate, may account for 60% (or more) of the bed erosion in the streams. Based on preliminary observations, there is a strong indication that headcuts and knickpoints can also have a greater influence on flow thalweg alignment (line of deepest flow) for small rivers. A shift in thalweg toward a riverbank or embankment is usually a prime factor contributing to riverbank erosion and scour.

Key words: bridges—erosion—headcuts—knickpoints

Field Investigation of Hydraulic Structures Facilitating Fish Abundance and Passage through Bridges in Western Iowa Streams

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ABSTRACT

The overarching goal of the proposed research was to evaluate the hydraulic performance of 22 fish passage structures located in close proximity to bridges in western Iowa and within the Hungry Canyon Alliance territory. Such structures include riprap weirs, fish ladders, and grouted ripraps. The hydraulic performance of the aforementioned structures was evaluated via detailed field tests for a range of flow conditions relevant to fish migration through bridge waterways in different streams in western Iowa. The best performance, without considering the drainage areas, was exhibited by the low-gradient grouted or riprap weirs or by the fish ladder with baffles. The medium-gradient weirs also performed satisfactorily. Considering also the drainage areas, it is recommended that when the drainage areas are less than 20 square miles, the best structure is the low gradient; when the drainage areas are between 20 and 100 square miles, the best structure is either the low or medium gradient; and when the drainage areas are larger than 100 square miles, the best ones are the medium gradient.

Key words: fish ladders—grouted ripraps—hydraulic performance—riprap weirs

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Seasonal Variation in the Subgrade and Base Layers of U.S. Highway 20, Iowa

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ABSTRACT

Seasonal variations in ground temperature and moisture content influence the load carrying capacity of the pavement subgrade layers. To improve pavement performance, pavement design guidelines require knowledge of environmental factors and subgrade stiffness relationships. As part of this study, field instrumentation were installed along U.S. Highway 20 near Fort Dodge, Iowa, to monitor the seasonal variations in temperature, frost depth, groundwater levels, and moisture regime in the pavement subgrade after construction. Dynamic cone penetrometer, nuclear gauge, and Clegg hammer tests were performed at 64 test points in a 6 ft. by 6 ft. grid pattern to characterize the subgrade stiffness properties (i.e., resilient modulus) during construction operations. The purposes of this paper are to present the field instrumentation results and the observed changes in subgrade stiffness properties due to freeze-thaw effects based on in situ test measurements.

Key words: resilient modulus—seasonal variation—spatial variability

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INTRODUCTION

Seasonal variation in ground temperature and moisture content of the pavement subgrade and base layers influence their load carrying capacity. Loss of support conditions (i.e., a reduction in stiffness) in these layers occurs during thawing periods and/or saturated conditions and is one of the primary contributors to distresses in pavements. A better understanding of seasonal variation in subgrade properties would benefit pavement design. As a part of this study, field instrumentation was installed to monitor the seasonal variations in temperature, moisture content, frost depth, and groundwater levels.

Field tests, including dynamic cone penetrometer (DCP), nuclear gauge, and Clegg hammer were conducted at 64 test locations prior to placement of base aggregate materials to analyze the spatial variability of the subgrade soils. Approximately two years after construction, the subgrade soil was sampled for laboratory resilient modulus testing. In addition, DCP tests were also performed during the 2006/2007 freeze-thaw cycle to observe the changes in subgrade stiffness index.

BACKGROUND

Resilient modulus (M_r) was first proposed by Seed et al. (1962) and is the ratio between repeated deviator stress (σ_d) and recoverable strain (ε_r) in the direction of the major principle stress (Li and Selig 1994). Resilient modulus is dependent on several factors: soil type, water content, and compaction level. In their study, Li and Selig developed relationships between the resilient modulus at varying moisture content to the resilient modulus at optimum moisture content, for constant compaction energy, and dry unit weight. The best fit polynomial equations to calculate these ratios using moisture content and the optimum moisture content were proposed by the authors, as following:

$$R_{m1} = 0.98 - 0.28(w - w_{opt}) + 0.029(w - w_{opt})^2 \quad (1)$$

$$R_{m2} = 0.96 - 0.18(w - w_{opt}) + 0.0067(w - w_{opt})^2 \quad (2)$$

where R_{m1} is the ratio for constant dry unit weight and R_{m2} is the ratio for constant compaction energy. According to these relationships, a small change of moisture content can result in significant changes in the resilient modulus. Given a constant dry density, M_r can be 3 times higher than $M_{r(\mathrm{opt})}$ if w is 5% lower than w_{opt}. M_r is equal to half of $M_{r(\mathrm{opt})}$ when w is 2% higher than w_{opt}. In case of constant compaction energy, M_r is approximate 2 times higher than $M_{r(\mathrm{opt})}$ if w is 5% lower than w_{opt}, and M_r is equal to half of $M_{r(\mathrm{opt})}$ when w is 3% higher than w_{opt}.

Most fine-grained soils moduli decrease if the water content is increased, resulting in increased deflections in the subgrade (Drumm et al. 1997). A method for correcting the resilient modulus for increased degree of saturation was proposed by the authors. In a study conducted by Rainwater et al. (1999), four sites across the state of Tennessee were instrumented with comprehensive monitoring systems to collect meteorologic, water content, infiltration, and temperature data. Rational methods were applied for data analysis to determine the environmental effects. The results of this study showed that subgrade volumetric water contents varied very little, except for brief periods after heavy rainfall events. A study by Hossain et al. (1996) showed a similar result. These small changes in subgrade moisture content did not show significant changes in subgrade support properties.

INSTRUMENTATION

The project site for this study is located at station 930+00 (mile marker 119.90) along U.S. Highway 20 (US-20) westbound, about 1/2 mile west of Kansas Avenue in Fort Dodge, Iowa. Field instrumentation was installed to monitor seasonal variation in moisture content, temperature, and freeze-thaw cycles. The instrumentation consisted of ten time domain reflectometry (TDR) probes, ten temperature sensors, one resistivity probe, two piezometers, a limited weather station, and a data logger (Figure 1).

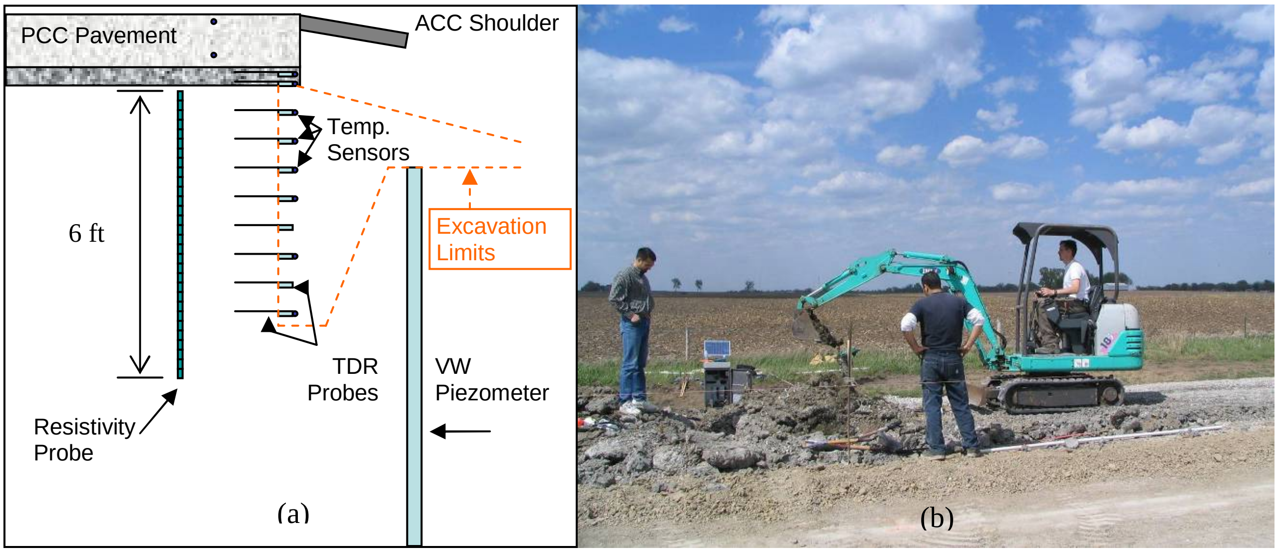

Figure 1. Instrumentation: (a) Cross section view of installation, (b) Installation at site

Table 1. Instrument elevations

Sensor ID	Depth Below Roadway Surface (ft.)
Temperature sensor 1	0.17
Temperature sensor 2	0.75
Temperature sensor 3 & TDR 1	1.10
Temperature sensor 4 & TDR 2	1.28
Temperature sensor 5 & TDR 3	1.75
Temperature sensor 6 & TDR 4	2.21
Temperature sensor 7 & TDR 5	2.82
Temperature sensor 8 & TDR 6	3.31
TDR 7	3.75
Temperature sensor 9 & TDR 8	4.25
TDR 9	4.73
Temperature sensor 10 & TDR 10	5.21
VW Piezometer 1	13.86
VW Piezometer 2	13.82
Resistivity Probe (top)	1.32

SEASONAL VARIBILITY: INSTRUMENTATION MEASUREMENTS

Air Temperature and Groundwater Levels

During the monitoring period, the groundwater reached its highest level of about 9 ft. below the pavement surface during the first week of April 2006. The groundwater reached its lowest level of about 13 ft. below the pavement surface during the last week of November 2005. The general trend shows that during late winter and spring the groundwater level rises and that during the summer and fall months the groundwater level falls. The increase in groundwater levels corresponds with the spring thaw and early rain events, while the decrease in groundwater levels matches the end of the rainy season and the beginning of fall. This trend roughly follows a sinusoidal curve (Figure 2).

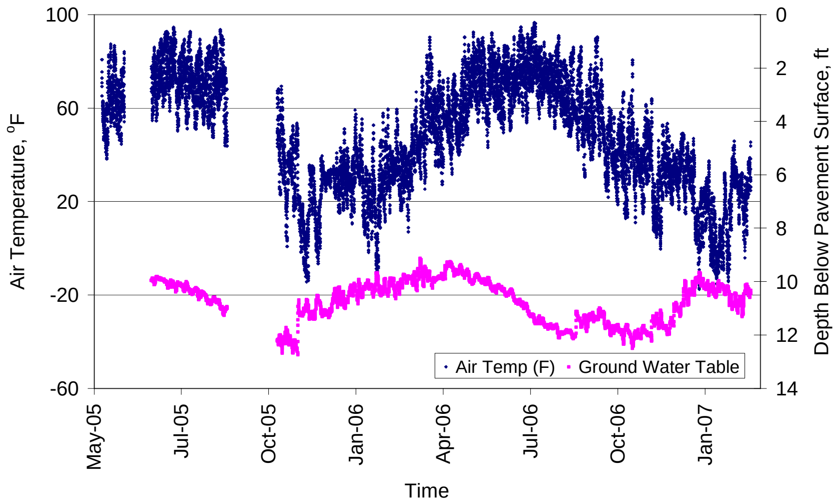

Figure 2. Air temperature and ground water table from May 2005 to March 2007

Base and Subgrade Moisture Contents

Volumetric moisture content in soil was collected by analyzing the waveform data from TDR probes. The TDR is based on measuring the one-way travel time of an electromagnetic wave from a source to an electrical discontinuity (Diefenderfer et al. 2000).

Several empirical relationships between the dielectric constant and volumetric water content have been developed. A commonly used equation (and the one utilized in this study) was presented by Topp et al. (1980) and is given as follows.

$$\theta_v = -5.3*10^{-2} + 2.92*10^{-2} K_a - 5.5*10^{-4} K_a^2 + 4.3*10^{-6} K_a^3 \qquad (3)$$

where θ_v is the volumetric water content and K_a is the dielectric constant.

During freezing periods, the apparent dielectric constant of frozen water results in an artificially low moisture content. Freezing periods in the base layer (Figure 3) occurred from early November 2005 to early February 2006 and from mid-November 2006 to the end of February 2007. The moisture content at the lower level (1.29 ft.) is higher than that at the higher level (1.10 ft.). In fact, higher amount of fine aggregate in the lower part of the base layer can be a factor that maintains higher moisture content in lower part of base layer. Capillarity from subgrade moisture content can be another factor for this phenomenon. Small and sharp peaks of moisture content diagram with time may be due to rain events.

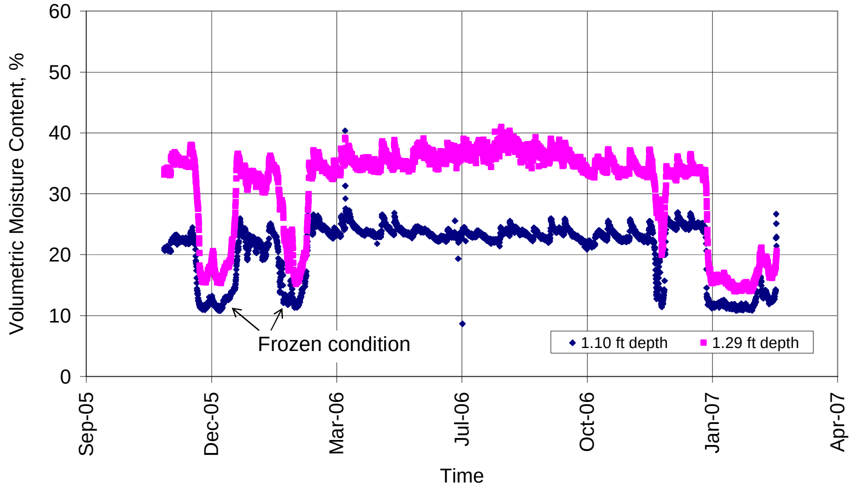

Figure 3. Moisture contents with depths from PCC surface in base layer

Figure 4 shows the fluctuation of moisture content with time at the highest, middle, and lowest levels of the subgrade layer. In general, the volumetric moisture content in the subgrade soils increased during the spring thaw and peaked in the early summer. It appears that relationships exist between precipitation and moisture content of the base (i.e., short-term increases and subgrade layers; a long-term annual trend).

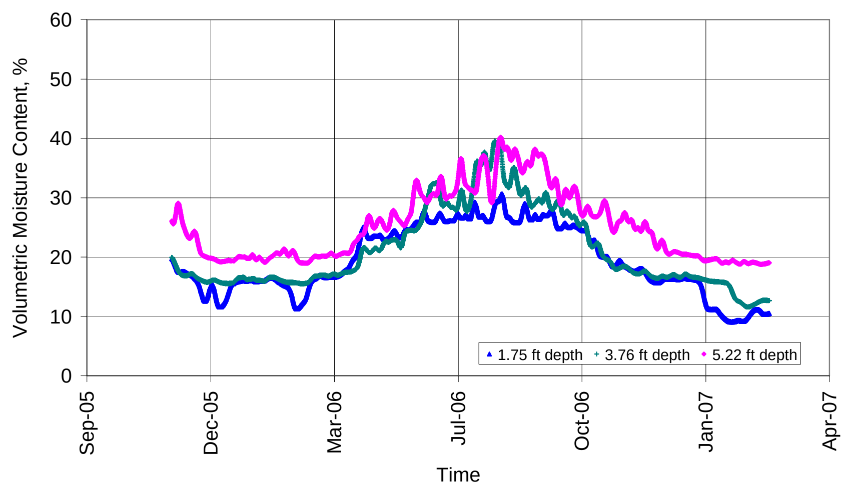

Figure 4. Moisture contents with depths from PCC surface in subgrade layer

The optimum moisture content (w_{opt}) of 12% for the subgrade materials was determined by the Standard Proctor test in the laboratory. The specific gravity of the materials was $G_s = 2.63$. Thus, the optimum volumetric moisture content was 31.6%. Figure 4 shows that the moisture content of subgrade layer at the depth of 1.75 ft. below PCC surface was normally lower than w_{opt} except at peak in rainy seasons. However, the moisture contents at lower levels of the subgrade layer were higher than w_{opt} from late in April to early in October. The maximum volumetric moisture content of these levels was approximate

8.4% (or 3.1% in gravimetric moisture content) higher than w_{opt}. The increase of moisture content results in significant loss of the resilient modulus, given that the dry density is constant and the subgrade layer is compacted at w_{opt}. Thus, rainy season strongly increases the moisture content in the subgrade layer, especially at lower levels and results in a sharp decrease of stiffness.

Frost Penetration

Prior to instrumentation, it was expected that frost would develop from the top down and that thawing would occur from the top down. The resistivity gage readings show that thawing occurs from the top down and bottom up (Figure 5). The gage readings support observations of other researchers that as the ice lens thaws, unfrozen water is trapped in the base layer due to ice in the subgrade prohibiting infiltration.

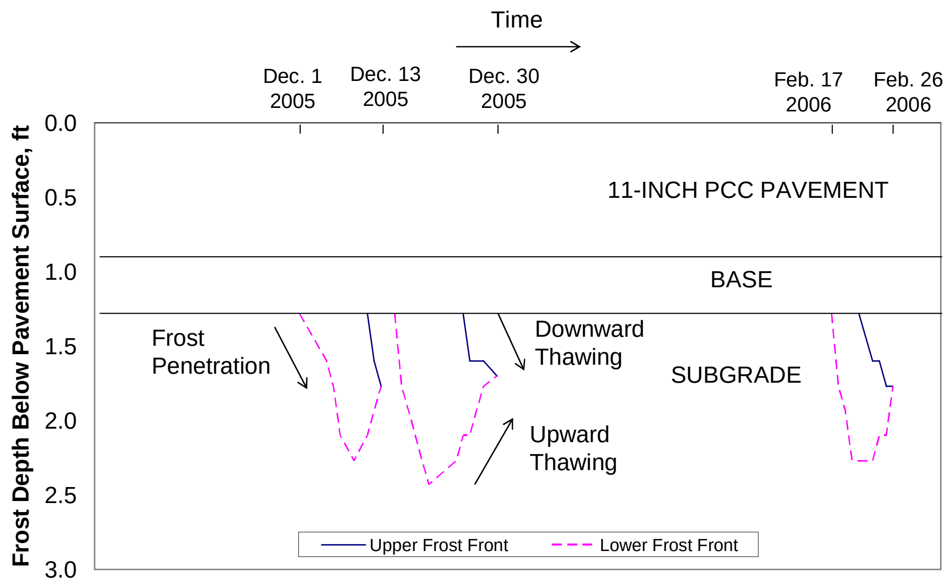

Figure 5. Frost penetration below the pavement surface

The top of the resistivity probe is about 1.3 ft. below the pavement surface. The resistivity probe was installed below the surface of the subgrade to limit damage during paving operations. Because of this, no frost data exists for the base layer material. To describe Figure 5, moving from the left to the right, the penetration of the frost layer is first detected by the resistivity probe on Dec. 1, 2005, reaching a maximum penetration on Dec. 9 and completely thawing on Dec. 13, 2005. A couple days later, another frost lens forms and completely thaws by Dec. 30, 2005. Of note is that thawing of the ice lens occurs from the top down and bottom up.

One explanation for thawing of the ice lens from the top and bottom is that solar radiation warms the shoulder and ditch materials. With the pavement surface the highest point in its cross section, it is expected that thawing occurs from the pavement surface downwards and from the shoulders inwards. The temperature at deep levels in the subgrade soils is normally about 50^{o}F to 60^{o}F. Thus, the subgrade soils are warming from the bottom up. Additional instrumentation of the pavement subgrade would be required to determine the heat flow along a cross section during thawing periods.

Pavement, Base, and Subgrade Temperature Profiles

The temperature fluctuation within the subgrade soils appears to follow a sinusoidal trend. This trend allows estimating the temperature in the periods of time that the data is not available. The temperature in the subgrade layer decreases with depth when it is above the freezing point but increases with depth when it is below the freezing point (Figure 6).

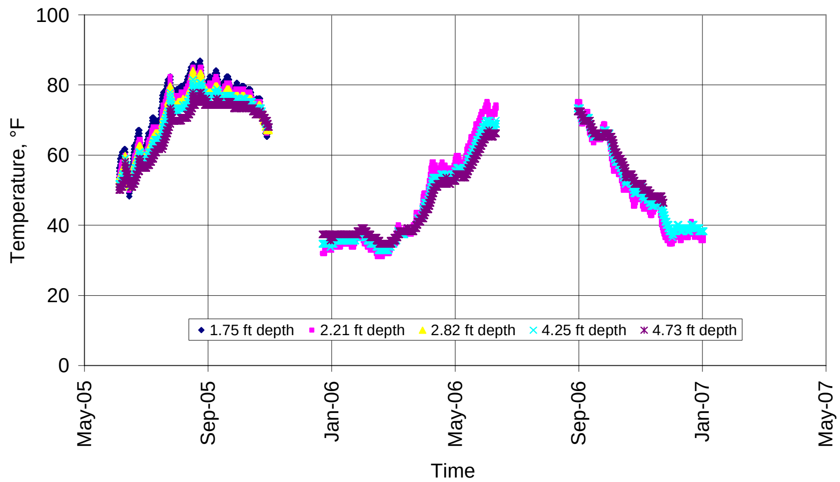

Figure 6. Seasonal trends of the subgrade layer

Figures 7 and 8 show that the PCC pavement experiences greater temperature extremes and that it changes at a higher rate than the base and subgrade layers. Due to the high failure rate of the temperature sensors, a representative graph of the yearly low temperatures is not shown. Several of the sensors ceased operation prior to the first winter. The maximum temperature gradient of the subgrade soil below a depth of about 1.5 ft. was about 1 ^{o}F, which corresponds with the sensitivity of the temperature sensors. Maximum temperature gradients in 1 hour in PCC layer and at the midpoint of subgrade layers were 18^{o}F and 5.4^{o}F, respectively.

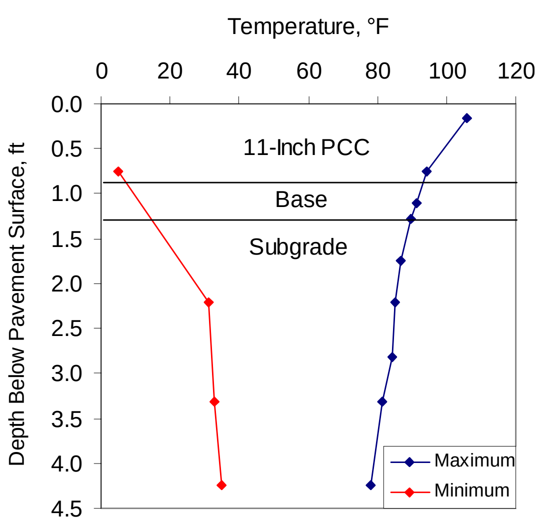

Figure 7. Yearly highest and lowest temperature profile

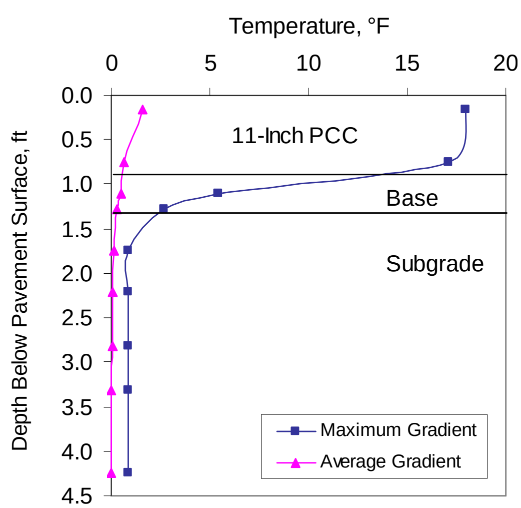

Figure 8. Temperature gradient

SPATIAL VARIABILITY

Field Measurements

One week prior to placement of the base layer, the study section was tested to determine the spatial variability of the subgrade soils. The Clegg hammer, DCP, and nuclear moisture density gage were used to quantify this variability. A test grid was developed which extended beyond the planned sensor installation location by about 50 ft. to the east and west. The test locations were spaced 6 ft. apart in the east-west and north-south directions. The test grid consisted of 4 rows spanning the 2 lanes of westbound traffic, and 16 columns for a total of 64 test locations (Figure 9).

Figure 9. US-20 test site: (a) test grid, (b) nuclear moisture density gage testing

Geospatial Data Analysis (Kriging)

The results of field tests were analyzed using a statistical technique known as Kriging. Kriging is a linear least squares estimation algorithm. This technique is used to interpolate some variable over an area where known values are recorded. In this study, the field test results along the test grid on the subgrade layer were used to develop a topographic style graph (Figure 10).

The exponential variogram model was used to analyze the data sets. The correlation distance for field testing ranged from 10 to 29.5 ft. The Kriging plot of Clegg hammer data shows a definite east-west correlation. This follows the construction sequence typically used for highway fill. In general, a truck unloads fill materials which are then spread in the direction of the road alignment and then compacted in the same direction. Such a system should result in strong correlation in a direction parallel to the roadway alignment. The nuclear density data less clearly shows a similar trend. It was expected that the moisture content would also show strong correlation along the alignment of the roadway, but all three variogram models (i.e, spherical, Gaussian, exponential) resulted in a correlation distance less than the grid spacing.

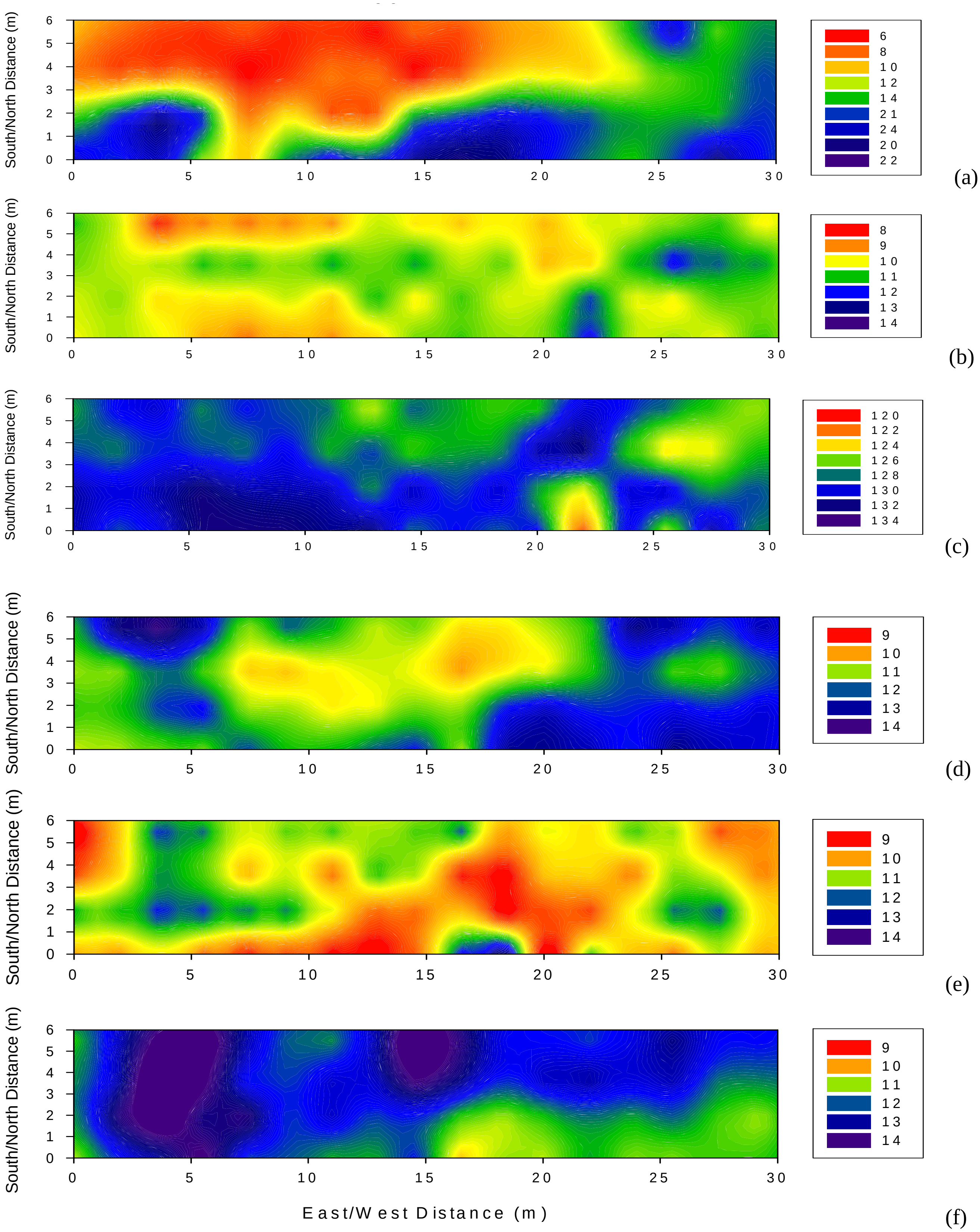

Figure 10. Kriging output of subgrade layers: (a) Clegg hammer, (b) moisture content, (c) dry unit weight, (d) DCP CBR values (0–1ft.), (e) DCP CBR values (1–2ft.), and (f) DCP CBR values (2–3ft.)

EFFECTS OF SEASONAL VARIATION ON SOIL STIFFNESS

In practice, resilient modulus is often estimated from standard CBR, R-value, and soil index test results. The correlation given by AASHTO's *Guide for the Design of Pavement Structures* (1993) is as follows:

$$M_r\,(psi) = 2555 \times (CBR)^{0.64} \tag{4}$$

where the CBR value is calculated from DCP index using the correlation proposed by the ASTM Standard D 6951-03 given as follows:

$$CBR = \frac{292}{DCP^{1.12}} \tag{5}$$

The resilient moduli were calculated from weighted averages of CBR values on each one ft. thick layers (Table 2). The CBR values were obtained from DCP tests, which were conducted prior to the placement of PCC pavement (May 2005) and two years after construction operation (April 2007). Moisture contents of the subgrade layer by the time of field testing in May 2005 ranged from 8.6 to 10.2%. Test specimens sampled by Shelby tubes in April 2007 were used to determine moisture content and resilient modulus in the laboratory. The moisture contents ranged from 10.6 to 11.7%, which were higher than those in May 2005. The average resilient moduli obtained from laboratory tests ranged from 4,836 to 10,332 psi. The maximum and minimum resilient moduli obtained from these tests series were 17,299 psi and 2,780 psi, respectively.

Table 2. Comparison of resilient moduli

Location	Depth Below Subgrade	Resilient Modulus, M_r (psi) May 2005	Resilient Modulus, M_r (psi) April 2007	Note
I4	0–1 ft.	12,553	14,580	DCP tests were at I4 grid point
	1–2 ft.	12,332	15,722	
Sta. 930+21 ft. on row 4	0–1 ft.	13,450	14,147	M_r for May 2005 is the average M_r
	1–2 ft.	12,060	14,825	values at grid points L4 and M4
Sta. 920+84 ft. on row 4	0–1 ft.	13,451	13,192	M_r for May 2005 is the average M_r
	1–2 ft.	12,466	14,334	values at grid points F4 and G4

A series of DCP tests were conducted during construction operation, a freeze-thaw cycle, and approximately 2 years after construction. DCP indices were directly used to compare the stiffness. Figure 11 shows the soil stiffness during various periods. Soil becomes extremely stiff when frozen. The average DCP index during this period was approximately 2 mm/blow. In contrast, soil is weakest during the thawing period when the penetration index was up to about 40 mm/blow. The resistance of soils located at deeper locations also showed variations between frozen and thawed states, but varied from 5 mm/blow to about 20 mm/blow, respectively. The variation of the DCP index with depth during freezing and thawing periods also indicated that thawing process was taken place downward from the top and upward from the bottom of the ice lenses. The results of DCP at the grid point I4 showed an improvement of soil stiffness after a period of time since the construction.

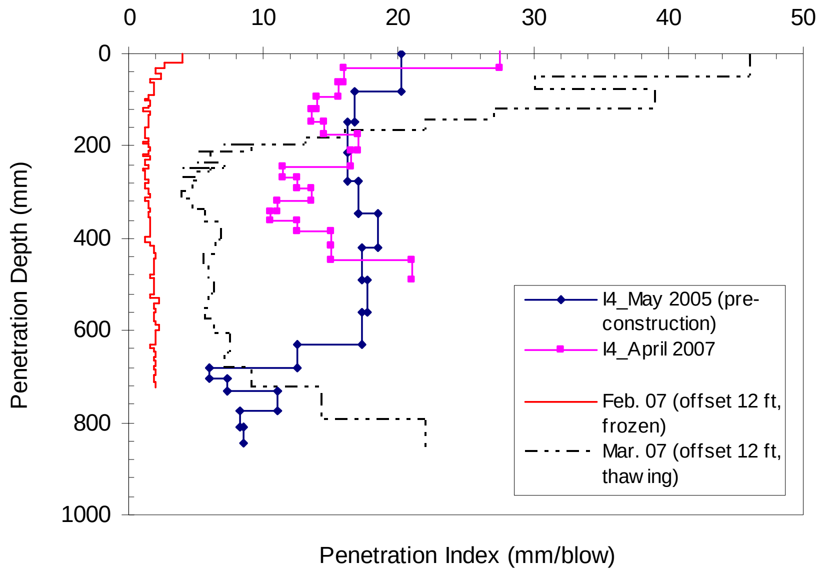

Figure 11. DCP tests during construction, freeze-thaw period, and two years after construction

CONCLUSION

The paper has documented the seasonal variations in temperature, moisture content, frost depth, and ground water levels using field instrumentation. The moisture content in the base layer remains relatively constant throughout the year. In contrast, the moisture content in the subgrade layer changes with the seasons. In the base layer, the moisture content in the lower level is higher than that of higher level. Small and sharp peaks of moisture content are associated with rain events. Moisture content in the subgrade soils increased during the spring thaw and peaked in the early summer. Moisture contents of the subgrade layer increased with depth and are affected by seasonal variations. At a depth of four ft. below the PCC surface, moisture content can increase more than 3% compared to the w_{opt}, resulting in sharp decrease of resilient modulus. The results from this study suggest that the moisture content in the subgrade layer is strongly dependent on the freeze-thaw processes and rain seasons.

In the subgrade layer, freezing penetrates downward, but thawing occurs in both downward and upward directions. During the thawing process, ice lenses under the pavement thaw from the top down and bottom up. The temperature in the subgrade layer decreases with depth when it is above the freezing point, but increases with depth when it is below the freezing point. The PCC pavement experiences greater temperature extremes and it changes at a higher rate than the base and subgrade layers. The temperature gradient within one hour at PCC surface was up to 18^{o}F.

The Kriging plots show the spatial variations in moisture content, density, Clegg Impact value, and DCP-CBR values. The Clegg hammer data reflects the methodology used in roadway construction. Fill material is delivered and is usually spread with a dozer or grader. This new lift is then compacted in the direction of the travel lanes. The Clegg Hammer plot shows that the south lane has a higher Impact value; possibly resulting from additional roller passes or from a variation in material being placed.

Soil stiffness is dependent on the moisture content. A small increase of moisture content can result in significant decrease of soil resilient modulus. Soil becomes extremely stiff when frozen and is weakest in a fully saturated condition. To improve pavement design and performance prediction, understanding the relationships between subgrade stiffness and moisture content are needed. The paper attempts to use in-ground instrumentation to provide these relationships.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge and thank the Iowa Department of Transportation for funding the project and traffic control for field tests. Special thanks to Heath H. Gieselman and other members of the Iowa State University field testing team that helped for the field tests and to Pavana Vennapusa for helping in the resilient modulus triaxial tests.

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FRP Deck Bridge: Design, Construction, and Evaluation as a Temporary Bypass Structure

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ABSTRACT

Bridge replacement projects are becoming common due to the deteriorating bridge infrastructure, both in Iowa and nationwide. To minimize the inconvenience to roadway users, temporary roadway bypasses can be constructed to route traffic around the construction site. The Iowa Department of Transportation (Iowa DOT) maintains a stockpile of steel temporary bridge spans that can be used as either single-span bridges or linked together to form multi-span temporary bridge crossings. Due to the deteriorating condition of the steel temporary bridges, the Iowa DOT initiated a research project to determine if a fiber-reinforced polymer (FRP) deck bridge could be a suitable replacement for the existing steel temporary bridge spans. To aid in answering this question, the Iowa DOT applied for and was awarded TEA-21 Innovative Bridge Research and Construction (IBRC) funding. The FRP deck bridge for these temporary bypass applications is comprised of two 39 ft. 10 in. by 13 ft. 6 1/2 in. pieces that are spliced together with steel splice plates along the centerline of the roadway to form a bridge that spans 39 ft. 0 in. and is 24 ft. 0 in. from face of barrier rail to face of barrier rail for two 12 ft. 0 in. traffic lanes. The bridge was fabricated on temporary supports and preliminarily load tested, as shown in Figure 1. Preliminary load testing of the FRP deck bridge was satisfactory and indicated that the design control is primarily with regard to deflection limitations rather than strength limitations.

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Figure 1. Bridge on temporary supports subjected to test load

In May 2007, the FRP bridge was delivered to its inaugural project site near Ft. Atkinson, Iowa, and installed as a temporary bridge to provide an alternate route for traffic while the mainline bridge was being replaced. The design of the bridge is such that the entire structure may be loaded onto one tractor trailer and hauled to the site and handled with a relatively small crane, as shown in Figure 2. Once onsite, the two slab sections are set on the abutments and joined with custom-fabricated steel plates and outfitted with steel barrier rails. The completed structure can be installed and ready for traffic in less than two days once the abutments are in place.

Figure 2. Bridge sections on trailer for transport

The bridge has been in service for several months and remains so, with only minor construction and serviceability issues requiring attention. Design details of the abutment seats and backwalls were found to be difficult to fabricate and required modification. In addition, variances in elevation between the approach slabs and the edge of the bridge sections resulted in minor deterioration of the edge of the bridge panels and subsequently modification of this design detail as well. The bridge has since been in use under service-level loads and functioning sufficiently. Prior to removal at the completion of the project, the bridge will again be load tested and evaluated.

Factors that may influence whether the Iowa DOT chooses to replace the stockpile of steel temporary detour bridges with FRP bridges include in-service load test results, the life-cycle cost analysis, durability of the system under service, and availability of FRP bridge fabricators. Life-cycle costs and durability have yet to be evaluated.

Key words: composites—fiber-reinforced polymer—FRP bridge—temporary bridge

Minnesota Trunk Highway 10 Anoka County Business Stakeholder Involvement Process

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ABSTRACT

Trunk Highway 10 (TH10) is a major arterial roadway connecting the Minneapolis/St. Paul and St. Cloud metropolitan areas in Minnesota. TH10 (signed as U.S. Highway 10) runs parallel to Interstate 94 along this entire route, but the two are separated by the Mississippi River. River crossings are limited. The TH10 corridor is developing rapidly as suburban growth radiates rapidly outward from Minneapolis/St. Paul. The highway is a freeway from its junction with Interstate 35W and for several miles to the northwest toward St. Cloud. However, at the city of Anoka it becomes a multilane expressway with at-grade intersections and some nearly direct accesses for commercial businesses. An overall planning study for the corridor completed in January 2002 recommended major capacity increases along the TH10 corridor, particularly in the Anoka area. One possibility for increasing capacity is an upgrade of part of TH10 from an expressway to a freeway with full access control and interchanges rather than at-grade intersections. This sort of conversion typically raises the hackles of commercial businesses and developers alike.

During 2006 and 2007, a team of consultants, local governments, and the Minnesota Department of Transportation collaborated on a detailed planning study and stakeholder involvement process for a one-mile long section of TH10 through the city of Anoka. This planning process included tasks designed to understand the needs of commercial businesses along the study corridor and to directly involve them in the design and evaluation of project alternatives for the corridor. The process involved the development of a corridor economic profile, a business inventory and classification study to determine the businesses most likely to be impacted by access changes, a set of detailed business interviews, a business forum, and

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a business-oriented design charette. The entire process was designed to understand the concerns of businesses, educate them, and also to fully engage them in the planning process. The results of this process led to significant modifications in project alternative concepts and a surprising lack of local business and citizen objections to freeway conversion alternatives. The freeway design alternatives would involve significant changes in roadway access for businesses adjacent to the corridor, including the elimination of all direct access from the roadway mainline.

Key words: business involvement—Minnesota—project stakeholders

Minnesota CSAH 42: A Case Study Illustrating Traffic Signal Removal as an Access Management Strategy

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ABSTRACT

County State Aid Highway (CSAH) 42 is a major east-west arterial serving the southern suburbs of the Twin Cities metropolitan area in Minnesota. This corridor represents an excellent case study in both how to and how not to manage access along a major suburban arterial.

The existing highway corridor configuration was developed over a number of decades and illustrates a number of poor access management practices, including lack of coordination among local government jurisdictions in terms of traffic engineering and land use planning, over-reliance on traffic signalization as a solution to traffic operations and safety issues, and lack of planning for alternative access to land development (e.g., backage or frontage roads).

In contrast, an access management project currently underway along the corridor illustrates a number of proactive access management practices being practiced by the local governments involved in the project, including use of microscopic traffic simulation to analyze alternatives and educate stakeholders, careful attention to commercial business concerns along the corridor, and use of innovative and difficult access management treatments.

Key words: access management strategies—signalization—traffic signal removal

PROBLEM STATEMENT

Managing access along existing urban and suburban arterial corridors that are also commercial corridors is an inherently difficult task. Arterial roadways should primarily function in a manner that channels through traffic with a high level of service and at a relatively high mean travel speed; direct access to adjacent land and development should be kept to a minimum. However, commercial businesses along such corridors depend on these roads for customer access; they prefer a high degree of visibility from the road to the business and the most direct access possible for potential customers. The need to serve through traffic while also meeting the desires of commercial businesses creates a difficult balancing act at best and conflicts at worst.

RESEARCH OBJECTIVES

This paper presents a case study that illustrates both how to and how not to manage access along a major suburban arterial. The case study focuses on County State Aid Highway (CSAH) 42, a major east-west arterial serving the southern suburbs of the Minneapolis/St. Paul, Minnesota metropolitan area. The existing highway corridor configuration, developed over a number of decades, illustrates a number of poor access management practices. In contrast, an access management project currently underway along the corridor illustrates a number of proactive access management practices.

CURRENT ACCESS MANAGEMENT PRACTICES ON CSAH 42

Corridor Location and Function

County State Aid Highway (CSAH) 42 is a major suburban arterial located in the southern potion of the Minneapolis/St. Paul, Minnesota metropolitan area (see Figure 1). As its name implies, the arterial is managed by the counties it traverses; however, it is an important enough route for through traffic that it receives state aid. The roadway is functionally classified as a nonfreeway principal arterial and is on the National Highway System (NHS). Figure 2 provides a view to the west down the corridor as it existed in mid-2002. Note that photo was taken during a weekday off-peak time. Four sets of traffic signals within the sub-corridor are visible in the image.

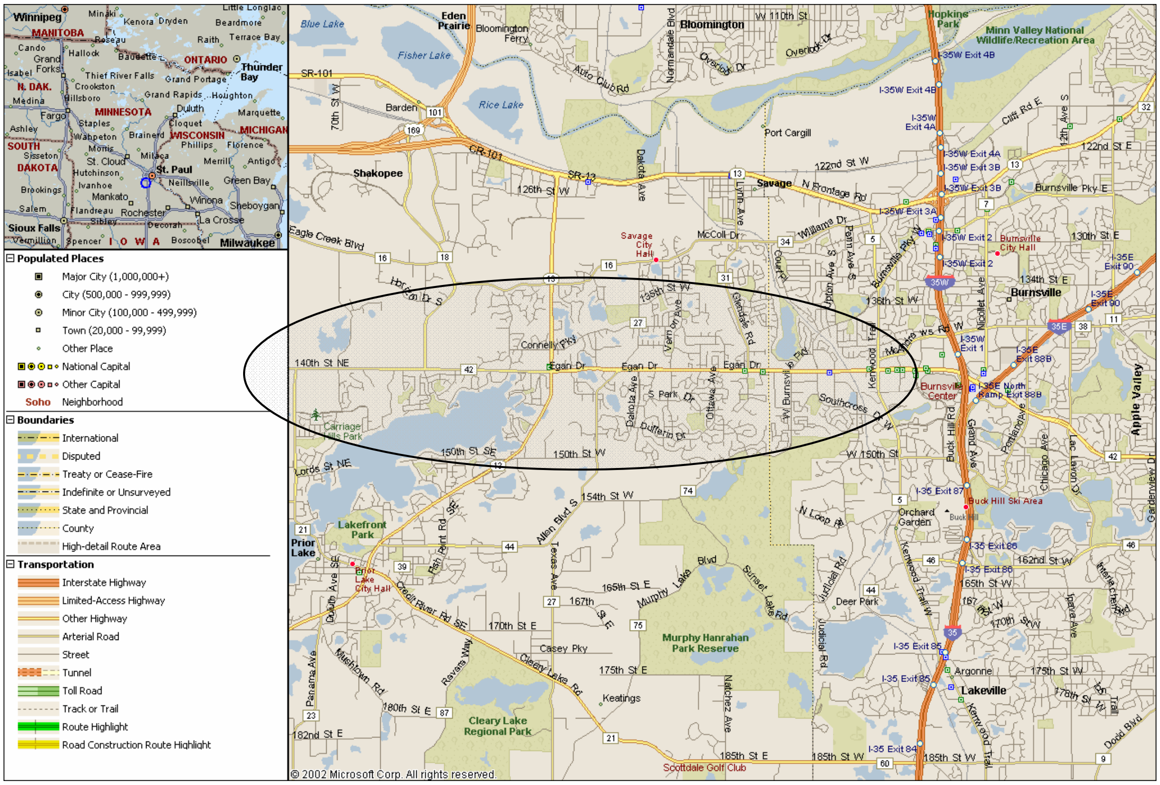

Figure 1. Location of CSAH 42 (from Microsoft Streets and Trips)

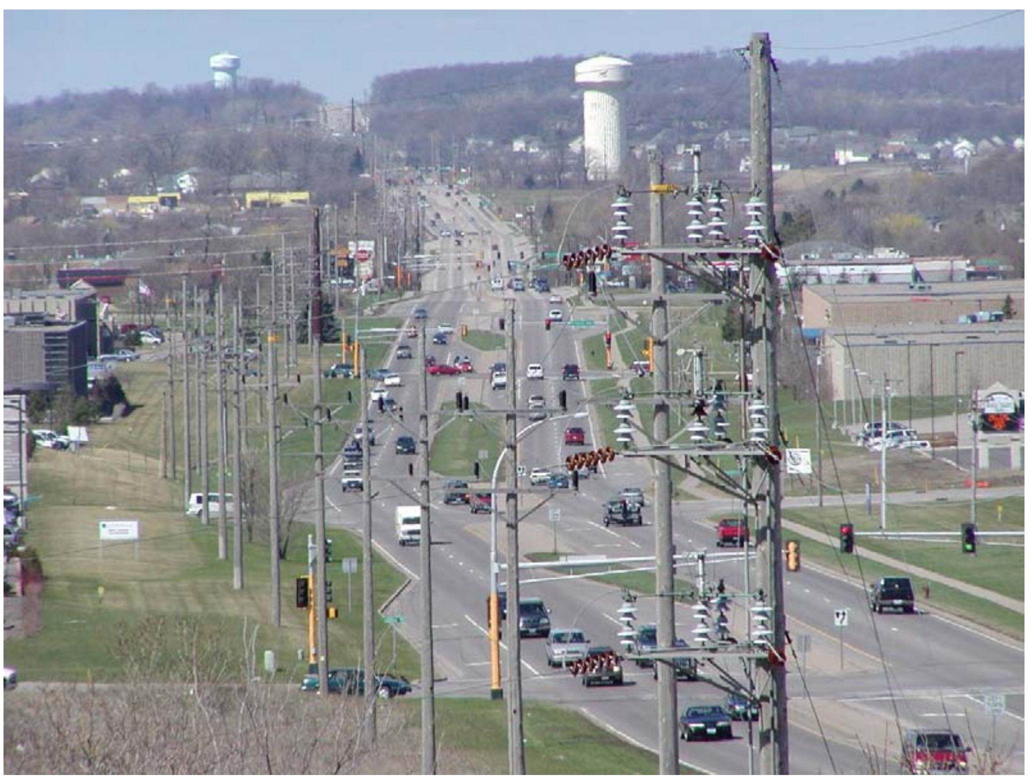

Figure 2. Pre-project view of CSAH 42 looking west (taken by the authors)

Overall Corridor Plan and Concept

An overall plan was developed for the entire CSAH 42 corridor in 1999 by BRW, Inc. consultants and other team members. This study noted the importance of the CSAH 42 corridor as the only continuous east-west arterial in the southern portion of the Twin Cities metropolitan area with good connections to the north-south freeway system. It also noted the importance of the roadway in serving major retail areas, including a regional shopping mall and a number of lower level retail facilities. The study noted a number of issues along the corridor, including large and growing traffic volumes, through-oriented travel, existing access management issues, and a high density of traffic signalization in some portions of the corridor.

The study noted that a low level of traffic congestion existed in the late 1990s at peak hours under current traffic conditions, but, given development growth rates along the corridor, that traffic would likely double over the next 25 years. The study also noted a problematic lack of a supporting roadway system along portions of the corridor, so that trips between commercial developments along the corridor had no route options but to return to the CSAH 42 mainline.

The 1999 overall corridor plan identified a range of potential solutions for the corridor, ranging from no-build to a high-cost alternative involving grade separating a number of high-volume intersections. The no-build alternative produced a large number of traffic level service F delays in the western half of the corridor by the year 2020. The recommended alternative involved a number of moderate cost improvements that would largely meet the desired objectives for level of service, travel speed, and safety. The recommended improvements included the following:

- Better access management and better coordinated land use planning throughout the corridor
- Addition of through traffic lanes in selected portions of the corridor
- Traffic signal modifications and signal removal at selected locations
- Development of supporting roadway systems, primarily backage roads at selected locations
- Addition of auxiliary (right and left turn) lanes in selected locations

Sub-Corridor Examined for This Paper

More detailed planning studies and design work has followed for particular segments of the corridor that are most in need of improvement. One of the most challenging sections of the corridor turned out to be the centrally located segment running between the cities of Savage and Burnsville, which also straddles the boundary between Scott and Dakota Counties. This segment (Segment 8 of the 1999 overall corridor plan) runs between Glendale Road in Savage to CSAH 5 in Burnsville, a distance of about 1.6 miles. Segment 8 was selected for a detailed planning study and ultimately for design; the detailed planning study took place in 2001 and 2002, and design work was finished in 2006. Figure 3 shows a map of Segment 8. The county and city boundary are in red.

When detailed planning began, Segment 8 had four through traffic lanes, raised or flush/grass medians throughout, and an extensive system of auxiliary turning lanes. There were very few (six) direct driveway accesses along this segment of CSAH 42; all but one of these had right-in right-out access only. However, there were 14 minor public road accesses, all with full movement access. There were six traffic signals within the 1.6 miles (including one on each end), creating an average spacing between signals of less than the half-mile or more that would be optimal for allowing through traffic to move at the desired mean travel speed at peak times.

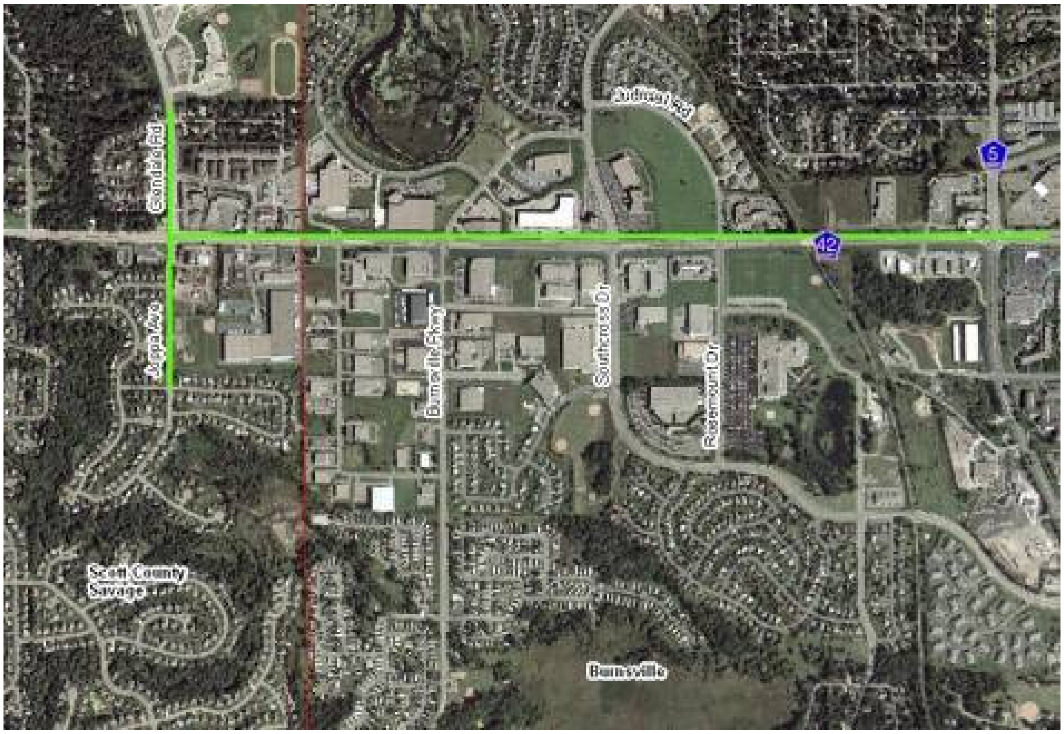

Figure 3. Aerial view of the sub-corridor (from Dakota County, Minnesota)

The two signals at the western end of the corridor were spaced only about 700 feet apart. Traffic signals, in particular the most closely spaced one at Huntington Avenue, were added along this corridor on an incremental basis to deal with “spot” safety issues; there was never a systematic analysis of traffic signalization until the detailed planning study was conducted after 2000. This, along with the lack of a supporting roadway system, turned out to be the main consideration in improving the future performance of the sub-corridor.

Segment 8 is a mixed land use corridor. The western and eastern portions are characterized by strip commercial development, while the middle portion is more industrial in nature. Figure 4 illustrates the development patterns: commercial areas are in red, industrial areas are in purple, undeveloped areas are in light green, a dashed black line represents the county and city boundary, and CSAH 42 is a solid red line. There are also recreational land use areas immediately adjacent to the corridor segment. One feature of the sub-corridor that became evident during this study was the lack of a supporting road system and, even more importantly, a supporting road system that crossed city and county boundaries. It can be seen in Figure 4 that the supporting roadway system parallel to CSAH 42 is incomplete and fragmented. Of particular interest is that development has been allowed to occur in places where backage roads could have been developed and supporting roads are not connected across city and county boundaries. Instead, a major warehouse and a city park were developed over logical routes for backage roads. Past lack of communication among the local government jurisdictions is quite visible on any map or aerial photograph.

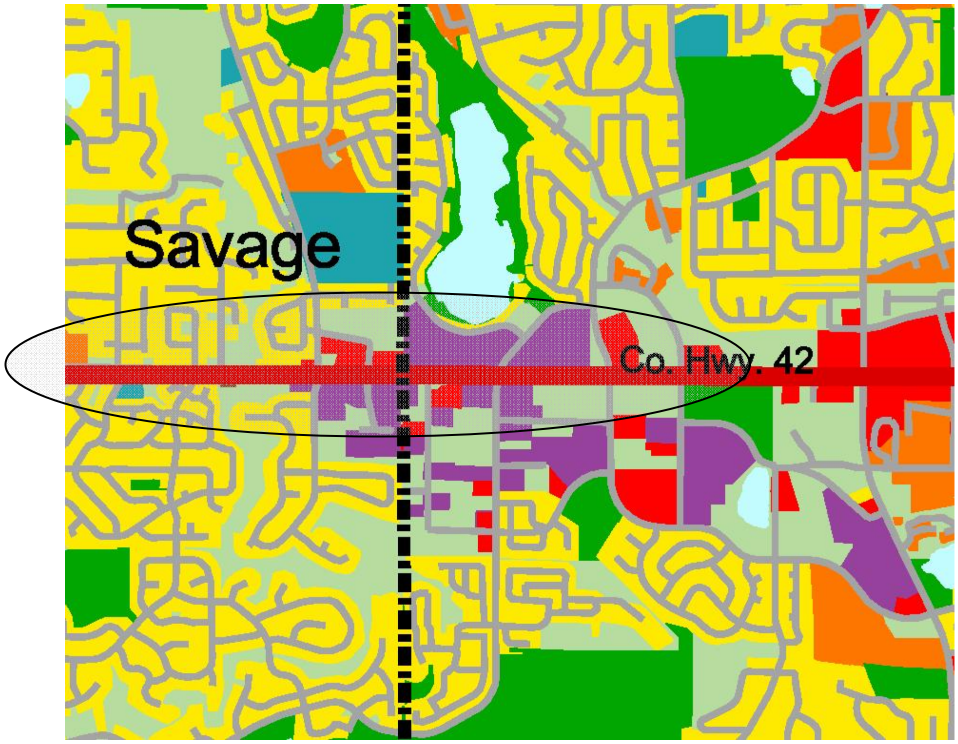

Figure 4. Corridor area land use map, circa 2000 (from Metropolitan Council)

Critical Issues for the Sub-Corridor

This corridor provides an excellent illustration of the proposition that access management is not just about driveways and medians. In fact, there were very few private, direct driveways and very few full median openings, yet access management concerns did exist in the form of many full intersections with minor public roads, the lack of an effective supporting public road system, a high density of traffic signalization, a lack of intergovernmental coordination of both land use planning and transportation system development, and a lack of necessary supporting roadway systems for land development.

PROACTIVE ACCESS MANAGEMENT PRACTICES FOR CSAH 42

Recommended Improvement Types for the Sub-Corridor

The detailed planning study conducted in 2001 and 2002 by a team led by Howard R. Green Company quickly began to focus on alternatives that involved a moderate-cost set of improvements to the Segment 8 sub-corridor. This is because high-cost improvements such as interchanges did not appear to be needed within the planning time horizon to attain most safety and operational objectives, and no-build approaches produced a situation by the year 2020 in which the sub-corridor was dysfunctional in terms of traffic flow, traffic level of service, and mean travel speed. Much of the poor future performance could be attributed to the high density of closely spaced traffic signals. The moderate-cost treatments considered included the following:

- Adding through lane capacity
- Adding auxiliary lanes, mainly at major public road intersections
- Closure of direct, private driveways

- Traffic signal removal and traffic signal system optimization
- Reconfiguration of minor public road intersections to reduce full median openings, precluding of some movements, and reduction of conflict points
- Completion of an alternative access road system to better interconnect commercial areas along the corridor
- Better coordination of land use planning and transportation planning along the sub-corridor

Detailed Sub-Corridor Planning and Stakeholder Involvement and Education Process

As might be expected given the commercial and industrial land uses along the sub-corridor, mentioning the moderate-cost treatments listed above provoked concern and opposition from business owners and developers. Businesses in the vicinity of Huntington Avenue went so far as to organize a small group to gain a greater opportunity to provide input during the detailed planning process. Most business concerns revolved around the prospect of the removal of traffic signals and the reduction of through and left turning movements at selected minor public road intersections. There was much less controversy about other potential treatments, including driveway closures.

At this point, the project planning team decided to embark on a comprehensive program of business stakeholder involvement and education. This program included the following components in addition to a more standard set of public involvement meetings and open houses:

- **A business inventory and classification**. A complete inventory of commercial and industrial businesses along the sub-corridor was prepared, along with a listing of vacant, lease-able spaces and vacant, developable land. Commercial businesses were classified into three groups: drive-by, destination, and mixed. Drive-by businesses are those that depend primarily on impulse traffic and perhaps on new customers. An example would be a convenience store. Destination businesses largely depend on planned trips and new, unfamiliar customers. An example would be a furniture store. Mixed businesses are in between. As is generally the case along arterial roadways, a large majority of businesses (about 80%) are either mixed or destination businesses. Drive-by businesses tend to be the most concerned about changes in access management because their customers are more impulsive and unfamiliar with the area. The inventory was used in part to identify the businesses that might be expected to be most concerned about changes to the roadway.
- **A formal meeting with the Huntington Avenue area business group**. This group was already organized, and the meeting was held to better understand their concerns about the sub-corridor, its problems, and their likely position on various potential improvements. If there was a “center” of discontent about the possibility of change along the sub-corridor, this was thought by the planning team to be it. This meeting in effect became a focus group about a small portion of the sub-corridor.
- **One-on-one interviews with selected businesspersons along the corridor**. Personal interviews (or telephone interviews) were conducted with a selected set of business owners and managers along the corridor. Emphasis was placed on drive-by businesses, businesses in the immediate vicinity of Huntington Avenue, and two major industries in the vicinity of Huntington Avenue who had not joined the Huntington Avenue group mentioned previously.
- **Development of a microsimulation traffic model for the sub-corridor**. The consulting team developed a microsimulation to illustrate the future performance of the sub-corridor under various improvement scenarios. SimTraffic 5.0 was the software used for this purpose. The micro-simulations were used at various times in the planning process as educational and discussion tools. Figure 5 shows a screen shot of a simulation of the “No-Build” scenario during the p.m. peak period. In this example, eastbound traffic backs up all the way between one signal and the

next. This alternative future was one that most business owners along the corridor were very eager to avoid. The visualization helped them understand how detrimental certain features of the existing roadway (especially the close traffic signal spacing) would become for their businesses as traffic doubled along the sub-corridor.

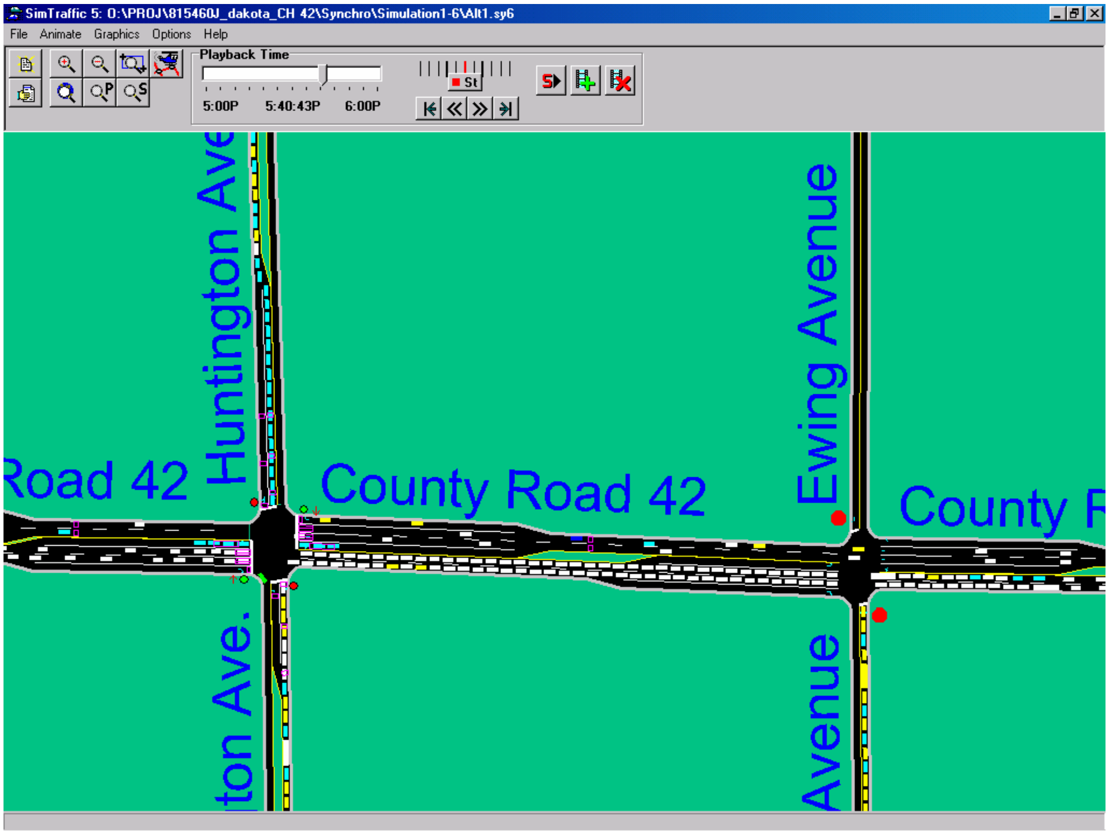

Figure 5. Screenshot of the results of a microsimulation, p.m. peak, no-build alternative (from Howard R. Green Company)

The outcomes of this involvement and education process were interesting. As a first benefit, a very good understanding of the businesses most likely to be impacted by change was developed. For each of these businesses, estimates of additional travel time and circuity of distance for customers were eventually developed, given the recommended improvements and changes in access. Several of these estimates were diagrammed for presentation to city and county officials so that they could understand the tradeoffs involved. Added access travel time could be shown to be relatively minor compared to the offsetting reductions in travel time on the mainline that could be gained through such strategies as adding through travel lanes, removing selected traffic signals, and limiting movements at minor public road intersections. Figure 6 shows an example of a circuity-of-access diagram that was developed.

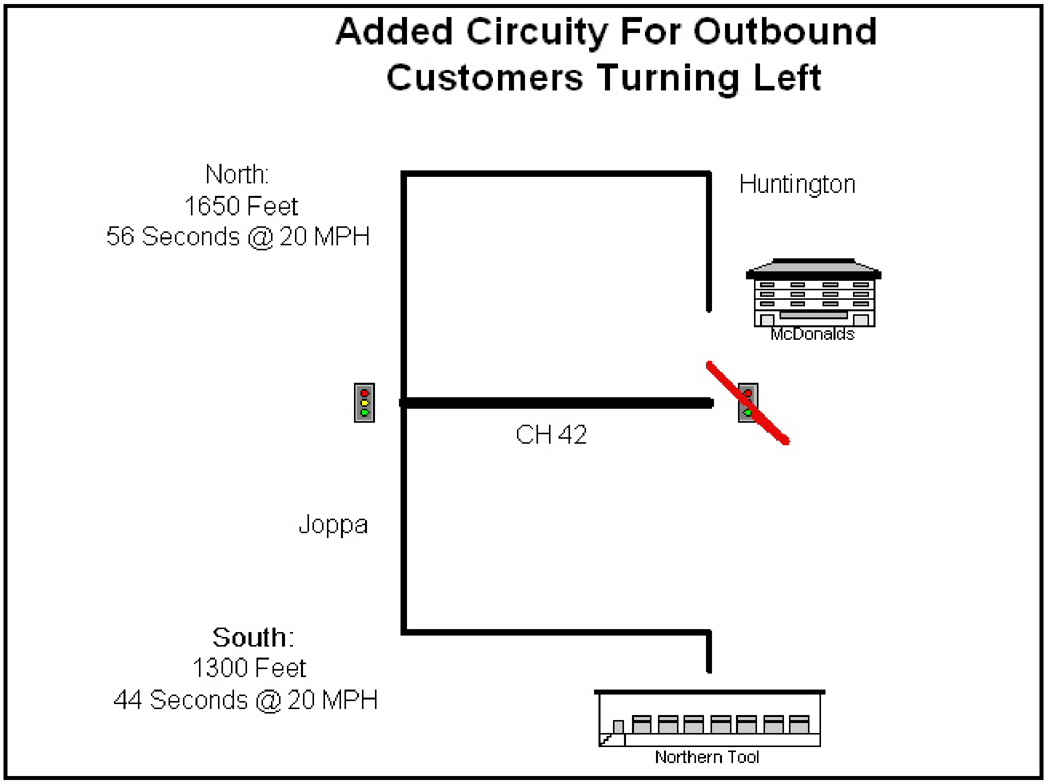

Figure 6. Circuity diagram example, Huntington Avenue area (from Plazak, 2002)

A second benefit was that the meeting with the Huntington group and the interviews revealed that the business community was already well informed about the potential improvements along the sub-corridor and that many of them understood that something substantial had to be done to handle future traffic volumes. There was a high level of support for adding through lanes along the corridor, but a need for additional education about access management, supporting roadway system development, and particularly the impacts of traffic signals and close signal spacing. This educational need was mainly filled by the micro simulation.

A final benefit of the process was that almost all businesses came to understand that a project involving moderate-cost improvements would be beneficial for most of them. Most businesses became supportive of a concept in which the project was staged with development of a supporting road system first and changes to the mainline following only after full completion of the backage road system. Opposition to the proposed alternative subsided until there was only one business in open opposition out of about 50 included in the business inventory. This business was a drive-by-oriented fast food restaurant located on a corner lot immediately next to the Huntington Avenue traffic signal. The owner and manager indicated that they felt that traffic signal removal would be highly detrimental to their business. Most businesses bought into the proposed alternative and felt that they had contributed the concept of staging the project.

A consensus began to develop among the business community that a reasonable improvement plan would involve a staged project, in which a supporting roadway system was developed first, followed by capacity improvements, intersection modifications, and traffic signal system modifications. This plan was agreed to by the two cities and counties involved. The fast food restaurant at the corner of Huntington Avenue remained concerned and opposed to the consensus alternative.

Final Plan for the Sub-Corridor

Because of the success of the business stakeholder involvement and education process, the final plan for CSAH Segment 8 was able to include what might be termed "difficult-to-accomplish" access management treatments, including the closure of almost every private driveway, the removal of two traffic signals, and the partial closure of several median openings along the route. Figure 7 shows a detail of the recommended design. The mainline is in yellow, the raised medians are red, new supporting roads are in purple, two three-quarters intersections are visible, and only one traffic signal is remaining.

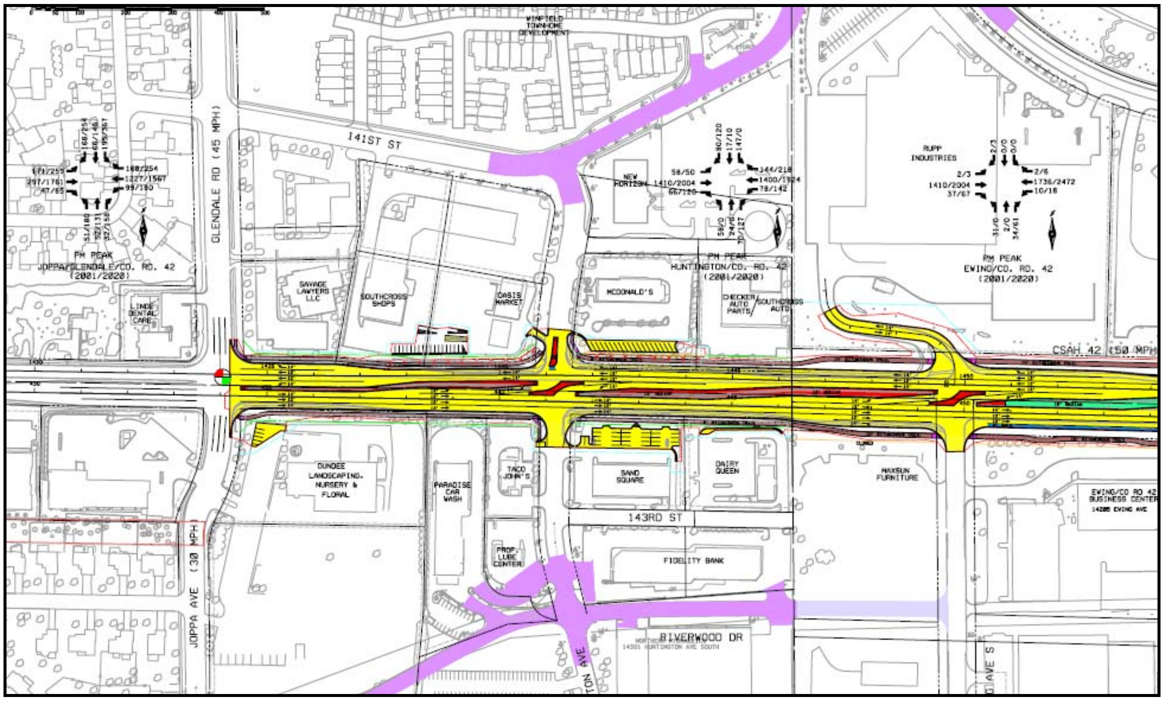

Figure 7. Detail from recommended design around Glendale Road and Huntington Avenue (from Dakota County, Minnesota)

The final plan included the following features:

- Two additional through traffic lanes for the entire 1.6 miles. The resulting cross-section would be a six-lane, median-divided roadway with auxiliary turning lanes.
- Closure of all but one direct driveway access and restricting it to a right-in right-out access. The remaining direct driveway access serves a major industrial facility and is well-spaced from adjacent minor public road accesses.
- Removal of the two traffic signals located at Huntington and Southcross. Four traffic signals would remain in operation, each with a spacing of greater than one-half mile to the next.
- Substitution of partial movement intersections for full intersections at Huntington, Ewing, Southcross, and Newton. The intersection design template chosen for the minor public road intersections was a three-quarters intersection, which allows for inbound right turns and left turns from the mainline, but no outbound left turns and no crossing movements (see Figure 8). This design reduces conflict points from 32 in a typical intersection to 10.
- Development of a more complete supporting (backage) road system along the sub-corridor, to be completed prior to the development of changes to the mainline of CSAH 42.

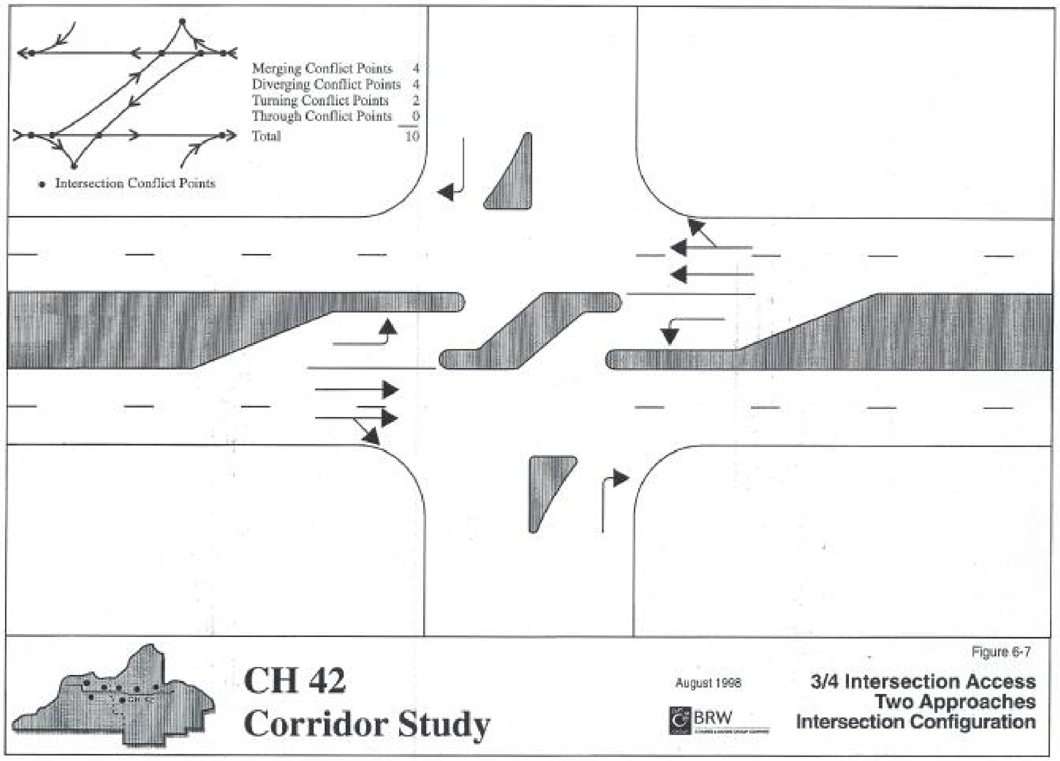

Figure 8. Three-quarter intersection design (from BRW, Inc.)

Project Design and Current Construction Status

The design of the project moved ahead between 2002 and 2006. Several open house-style meetings were held along the way by the design engineering team (led by Bolton and Menk) to do the following:

- Present the recommended roadway design
- Outline activities and the schedule to complete the project, especially project staging
- Discuss potential construction impacts on adjacent properties during construction
- Identify future public involvement opportunities

The supporting roadway system was designed beginning in 2005. It should be completed in 2007. The entire project, including all mainline improvements, is scheduled to be completed in 2008.

CONCLUSIONS

A number of lessons can be learned from this case study project, including the following:

- Access management is not just about driveways. Considerations such as supporting roadways, coordinated land use planning, minor public roadway intersections, and traffic signal density are just as important to keep in mind along commercial arterial corridors.
- Traffic signal removal is difficult, but may be of great value as a strategy for managing access along urban and suburban arterial corridors.
- Close involvement of the business community should occur throughout the planning and design process for corridor access management projects along commercial corridors.

- An inventory of businesses by type is useful for understanding which businesses in a commercial corridor might have the most at stake under various project alternatives.
- Microsimulation is very valuable as an educational tool, suggesting a "seeing is understanding" approach. In the case of this project, businesses came to understand the undesirable outcomes of the "no-build alternative" as well as the implications of keeping the closely spaced traffic signals.
- In the end, not all business owners and developers will be satisfied, but the majority probably can be satisfied even when difficult changes are proposed.
- Project staging can be a valuable tool for gaining support or at least for reducing opposition. In the case of this project, designing and building the supporting road system first and then modifying the mainline became a strategy that both the businesses and the local jurisdictions could support.
- Access management problems are usually generated incrementally, and they can be undone or at least partially improved in the same way. This project will eventually involve the provision of a supporting road system that was neglected, removal of two traffic signals that were unwisely installed, and reductions in private driveways and minor public roadway access.

A modified version of business stakeholder involvement and education process developed for the CSAH 42 project has recently been used in developing a plan for a section of U.S. Highway 10 in the city of Anoka, Minnesota. This highway segment is located in the northwest part of the same metropolitan area as CSAH 42 and is an arterial roadway in a commercial area that will almost certainly need to be converted into a freeway given anticipated future traffic volumes. Again, the process appears to have been successful in creating a near-consensus about the preferred direction for improvements. Components included a group meeting with concerned businesspersons, an inventory of businesses along the route, personal or telephone interviews with selected business owners who were thought to have the most at stake, a planning and design charette with business owners only, and an open house for the general public, at which general project alternatives were discussed.

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Thirteen County Road Safety Audit Reviews: Lessons Learned

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ABSTRACT

The Minnesota Department of Transportation (Mn/DOT) completed a Comprehensive Highway Safety Plan (CHSP) in 2004 that identified the critical safety strategies that, if implemented, were most likely to reduce fatal and life changing injury crashes in order to help meet the adopted safety goal of getting below 500 fatal crashes in 2008. In addition, one of the key conclusions of the CHSP was that, for Minnesota to be successful at meeting the goal, additional safety investment was needed on local roads, where over 40% of fatalities occur, and in particular along county state aid highways, which have a 30% higher fatality rate than similar two-lane rural state highways.

These items taken together resulted in Mn/DOT directing highway safety dollars specifically towards counties and with the highest priority given to projects that involved implementation of projects from the list of the state's critical strategies. One of the projects funded in the first year of this program consisted of conducting road safety audit reviews (RSARs) of approximately 130 intersections in the 13 counties in the north-central part of the state that together comprise Mn/DOT's District 3.

The RSARs were completed by a team of seasoned safety veterans from CH2M Hill, Mn/DOT, the Federal Highway Administration, and the Local Technical Assistance Program Center at the University of Minnesota. It was originally expected that the results of the field reviews would primarily apply to the specific counties and intersections that were the subject of the review. However, during the conduct of the reviews it became apparent that the lessons learned from these field reviews, while still of value at the specific intersections, could in fact have widespread application along both the state and county system of highways. Some of the key observations include the following:

- Intersection sight distance restrictions were widespread.
- Mn/DOT's recommended guide sign layout often contributed to the sight restrictions.
- County highways were in generally good shape, but pavement markings were in only fair to poor condition.
- There was virtually no enforcement presence on the county system.

During the RASR process, two other important items were documented. First, from a risk management perspective, counties should consider adopting written maintenance policies, especially as it relates to pavement marking and sign maintenance. Finally, counties should consider developing standalone safety improvement programs.

Key words: Comprehensive Highway Safety Plan—Minnesota—safety audit

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Overview of the Strategic Highway Safety Planning Process

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ABSTRACT

The Federal Highway Administration (FHWA) has adopted a new national safety goal: reducing the fatal crash rate by 33% to 1.0 fatalities per 100 million vehicle miles of travel. In addition, the FHWA has requested each state to prepare a Strategic Highway Safety Plan (SHSP) that identifies how they will contribute to this effort by documenting the following items:

- The state's vision and goal for reducing the number of traffic related fatal and life changing injuries
- The key safety strategies that, when implemented, would be most effective at achieving the adopted goal
- An analysis of safety investments in order to confirm that achieving the goal is, in fact, feasible in a reasonable period of time.

CH2M Hill was selected to help four states (Minnesota, Nevada, Nebraska, and Mississippi) prepare their SHSPs, and even though the states are geographically diverse with a wide range of crash characteristics, the key conclusions of a data-driven analytical process are strikingly similar. First, each state has selected a "stretch goal," and the analysis indicates that the adopted safety goals can be achieved. However, the biggest challenge for each state appears to be institutional inertia. The analysis of alternative safety investment scenarios suggests that the only way to meet the safety goal involves changing how the safety program is delivered; it needs to be more comprehensive (all four safety "E's"), more systematic (all roads), more focused on a very short list of strategies linked to fatal crash causation factors, and more proactive. These characteristics do not describe how the safety program is currently being delivered in any of the states.

One final point also needs to be noted: the challenge dealing with the state legislatures. The data driven analysis clearly identified that safety investments should include a focus on seat belts, impaired drivers, young drivers, speed, and red light running enforcement. However, the legislatures (with very few exceptions) have repeatedly declined the opportunity to positively address these issues; fewer than one-half of the states have a primary seat belt law, states are hindered in their response to impaired driving, only a handful of states have a comprehensive graduated drivers license program, and only a few states (mostly on the coasts) allow automated enforcement.

The lessons learned from these very different states are consistent, support the guidance from the FHWA, and can be used at every level to adjust how safety projects are delivered in order to support the efforts to reduce the massive burden on citizens associated with traffic deaths and injuries.

Key words: safety—Strategic Highway Safety Plan

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Traffic Flow Characteristics of a Congested Work Zone in Missouri

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ABSTRACT

This paper analyzes the traffic flow characteristics for a typical crossover-type work zone in Missouri. Data were collected at locations upstream and within the work zone in dynamic merge system (DMS) condition and non-DMS condition (standard condition) on two different days each. The traffic characteristics under the two conditions were studied based on the speed-flow relationship, capacity, and time headway parameters. An increase in traffic flow on one of the afternoons resulted in a continuous decrease in the speed of vehicles, leading to congested conditions at certain locations within the work zone. Any causative factor leading to the intermittent queue formations along the work zone is proposed to be studied from the captured data. The oscillation (the unsteady forward movement of traffic with no immediate dissipation in queue) and the propagation of the queue from the downstream exit further upstream under congested conditions is also proposed to be studied. The effects of lane changes that occur before the merge point will be studied to determine the impact on queue lengths. From the cumulative curves of vehicle arrival number versus time, N-curves, individual vehicle information within the queues and upstream of the merge point, will be extracted. Cumulative curves constructed from these observations describe completely and in great detail the evolution of the resulting long queues. Headway distributions for free-flow, moderate-flow, and high-flow conditions will be proposed using statistical distributions that provide the best fit. This study is significant because it would indicate the general behavior of Missouri drivers and help establish parameters that can be used to model and simulate the traffic flow at other work zones and freeways in Missouri.

Keywords: headway distribution—N-curves—queues—work zone

Wisconsin Ramp Metering Operations and Implementation

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ABSTRACT

With the objective of improving wise deployment and efficient operations of ramp meters, the Wisconsin Department of Transportation (WisDOT) has undertaken a comprehensive evaluation project comprising several elements.

Already completed components include two evaluations of metering impacts on major corridors in the Milwaukee metropolitan area and on the freeway beltline in Madison. A third component was an evaluation of the timing and retiming strategy used by WisDOT. The most recently completed component is the *Statewide Ramp Metering and Control Plan*, which emphasizes applying consistent implementation criteria for potential installations.

WisDOT reviewed algorithms and metering practices used throughout the country. Each algorithm was evaluated against its benefits, relative costs, and concerns related to implementation and operation. Discussions were held with selected states, to aid in understanding other states' experiences with ramp metering algorithms. This effort has resulted in valuable insight into the widely varied practice of ramp metering nationwide.

Tasks currently underway include a revisit of metering benefits for existing deployments and the Statewide Freeway Surveillance and Ramp Control sketch planning activity, which is one component of the broader sketch planning project for statewide traffic operations. Preliminary results of a prototype implementation planning tool should be available late 2007.

Lastly, with a focus on improving metering operations in Wisconsin, WisDOT is utilizing microsimulation models, as well as simpler techniques, to improve their existing ramp meter algorithm. The initial effort is applied to five metering locations in Milwaukee and Madison to determine whether more optimal plans are achievable for low cost.

Key words: algorithm—meter—operations—ramp—Wisconsin

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INTRODUCTION

Ramp metering is a critical component of Wisconsin's intelligent transportation systems (ITS) program, which applies advanced technologies for traffic management and traveler information. Ramp metering has been relied upon in Wisconsin since 1969 and there are now over 120 meters in operation in Milwaukee and Madison.

Ramp meters are traffic signals on freeway entrance ramps that break up clusters of vehicles entering the freeway. Doing this reduces disruptions that clusters cause to freeway flow and improves merging safety. The *2005 Urban Mobility Report* estimated that there were 938,000 hours of delay saved in 2003 alone through the use of ramp meters in Milwaukee (TTI 2005). Furthermore, ramp metering evaluations have been completed in Wisconsin for the US-45 corridor in the Milwaukee area and also for the beltline freeway in Madison.

The benefits of ramp metering are well established in the literature, but much work remains in the area of operations, especially in an environment of substantial fiscal constraints and political pressure. The Wisconsin Department of Transportation (WisDOT) is confronted with competing pressures to deploy additional meters where they may or may not be warranted, to improve operations at existing meters or decommission them, and to maintain the ongoing ramp metering and broader ITS program without additional funding or staffing.

With the objective of improving wise deployment and exploring more efficient operations, WisDOT has undertaken a comprehensive ramp metering evaluation project, comprising several elements. This paper briefly summarizes the parts already completed and their key findings, describes ongoing and upcoming efforts, and discusses results, benefits, and next steps.

Components of this project that are already completed include deployment evaluations from Milwaukee and Madison, an evaluation of the timing/retiming procedure, a survey of practices nationwide, including interviews with selected system operators, and a statewide ramp control plan. Components that are underway include the Statewide Freeway Surveillance and Ramp Control sketch planning project, a revisit of the benefits of existing meter installations, a forthcoming evaluation of key metering locations utilizing microsimulation, and ongoing project-related ramp metering considerations throughout the five WisDOT regions.

Given the prohibitive time and expense needed to implement an entirely new algorithm in Wisconsin, the objective of the latter phase of this project is to increase economic efficiency through an exploration of opportunities to streamline deployment decisions and improve operations through low-cost modifications. The results of the research will collectively lead to a more reliable and consistent methodology for new ramp metering deployments throughout the state and for more efficient operations of meters, whether a new algorithm is implemented or not.

BACKGROUND AND RECENTLY COMPLETED RESEARCH COMPONENTS

In this section, three recently completed evaluations are discussed. The first is a before and after evaluation of a deployment in 2000 of additional meters on the US-45 corridor in Milwaukee. The second evaluation is from the beltline freeway in Madison, and third is an evaluation of the current retiming practice used by WisDOT.

The other recently completed task is a survey of practices and algorithms used around the country. This work included several interviews with ramp meter operators in other states and an initial pass at whether other strategies may be applied in Wisconsin for low cost.

Recently Completed Evaluations

The documents for the three evaluations discussed in this section are available on the Sketch Planning Workgroup page: http://www.topslab.wisc.edu/workgroups/sketchplanning.htm#ramp. The Sketch Planning project is further discussed later in this document.

Ramp Metering on US-45 in Milwaukee

Roughly 15 miles of southbound US-45 entering the Milwaukee metropolitan area is metered (Figure 1). In early 2000, seven ramps were furnished with meters. This was in addition to the six currently metered southbound on-ramps. WisDOT pursued a before and after evaluation of this installation. The report on this evaluation, *Evaluation of Ramp Meter Effectiveness for Wisconsin Freeways, A Milwaukee Case Study*, was published in October 2004.

The study was completed jointly by personnel at Marquette University and the University of Wisconsin, Milwaukee (2005a), and it covered several facets of operations and safety effects of ramp metering. The final report includes discussion of the following:

- Traffic diversion, primarily spatial diversion because of little evidence of modal or temporal diversion, utilizing cutlines crossing two parallel arterials. The data collected included 18 hours of operation during the before condition and 18 hours during the after condition.
- Origin-destination trip length changes based on license plate video survey. However, the original data was not available to the project team. Without knowing the sample size, collection times, or capture rate, meaningful conclusions are difficult to draw.
- Potential modeling approaches to evaluating ramp meter effects, including microscopic, mesoscopic, and macroscopic approaches.
- Traffic operations effects, including speeds and delays.
- Safety effects.

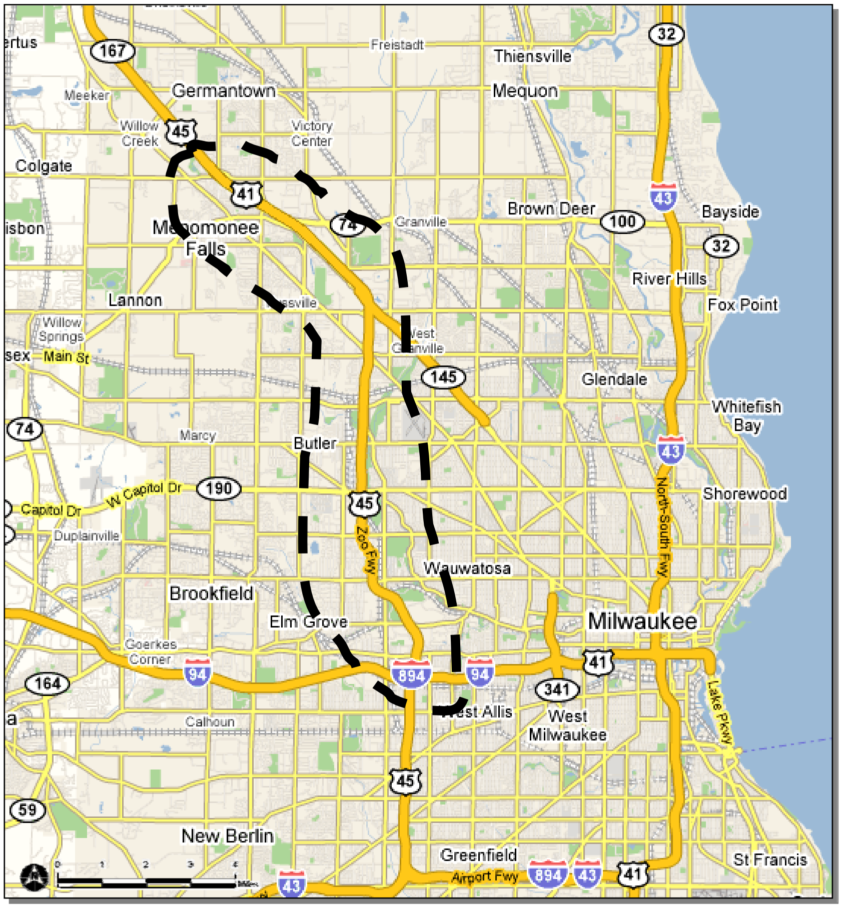

Figure 1. US-45 location map

The “with ramp metering” data collection for the diversion evaluation began three weeks after the additional meters were activated. Evidence from the Twin Cities ramp meter shutdown indicates that even eight weeks may not be sufficient to reach equilibrium after a system shock, although that was a larger change to the system (Mn/DOT 2001). Furthermore, only 18 hours of data were evaluated in each condition. With historical loop data readily available via the recently developed WisTransPortal transportation data hub, additional data will be incorporated and a validation of these results completed.

Among the key findings were increased mainline speeds and reduced crashes. Overall, vehicle hours of travel decreased by 2% following meter installation. It is noted in the report that much of the corridor was not congested during the evaluation times, so as traffic volumes continue to grow, this travel time savings is expected to grow. The existing meters were primarily deployed in the southern, more congested portion of the corridor.

A stated preference survey indicated that drivers would respond to delays at metered ramps. Where traffic volumes were heaviest or ramp queues longest, a significant number of drivers would divert their travel away from the freeway or from a specific ramp. If a metered ramp had waiting vehicles, 82% of surveyed drivers said they would take an alternate route.

Following meter deployment, an origin-destination survey indicated that drivers may be less likely to use the freeway for very short trips, which would cause less entering and exiting and less disruption to traffic flow. However, the original data from origin-destination license plate video survey was not available to the project team. Without knowing the sample size, collection times, or capture rate, any conclusions drawn from that information may not be reliable.

Travel speeds in the most congested south portion of the corridor increased by as much as 13% during the afternoon peak period. The average speed on the entire southbound US-45 section evaluated increased by 4% during the afternoon peak. New ramp meter operation, in conjunction with relatively minor geometric improvements in ramp merging areas and mainline resurfacing, resulted in a 21% crash rate reduction.

Ramp Metering on Madison Beltline

In July 2001, WisDOT implemented ramp metering along the US-12/18 Beltline freeway in Dane County. The evaluation analyzed the impact of ramp metering on the Madison Beltline from five metering locations, including qualitative and quantitative impacts such as travel time, traffic flow, safety, public perception, and air quality. The report was published in January 2005.

The findings showed a crash reduction. While the entire Beltline from Stoughton Road to Old Sauk experienced a 57% reduction in crashes, the area identified as the eastbound ramp meter influence zone near Whitney Way experienced an even greater reduction in crashes during metered and non-metered periods: 86% for both periods. The westbound ramp meter influence zone near Park Street and Fish Hatchery Road showed a 50% reduction in crashes during metered time periods and an overall reduction of 27%.

Ramp metering improved WisDOT's ability to mitigate effects of traffic incidents. About 96% of public safety agency representatives surveyed for the study found the time to clear crashes has improved because of the introduction of ramp meters along the Beltline, while approximately 64% of the agency respondents found that the time to respond to accidents has improved with ramp metering.

Despite significant growth in traffic volumes, travel times increased only slightly during three of the four metering periods, with a slight reduction in the westbound a.m. metering period. Three out of the four travel periods experienced a lower variability in travel speeds, which translates to improved travel time reliability, an increasingly prominent economic consideration. During the westbound morning peak period, the variation of travel times was reduced from +/- 10.9 seconds down to +/- 3.8 seconds after ramp metering.

Although the Madison Beltline has relatively few alternative routes, results from the ramp counts indicate that motorists at some locations are seeking alternative routes to avoid congested ramps.

Evaluation of Ramp Meter Retiming Procedure

Many ramp meters were added to the Milwaukee freeway system in the 1990s. After initial adjustments, many of these meters were not retimed for several years. By early 2000s, a standard retiming approach was developed, and in 2003 and 2004, all meters were retimed and based on this approach as well as feedback from operators. The need to balance quality freeway operations with the negative effects of ramp queue spillback was paramount.

A December 2005 report (UW Madison 2005b) documented a review of retiming process developed in southeast Wisconsin and evaluation of whether the process is effective in minimizing delay and crashes in the southeast Wisconsin freeway system. After the 2003–2004 retiming, no significant change in traffic flow was observed. Surveyed operators generally agreed with the approach of the retiming procedure, but field observations and adjustments based on experience are necessary. This is consistent with traffic signal retiming practice.

The evaluation recommended four key improvements. First, there needs to be a better understanding and accommodation of temporal variations in traffic flow and its effect on ramp meter timing. Second, although resource-intensive, simulation may provide valuable insight into alternative timing plans. Third, WisDOT operates their meters as local-responsive only (not systemwide), but relative to other states, meter operation is "quite sophisticated." And fourth, there remains a longer-term need to move forward with systemwide or corridor-based algorithms.

National Experience

According to the Federal Highway Administration, evaluations from across the country show that ramp metering reduces collisions on freeways and ramps from 15 to 50%. Ramp management strategies often increase travel speeds while reducing travel time and delay. Freeways that have metered entrance ramps usually carry more traffic than they did before metering began while attaining the improvements mentioned previously. Table 1 below provides a brief summary of common measures of effectiveness for ramp metering in other places.

Table 1. National ramp meter benefits (WisDOT 2006)

City	Study road	Speed increase	Travel time reduction	Crash reduction	Flow increase	Program initiation
Minneapolis	I-35	26%	-	- 27%	25%	1970
Portland	I-5	61%	12 min	- 43%	-	1981
Seattle	I-5		11.5 min	- 39%	62%-86%	1981
Long Island	Multiple	9%	-	- 15%	2%	1989
Detroit	I-94	8%	-	-50%	14%	1982
Austin	I-35	60%	-	-	7.9%	Late 1970s
San Francisco	I-80	-	- 1 min	-	14%	1974
Denver	I-25	57%	37%	-5%	-	1981
Milwaukee	US-45	6%–13%	5%	-16%	-	1969

Nationwide Ramp Meter Algorithms

In late 2006, WisDOT reviewed many ramp metering algorithms used throughout the country. Each algorithm was evaluated against its benefits, relative costs, and concerns related to implementation and operation. Furthermore, discussions were held with selected states, including Oregon, Colorado, and New York, to aid in understanding other states' experiences with ramp metering algorithms. This effort has resulted in valuable insight into the widely varied practice of ramp metering nationwide.

Below is a list of many diverse ramp meter algorithms (Scariza 2003; Zhang et al. 2001). Traffic-responsive ramp meters use local and/or coordinated ramp meter algorithms. This is an abbreviated and descriptive list that includes neither interview material nor the initial assessment of feasibility and benefits.

Local Ramp Meter Algorithms

Local control is a process of selecting ramp meter rates based solely on conditions present at an individual ramp. In some cases, congestion problems at the ramp may appear to be fixed, when in reality problems are transferred to or uncovered at upstream or downstream locations.

- **ALINEA.** The control input is based on the system output. The goal of ALINEA is to sustain near maximum flow downstream of the on-ramp by regulating the downstream occupancy to a target value, which is set a little below the critical occupancy at which congestion first appears.
- **ALINEA/Q**. This algorithm calculates two metering rates. The first rate is calculated exactly the same as ALINEA. The second rate that is calculated is the minimum rate needed to keep the ramp queue at or below the maximum allowable queue length.
- **FL-ALINEA**. FL-ALINEA uses flow measurements from downstream detectors rather than occupancy measurements.
- **MALINEA**. MALINEA addresses a shortcoming of ALINEA by measuring the upstream occupancy.
- **UF-ALINEA**. It simply uses the sum of the upstream flow and the ramp flow to estimate the downstream flow.
- **UP-ALINEA**. Uses occupancy measurements, but from upstream detectors, and estimates the downstream occupancy.
- **X-ALINEA/Q.** This is where any of the modified ALINEA algorithms are used with queue control. All of these algorithms, except for X-ALINEA/Q are less efficient than the traditional ALINEA algorithm but are useful when various occupancy measurements are not available.
- **Demand-Capacity**. This traffic responsive algorithm measures the downstream occupancy. If it is above the critical occupancy, congestion is assumed to exist. The metering rate is then set to the min rate. Otherwise, the volume is measured upstream of the merge, and the metering rate is set to the difference between the downstream capacity and the upstream volume.
- **Fixed-Rate or Time-of-Day**. Ramp meter timings are adjusted automatically by specified time-of-day parameters. This algorithm does not afford flexibility for changing traffic conditions.
- **Percent-Occupancy.** This strategy uses only upstream sensor occupancy measurements to identify and measure congestion. The critical occupancy is measured using historical data.
- **RPMS (Ramp Metering Pilot Scheme).** The heart of the algorithm is the function that cycles through each of the following conditions: determines if ramp metering needs to be switched on or off using mainline smoothed flows and speeds downstream of the ramp, determine if a new cycle length needs to be calculated, determine if queue adjustment is necessary, and determine the technical issues related to the ramp meter signals.

Systemwide Coordinated Ramp Meter Algorithms

This is a process of selecting metering rates based on conditions throughout the entire length of the metered corridor. This makes systemwide control more flexible in handling reductions in capacity that occur as a result of congestion or non-recurring incidents.

- **ARMS (Advanced Real-time Metering System)**. ARMS works on two levels. In the first level, a systemwide control policy is to maintain free flow conditions. A prediction and pattern recognition algorithm is also developed to predict in real-time the potential occurrence of recurrent congestion. In the second level, the algorithm works to resolve congestion once it develops. It does this by minimizing the congestion clearance time and queues on the controlled ramps.

- **BEEX (Balanced Efficiency and Equity).** BEEX seeks to minimize the total weighted travel time, which involves weighting both the freeway mainline travel time and the ramp delays.
- **Fuzzy Logic.** It can balance several performance objectives simultaneously, such as occupancy, flow rate, speed, and ramp queue. The performance objectives are divided into finite categories and then rules are developed with different weighting factors to relate traffic conditions with metering levels. Fuzzy logic can anticipate a problem and take temperate, corrective action before congestion occurs. With congestion indicators as inputs, the Fuzzy Logic can handle poor data, incidents, special events, and adverse weather without modifying the control parameters.
- **Linear.** The linear algorithm maximizes the weighted sum of ramp flows. It also computes a real-time capacity for each road segment. The drawbacks of this algorithm are (1) its performance is heavily dependent on accurate origin-destination data, and (2) it is static, i.e., it neglects the variation of travel time in its computation of ramp metering rates.
- **METALINE**. The metering rate of each ramp is computed based on the change in measured occupancy of each freeway segment under METALINE control and the deviation of occupancy from critical occupancy for each segment that has a controlled on-ramp.
- **Metering model for non-recurrent congestion**. It has a dynamic traffic flow model to describe the traffic flow process, explicitly links control with a clear set of objectives, takes into account systemwide physical and environmental constraints and projected traffic conditions, and uses a rigorous yet straightforward solution procedure to obtain real-time metering rates.
- **MILOS (Multi-Objective, Integrated, Large-Scale, & Optimized System)**. The areawide coordinator assigns target ramp metering rates to maximize freeway throughput, balance ramp queue growth rates, and minimize queue spill-back into the adjacent surface street interchanges.
- **SZM (Stratified Zone Metering)**. Effective in reducing ramp delays and queues, reducing freeway travel time and delay, increasing freeway speed, smoothing freeway flow, as well as reducing the number of stops.

Local and Systemwide Coordinated Ramp Meter Algorithms

The following algorithms have both local and coordinated capabilities.

- **Bottleneck.** For each ramp, the more restrictive of the two rates is chosen. <u>Local</u>: A control strategy compares the upstream demand with the downstream supply, then takes the difference of them as the locally determined metering rate. <u>Systemwide</u>: A coordinated control strategy first identifies bottlenecks, decides the volume reduction for the bottleneck based on flow conservation, and then distributes the volume reduction to upstream ramps according to predetermined weights.
- **Compass**. The most restrictive of the following two rates is selected. <u>Local</u>: Determines the metering rates from an ad-hoc lookup table, which has multiple levels for each ramp, determined by the local mainline occupancy, the downstream mainline occupancy, and the upstream mainline volume. <u>Systemwide</u>: Coordinated control use of off-line optimization to generate metering rates based on systemwide information. Compass addresses spillback through overriding restrictive rates. If the occupancy at a ramp queue detector exceeds its threshold value, the metering rate is increased by one rate level until the detected occupancy is back below the threshold level.
- **Dynamic metering control**. Local control attempts to maintain traffic conditions close to the target traffic conditions that are provided by area-wide control. It obtains metering rates through minimizing the total system travel time that includes travel time on freeway and delay on ramps, subject to demand and queue capacity constraints.
- **FLOW**. FLOW tries to keep traffic at a predefined bottleneck below capacity and works best at very high traffic volumes. The most restrictive of the following two rates is chosen. <u>Local</u>: The

metering rate associated with each upstream occupancy is the difference between the capacity and volume associated with the occupancy on the fundamental diagram. Systemwide: For the bottleneck metering rate, bottleneck locations on the freeway must be determined. The bottleneck metering rate for each ramp is then calculated by subtracting the bottleneck metering rate reduction from the measured on-ramp flow during the previous interval.

- **Helper (or incremental)**. A freeway corridor is divided into six groups consisting of one to seven ramps per group. Local: In the local traffic responsive metering component, each meter selects one of six available metering rates based on localized upstream mainline occupancy. Systemwide: If a ramp grows a long queue and is classified as critical, its metering burden will be sequentially distributed to its upstream ramps.
- **Linked**. Local: It is separated into a number of local traffic responsive controllers. This algorithm is based on the demand-capacity concept, and the local metering rate is determined based on upstream flow measurement at each location where the metering rate is equal to the target flow rate minus the upstream flow rate. Systemwide: Whenever a ramp's metering rate is in one of its lowest three metering rates, then the upstream ramp is required to meter in the same rate or less, and, if necessary, the further upstream ramps are also required to do so.
- **Neural Control**. Local: This algorithm uses feedback regulation to maintain a desired level of occupancy, or the target occupancy, which is usually chosen to be the critical occupancy. Moreover, the neural control algorithm is limited in adaptive control if on-line tuning is not implemented. Systemwide: This uses artificial neural networks to learn and memorize the metering plans generated by a traffic simulation model and a ramp control expert system.
- **RAMBO (Ramp Adaptive Metering Bottleneck Optimization)**. Local: RAMBO I evaluates plans generated based on ramp metering specifications. Systemwide: RAMBO II evaluates ramp metering rates based on forecasted traffic conditions along an extended section of freeway containing up to 12 metered on-ramps and 12 exit ramps. RAMBO II develops ramp metering rates using capacity and merge constraints for the entire freeway segment specified by the user.
- **SWARM (System Wide Adaptive Ramp Metering)**. SWARM has to stay within a TOD max and min range. The most restrictive rate is selected for each ramp. Local: The local control decides ramp metering rates based on local density. Systemwide: When a bottleneck is detected, a new set of ramp metering rates are determined. Downstream ramp meters will be shut off and upstream ramp meters will have a more restrictive timing. SWARM has the potential to be proactive, rather than reactive. It has a built-in failure management module to clean faulty input data from detectors. It also allows further adjustment to accommodate queue spill-back handling. It automatically adjusts timing for incidents and holidays.
- **ZONE**. Local: Zone provides for local control by using the occupancy control philosophy. Systemwide: ZONE divides a freeway into several zones of three to six miles in length. The upstream end of a zone is a free-flow area, whereas downstream end of a zone is a critical bottleneck. ZONE calculates metering rates based on volume control in each zone. To accomplish this, ZONE relies on proper division of zones, accurate estimates of bottleneck capacity, and accurate measurements of all in- and out-flows from a zone.

STATEWIDE RAMP METERING INITIATIVES

With the need to consider new ramp meter deployments around the state on an equal basis, WisDOT is developing high-level screening procedures. These new deployments must be evaluated on an equal basis not only with competing ramp meters around the state but also with other capacity improvements and other ITS strategies. There are two current components to this effort: the Statewide Ramp Control Plan and the Sketch Planning activity.

Wisconsin Statewide Ramp Control Plan

This effort laid the groundwork for an institutional and procedural plan for integrating the implementation criteria for ramp control strategies into statewide planning and programming processes. This project encompassed not only ramp meters but also ramp control gates. The focus was on implementation guidelines or warrants, and this part concluded with the issuance of a report *Statewide Ramp Metering and Control Plan* in July 2006 (WisDOT 2006).

The study cautioned that it is not appropriate to make final implementation decisions based on a high-level scan and that metering success is highly dependent on local conditions. The report presents a methodology for deployment considerations that could be applied statewide with minimal data input. This was key because of the limited resources available for additional data collection. The methodology was incorporated into a spreadsheet tool, the Wisconsin Ramp Analysis Tool, and piloted on select corridors.

Wisconsin Statewide Freeway Surveillance and Ramp Control Sketch Planning

Currently underway is the Statewide Freeway Surveillance and Ramp Control sketch planning activity, which is one component of the broader sketch planning project for statewide traffic operations. The other two components are traveler warning / information systems and traffic signal systems.

Each component will apply the overarching corridor planning methodology across the state to identify potential areas or corridors that may benefit from various ITS deployments. The second application will aid in prioritizing deployments subject to the considerable financial constraints facing the state. The criteria are based on readily accessible data such as traffic volumes, heavy vehicle volumes, forecast growth, and crash history.

Preliminary results of a prototype implementation planning tool should be available later in 2007. More information is available on the Sketch Planning workgroup page: http://www.topslab.wisc.edu/workgroups/sketchplanning.htm.

RAMP METERING OPERATIONAL EVALUATION

The current ramp metering evaluation project focuses on two aspects of ramp metering in Wisconsin. First is an across the board evaluation of before and after traffic flow and safety for each metering installation. This builds in part on the previous evaluations already completed on corridors in Milwaukee and Madison. The operations and safety data availability has been tremendously enhanced since those studies through the development of the WisTransPortal, a transportation data hub containing all loop detector data, crash data, and other information from the last ten years or more. See http://transportal.cee.wisc.edu for more information on this resource.

The second aspect is an in-depth look at low-cost alternatives for improving metering operations. WisDOT currently utilizes a time of day local ramp metering algorithm, and there has never been a comprehensive study of the effectiveness of this ramp metering algorithm in Wisconsin and whether or not the algorithm could be improved.

The existing ramp meter algorithm will be modeled at each of five locations, along with different ramp metering timing schemes to determine whether more optimal plans are achievable for low cost. Simple examples of possible improvements include modifying the volume to capacity thresholds currently in use and gradual quickening of the metering rate at the end of the metering time period so that the residual queue does not flood the freeway all at once.

It is expected that the results of this evaluation will lead to more efficient operations of meters utilizing the existing algorithm or a low-cost modification of it.

The five proposed ramp metering locations for evaluation include the following, which are intended to provide a meaningful cross section of operating conditions.

1. I-894 Northbound Corridor, Milwaukee. This corridor has five ramp meters and is delayed in both the AM and PM peak periods. This series of ramps will provide an opportunity to explore corridor-based metering operations.
2. I-94 Westbound at Moorland Road, Milwaukee. This ramp has very high volumes that back up onto Moorland Road during the afternoon peak, and the mainline volumes are relatively high.
3. I-94 Eastbound at 35th Street, Milwaukee. This ramp enters into a very dense central business district and has insufficient storage.
4. I-894/43 Westbound at 60th Street, Milwaukee. This meter has relatively sufficient storage with minimal delays on the mainline downstream.
5. US-12/14 Eastbound at Whitney Way, Madison. This location has both high ramp and mainline volumes, and the ramp has insufficient storage.

All controllers in use in these locations are type 170 except in Madison; it is a type 2070.

CONCLUDING REMARKS

This paper intentionally covers a broad spectrum of ramp metering issues and activity in Wisconsin. This was done in part because of the parallel needs for a synthesis of recent evaluation work, to draw together the statewide deployment planning efforts and to reiterate the emerging policy implications.

With increasingly scarce operating funding comes an increasing need to carefully and economically justify new ramp metering installations. Furthermore, with perennial political and public pressure to reconsider existing installations and their operations, the state is compelled to revisit the justification for ramp metering. Fortunately, the disparate needs for greater scrutiny point to the same three-fold solution: developing a defensible strategy for identifying and prioritizing ramp metering deployments statewide, revisiting the operations and safety benefits of recent installations, and evaluating low-cost yet effective ways to improve metering operations.

Related to that last part, the recommendations from preceding research and evaluations invariably include the need for further in-depth analysis of metering operations, utilizing microsimulation. The current objectives of the remaining research include finding and applying those solutions that will lead to improved metering operations.

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Many of these documents are available in PDF format at http://www.topslab.wisc.edu/workgroups/sketchplanning.htm#ramp

Impact of *Go!*, an Online Magazine About Transportation, on Teens' Career Plans

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ABSTRACT

Recruiting young people to the transportation industry is a difficult task because transportation is often misunderstood or it's just not on young people's minds (or their parents' and guidance counselors' minds) when they think about their future careers. To help make the transportation industry more visible and to show the range of interesting career possibilities to young people, the Center for Transportation Research and Education (CTRE) pilot tested a free online magazine for teens about careers in transportation. CTRE wanted to determine if regular (bimonthly) communication about the transportation industry would have a positive impact on teens' interest in transportation-related careers. Three issues were published during the winter and spring of 2007 (see www.go-explore-trans.org). At the end of the pilot test, CTRE analyzed the website traffic and number and diversity of subscribers (people who've provided their email addresses so they received notice when each issue of *Go!* was published). CTRE also surveyed subscribers about the impact of the magazine on their college/career plans and their impressions about careers in transportation. Based on the positive results of the pilot study, CTRE is pursuing national-level sponsors and ongoing, sustainable funding in order to continue publishing and generating interest in and enthusiasm for transportation among teens nationwide.

Key words: communication strategies—recruitment— workforce

INTRODUCTION

The transportation industry has been talking for years about the looming workforce shortage due to baby boomer retirements and the lack of young people interested in transportation. One ongoing question has been how the industry can recruit more young people. There are many recruitment programs being used across the country that introduce kids and teens to the transportation industry through career fairs, camps, and other hands-on events. The Center for Transportation Research and Education (CTRE) took a different approach.

CTRE regards workforce recruitment, at least in part, as a communications issue. One reason that young people may not be considering careers in transportation is that they don't know about them. To inform teens (and interested adults) about transportation, CTRE began publishing *Go!*, an online magazine, in January 2007. The purpose is to draw attention to the transportation industry and its wide variety of career possibilities. CTRE wanted to evaluate whether regular communication with teens about transportation would have any impact on their career plans. Would reading fun, lively articles about things like driving a snow plow simulator or designing all-terrain vehicles encourage teens to pursue a career in transportation?

ABOUT *GO!*

Content and Design

Go! shows readers that transportation is a varied and exciting industry full of opportunities for many different people. It covers all modes and the infrastructure in ways that young people can relate to. Each issue focuses on a different theme such as winter work, shipping, and design/engineering. The January/February 2007 issue, for example, included articles on a high school student maneuvering a virtual snowplow on a state-of-the-art driving simulator and how the City of Des Moines handles snow and ice on the streets and at the airport. Wondering how you get young people interested in shipping? Focus on something that they're waiting to be delivered—like the last book in the Harry Potter fantasy series.

Fun and quirky feature articles also provide basic information about specific careers: general qualifications, educational requirements, working conditions, etc. In addition, *Go!* runs five or six departments per issue like "School Spotlight," "Historically Speaking," "Green Scene," "Book Stop," "Train Your Brain," and "Mystery Photo."

The design incorporates numerous photos, different masthead graphics and colors in each issue, and a consistent layout that's easy to navigate. See Figure 1, the home pages of the first two issues.

Go!'s content is guided by the editor, Michele Regenold, a professional writer in transportation since 1996, who is currently earning her MFA in writing for children and young adults. An editorial board that includes high school and college students, educators, and transportation/public works professionals contributes regular feedback about content. Most of the content for the first three issues was produced in-house by Regenold and a graduate student/editorial assistant.

a. January–February 2007 (premiere issue) **b. March–April 2007**

Figure 1. Home pages of first two issues of *Go!*

Readership

Approximately 2,500–3,000 people visited *Go!*'s website each month from January through May 2007. About 25% of all visitors spend an average of four minutes on the site (the other 75% have likely landed on *Go!* via a search engine, don't find what they were looking for, and leave in less than ten seconds). The number of visits has increased steadily since the site launched in January 2007. About 5,500 people have bookmarked the site. Of the visitors who stick around for more than a minute, about 91% are in the United States, 2% are in Canada, and the rest elsewhere.

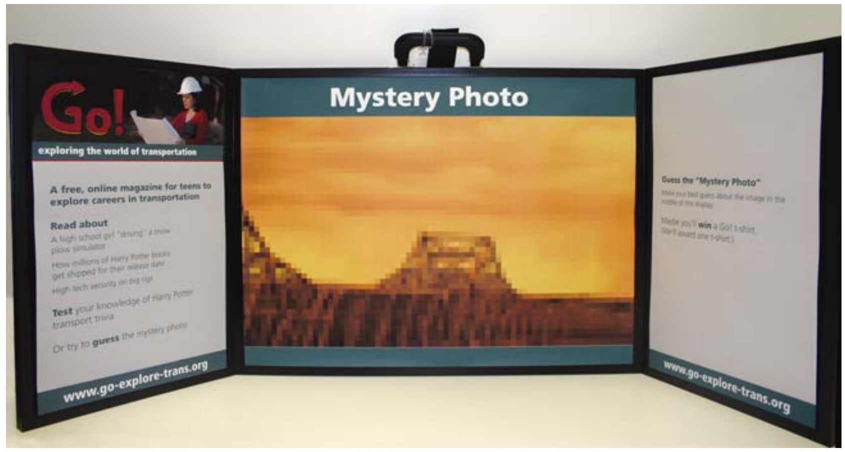
Go! encourages visitors to become subscribers by completing a brief form that asks for an email address and age range (13 or under, 14–20, and 21 or over). Subscribers receive an email announcing each new issue of *Go!* and they're eligible to win a *Go!* t-shirt in the "Mystery Photo" contest in each issue (see Figure 2). They can also comment on articles, and a few subscribers have done so. The number of subscribers is also growing steadily. Between January and May 2007, nearly 300 people had subscribed; about 100 of them are under 21. (*Go!* seems popular with the over-21 crowd.)

Figure 2. Ben Hucker, winner of the first "Mystery Photo" contest, wears his *Go!* t-shirt

Marketing *Go!*

Getting teens' attention, much less their email addresses, is tricky. Marketing efforts have been varied and continue to evolve. Because initial sponsor support came primarily from Iowa-based organizations, the *Go!* staff focused its attention first on marketing to Iowa high school and college students. For example, email notices were sent to 5,600 prospective Iowa State University students and 11,000 current Iowa State students in relevant majors.

Go! has also reached out to teens at Iowa events. Colorful *Go!* bookmarks have been distributed at the State Science and Technology Fair of Iowa, Invent Iowa, and numerous high school career fairs. In June, the *Go!* editor conducted transportation-related workshops for teens during the Iowa 4-H conference and an Iowa State University College of Engineering recruiting event. A tabletop display encourages people at events to guess a "Mystery Photo" to win a shirt. They can also subscribe to the magazine at the same time. See Figure 3.



a. Bookmarks **b. Tabletop display**

Figure 3. Marketing pieces for *Go!*

Marketing efforts have extended beyond state lines too. *Go!* staff contacted several organizers of construction career days for teens. Utah and Rhode Island distributed thousands of bookmarks in their students' packets. Other transportation and/or engineering teen recruiting programs in Georgia, Michigan, Texas, and Wisconsin have also distributed bookmarks. In addition, these groups received *Go!* t-shirts to give away as door prizes.

Marketing efforts have also focused on adults, particularly teachers, guidance counselors, librarians, and professionals in transportation. Messages were emailed to about 4,000 Iowa high school teachers and guidance counselors. Messages have also been sent via various listservs to librarians and professionals in transportation.

Go! has also attracted some publicity. The *Des Moines Register* covered *Go!*'s launch on the front page of its business section. The *Ames Tribune* also published a story about the launch and followed up a few months later with another story. *ENR* magazine's transportation editor Aileen Cho blogged about *Go!* The Transportation Research Board's e-newsletter has run a blurb about *Go!* for each issue. The *APWA Reporter* is running a story in its July 2007 issue about *Go!* as well.

Funding *Go!*

Through a combination of grants, sponsorships, and internal funding, CTRE produced the first three issues of *Go!* on a shoestring budget: about $27,500. Excluding the initial web design/development costs, the average cost to produce and market each issue was about $6,500. Most of that cost was salaries. *Go!* got off the ground with the generous contributions of the following organizations:

- American Public Works Association, Iowa chapter
- Associated General Contractors of Iowa Foundation
- Iowa State University's (ISU) Professional and Scientific Council (Retention and Recruitment Grant)
- ISU's Provost (Women's Enrichment Fund Mini Grant)

- ISU's Midwest Transportation Consortium
- ISU's CTRE
- Des Moines Truck Brokers
- Iowa Laborers/Employers Cooperation and Education Trust Fund
- Street Smarts (engineering consultants)

GO! READER SURVEY

In June 2007, approximately one month after the third issue went online, CTRE surveyed *Go!* subscribers to learn what, if any, impact *Go!* had so far. CTRE decided to restrict the survey to subscribers versus casual web visitors. Subscribers had already shown an interest in *Go!*, and CTRE anticipated more careful responses from them than from people just stumbling onto the site. Questions addressed subscribers' *Go!* reading habits, opinions about the content and design, and basic demographics.

The 300 subscribers received two separate emails inviting them to participate, and 38 people participated in the survey for a response rate of 13%. However, everyone did not answer every question, so actual response numbers vary.

Demographics of Survey Participants

Questions about gender, age, etc. were asked at the end of the survey so that participants would not be immediately put off by asking these questions. A majority of the survey participants were female, most were 21 or older, most were white, and most live in the United States. Tables 1–3 show the demographics of the survey participants:

Table 1. Gender of survey participants

Female	Male
59% (20)	41% (14)

Table 2. Age of survey participants

13 or under	3% (1)
14–15	9% (3)
16–17	3% (1)
18–20	9% (3)
21 or older	77% (26)

Table 3. Race/ethnicity of survey participants

American Indian and Alaska Native	9% (3)
Asian	9% (3)
Black	0%
Hispanic	9% (3)
White, non-Hispanic	79% (27)

(Note: participants could mark more than one)

Opinions of *Go!*'s Content

To get an idea of which stories subscribers found interesting, CTRE asked them to identify all articles they've read. In Table 4, each feature story is listed by its issue date and ranked by the number of subscribers who reported reading it. Subscribers were also asked to identify the departments they'd read, such as the "Mystery Photo," "Historically Speaking," and the puzzle called "Train Your Brain." These three, along with the "Tech Trends" department, were read by 62%–72% of survey respondents.

Table 4. Feature articles that subscribers read

Issue date	Article title	Percent who read article
January–February	Learning to drive a snowplow	69% (20)
	Brr! Cold weather construction	55% (16)
	Winter weather flying: How do they do that?	24% (7)
March–April	Big rigs: Not just for boys anymore	59% (17)
	Shipping Harry Potter: How do they do that?	45% (13)
	What's for supper?	31% (9)
May–June	Bridge building contests: For fun and profit (someday)	55% (16)
	On the GO with an ATV engineer	48% (14)
	Adventures in wayfinding	28% (8)

Note: Website visitors who spent more than 20 seconds on the site show a somewhat different rate of visitation. "Shipping Harry Potter," for example, was visited more often than "Big rigs."

Subscribers suggested that *Go!* do articles on traffic engineering, including women traffic engineers, motorcycle safety, work zone safety, future vehicle/driving automation, public works, the transportation planning process, crash tests, accident investigations, opportunities in construction for engineers, and "any types of unconventional careers that you don't necessarily go to college to major in."

Subscribers rated *Go!*'s content as "very interesting" (70%) and "sort of interesting" (30%). No one rated it as "boring." In terms of the visual design, 82% rated it "attractive," 15% rated it "okay but could use improvement" and 3% rated it "unattractive."

Impact of *Go!*

Gauging any impact of *Go!* magazine on regular readers is difficult; however, CTRE asked several questions to help determine if *Go!* was having any effect. One question listed all the careers that were mentioned in the first three issues of the magazine and asked respondents to check all that they found interesting. See Table 5 for the list of careers and responses, listed in order of most interesting career.

Table 5. Careers mentioned in *Go!* and the number of subscribers interested in them

Airline pilot	50% (16)
Bridge engineer	47% (15)
Community planner	44% (14)
Road construction worker	38% (12)
Equipment operator	34% (11)
Graphic designer	31% (10)
Mechanical engineer	28% (9)
Truck driver	28% (9)
Airport field maintenance staff	19% (6)

When asked if they were interested in learning about other transportation-related careers, 79% said yes. When asked how much subscribers knew about transportation before they started reading *Go!*, 64% said "quite a bit," 30% said "some," and 6% said "hardly anything." Considering the high number of people over 21 who completed the survey, it's likely that many of these adults are transportation professionals and therefore already know a lot about it. CTRE also asked subscribers if they've learned anything new about transportation and/or careers in transportation from reading *Go!* and 85% said yes.

Finding a topic interesting is one thing. Directing that interest toward action is another. To gauge this, CTRE asked the following questions:

- Would you consider pursuing a career in transportation (or recommend it to someone you know)?
    - Yes: 73%
    - Maybe: 18%
    - No: 9%

- Has reading *Go!* had any influence on your considering a career in transportation (or recommending one)? (asked of those who answered yes or maybe to the question above)
    - Yes: 59%
    - No: 41%

CONCLUSIONS

The reader survey sample is so small that it's difficult to draw significant conclusions from it. Nevertheless, the responses were overwhelmingly positive. Also, the increasing website traffic means that people are finding their way to the site and some of them are spending several minutes on it. It's still too early to tell if *Go!* will have an effect on young people's career plans, but so far it looks promising. At the very least, the magazine is introducing hundreds of people to careers they may have known little or nothing about. Meanwhile CTRE plans to continue publishing *Go!* (six issues for the 2007–2008 school year) because it believes the magazine is a powerful tool for workforce recruitment. CTRE is seeking additional sponsors and has developed a multi-tiered sponsorship program with sponsorships starting at $500. A major sponsor is Iowa State University's Midwest Transportation Consortium, which has committed to supporting a half-time graduate student editorial assistant. The magazine needs two to three years of support to thoroughly evaluate its impact on young people's career plans and the transportation workforce. Ultimately, CTRE believes *Go!* has a strong future.

ACKNOWLEDGMENTS

CTRE would like to thank the Iowa State University Professional and Scientific Council's Retention and Recruitment Committee for providing a grant in 2006 that helped launch *Go!*. That was the first money we received, and we were able to leverage it into about ten times more funding.

Overview of *SafetyAnalyst*: Software Tools for Safety Management of Specific Highway Sites

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ABSTRACT

SafetyAnalyst is a set of computerized analytical tools being developed for the Federal Highway Administration to assist state and local highway agencies in highway safety management. The main purpose of *SafetyAnalyst* is to improve a highway agency's systemwide programming of site-specific safety improvements. *SafetyAnalyst* applies advanced statistical techniques for safety management in four main analytical tools: Network Screening, Diagnosis and Countermeasure Selection, Economic Appraisal and Priority Ranking, and Countermeasure Evaluation. This paper presents a general overview of the *SafetyAnalyst* program features.

The Network Screening tool includes six algorithms to identify sites with potential for safety improvement: peak searching for high accident frequencies, sliding window for high accident frequencies, high proportion of target accident types, steady increase in accident frequency, sudden increase in accident frequency, and corridor analysis. This tool uses traditional screening techniques as well as new statistical methodologies such as the empirical Bayes (EB) approach.

The Diagnosis and Countermeasure Selection tool enables users to investigate the nature of safety concerns at specific sites through accident summary statistics, collision diagrams, and statistical tests for specific accident patterns. Through the use of an expert system, this tool also assists users in the diagnosis of specific accident patterns as well as the identification and selection of countermeasures that address these specific accident patterns.

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The Economic Appraisal and Priority Ranking tool performs economic appraisals of alternative countermeasures for a specific site or group of sites. Several economic criteria are available for use within *SafetyAnalyst*: cost-effectiveness, benefit-cost ratio, or net present value. This tool also provides the ability to determine an optimal set of countermeasures for a group of sites that will maximize the safety benefits of the improvements with a user-specified improvement budget.

The Countermeasure Evaluation Tool provides the ability to conduct well-designed before-after evaluations of implemented safety improvement projects. Two statistical approaches are provided for such evaluations: an EB approach for determining changes in accident frequencies and a nonparametric test for shifts in the proportions of specific accident severity levels or accident types.

Key words: countermeasure effectiveness—economic analysis—empirical Bayes—network screening—safety management—software

INTRODUCTION

SafetyAnalyst is a set of computerized analytical tools being developed for the Federal Highway Administration (FHWA) to assist state and local highway agencies in highway safety management. The main purpose of *SafetyAnalyst* is to improve a highway agency's systemwide programming of site-specific safety improvements. *SafetyAnalyst* incorporates state-of-the-art safety management approaches for guiding the decision-making process to identify safety improvement needs and has a strong basis in cost-effectiveness analysis. *SafetyAnalyst* will help highway agencies gain the greatest possible safety benefit from each dollar spent in the name of safety.

SafetyAnalyst addresses site-specific safety improvements that involve physical modifications to the highway system. *SafetyAnalyst* is not intended for direct application to non-site-specific highway safety programs that can improve safety for all highway travel, such as vehicle design improvements, graduated licensing, occupant restraints, or alcohol/drug use programs. However, *SafetyAnalyst* has the capability to identify accident patterns at specific locations and determine whether those accident types are overrepresented. In addition, *SafetyAnalyst* has the capability to determine the frequency and percentage of particular accident types along specified portions of the highway system.

SafetyAnalyst incorporates advanced statistical techniques for safety management approaches in four main automated analytical tools: Network Screening, Diagnosis and Countermeasure Selection, Economic Appraisal and Priority Ranking, and Countermeasure Evaluation. *SafetyAnalyst* development is being sponsored by the FHWA as a pooled fund project that has received financial support from over 20 state highway agencies (FHWA 2007). The *SafetyAnalyst* development work has been guided by a Technical Working Group (TWG) with representatives from each participating state highway agency. Two FHWA contractor teams are developing the software. An engineering team, led by Midwest Research Institute, is responsible for developing functional specifications for the software and testing the software to assure that it conforms to the functional specifications. A software development team, led by ITT Industries, has designed and developed the software. An interim version of the *SafetyAnalyst* software has been completed and is being tested by TWG members. The first public release version of *SafetyAnalyst* is expected in 2008.

This paper is organized as follows. Following this introduction, the paper presents an overview of the empirical Bayes (EB) methodology, which lies at the heart of many of the *SafetyAnalyst* methodologies. The paper then reviews the data requirements for using the software and provides a general overview of the capabilities found in the tools. This paper concludes with summary and key advantages of the software.

EMPIRICAL BAYES METHODOLOGY

The statistical approaches for most of the analytical procedures in *SafetyAnalyst* are based on the EB methodology. This technique was first reduced to practice in the highway safety field by Hauer (1997) to overcome the shortcomings of safety studies relying solely on observed accident data or accident rates. A brief background of the related issues and an introduction to this methodology are presented in this section.

There are several potential drawbacks to traditional safety studies based solely on observed accident frequencies or accident rates. Such studies may be biased due to regression to the mean. This phenomenon can be described as a characteristic of repeated measures data, in which high short-term accident frequencies are likely to decrease or low short-term accident frequencies are likely to increase, i.e., returning to the mean. It is important that such natural decreases (or increases) in accident frequency resulting from regression to the mean, which may lead to an overestimation (or underestimation) of safety, not be mistaken for the effect of a countermeasure.

The use of accident rates can also lead to biased results. Traffic volumes, or annual average daily traffic rates (AADTs), are used directly in the computation of this measure, i.e., accident rate = accident frequency/AADT (or some scalar multiple of this). The relationship between accident frequency and traffic volume is known to be nonlinear, and problems can occur when procedures based on accident rates treat that relationship as if it were linear. The nonlinearity in the relationship implies that equal accident rates do not mean equivalent level of hazard if different AADTs are involved, since the accident rate is expected to be lower at higher traffic volumes.

To remedy these issues, *SafetyAnalyst* uses an EB approach that combines observed and expected accident frequencies to provide estimates of the safety performance of specific sites that are not biased by regression to the mean. The EB approach incorporates nonlinear regression relationships between traffic volume and expected accident frequency.

Regression equations to predict accident frequency, typically developed by assuming that accidents follow a negative binomial distribution, are commonly known as safety performance functions (SPFs). SPFs are developed from a reference population of sites that share the same characteristics and have been generated for 90 combinations of site types and accident severity levels in *SafetyAnalyst*. Separate equations were generated for total accidents and fatal and all injury accidents for the site types listed in Table 1 (Harwood et al. 2004).

Table 1. Site types for which safety performance functions are included in *SafetyAnalyst*

Roadway Segments
Rural two-lane roads
Rural multilane undivided roads
Rural multilane divided roads
Rural freeways, 4 lanes
Rural freeways, 6+ lanes
Rural freeways within interchange area, 4 lanes
Rural freeways within interchange area, 6+ lanes
Urban two-lane arterial streets
Urban multilane undivided arterial streets
Urban multilane divided arterial streets
Urban one-way arterial streets
Urban freeways, 4 lanes
Urban freeways, 6 lanes
Urban freeways, 8+ lanes
Urban freeways within interchange area, 4 lanes
Urban freeways within interchange area, 6 lanes
Urban freeways within interchange area, 8+ lanes

Table 1. Continued

Intersections
Rural three-leg intersection with minor-road STOP control
Rural three-leg intersection with all-way STOP control
Rural three-leg intersection with signal control
Rural four-leg intersection with minor-road STOP control
Rural four-leg intersection with all-way STOP control
Rural four-leg intersection with signal control
Urban three-leg intersection with minor-road STOP control
Urban three-leg intersection with all-way STOP control
Urban three-leg intersection with signal control
Urban four-leg intersection with minor-road STOP control
Urban four-leg intersection with all-way STOP control
Urban four-leg intersection with signal control
Ramps
Rural diamond off-ramp
Rural diamond on-ramp
Rural parclo loop off-ramp
Rural parclo loop on-ramp
Rural free-flow loop off-ramp
Rural free-flow loop on-ramp
Rural free-flow outer connection ramp
Rural direct or semidirect connection
Urban diamond off-ramp
Urban diamond on-ramp
Urban parclo loop off-ramp
Urban parclo loop on-ramp
Urban free-flow loop off-ramp
Urban free-flow loop on-ramp
Urban free-flow outer connection ramp
Urban direct or semidirect connection

Two functional forms of default SPFs provided with *SafetyAnalyst* were developed using Highway Safety Information System data from four states. Level 1 default SPFs use traffic volume and roadway segment length as the primary explanatory variables to predict accidents. Level 2 SPFs also include other geometric and traffic control characteristics as explanatory variables. The interim version of *SafetyAnalyst* uses only the Level 1 SPFs, but both the Level 1 and Level 2 SPFs are expected to be used in the first public release version. Calibration procedures are available in the software to adjust default SPFs to reflect local conditions. *SafetyAnalyst* also provides the capability for user-defined SPFs to be entered for use with the software.

DATA REQUIREMENTS

SafetyAnalyst utilizes an agency's highway system inventory, accident data files, and data on implemented countermeasures. The data included in these files pertain to individual sites or locations within an agency's highway network. Additionally, other data files are maintained by the software for computational purposes when analyzing a site. The information within these files does not pertain to individual sites, but instead to a collection of sites or to all sites. The datasets in Table 2 are used within *SafetyAnalyst*:

Table 2. *SafetyAnalyst* datasets

Highway agency data files	Internal data files used in computations
Roadway segment inventory	Safety performance functions (SPFs)
Intersection inventory	Accident proportions
Ramp inventory	Countermeasures
Accident characteristics	Accident costs
Implemented countermeasures	Equivalent property damage only (EPDO) weights
	Beta distribution parameters

SafetyAnalyst has been developed to be flexible enough to fit into diverse highway agency operating environments. There are data importing methods supporting multiple commercially available database management software packages, including an embedded database packaged with the software to store and manage the data. All methods available to deploy the software require that specific data items from the highway agency file format and coding are converted to a common *SafetyAnalyst* format and coding. The exception to this requirement is that *SafetyAnalyst* can accommodate multiple location identifier systems.

Location identifier data are used to describe the exact location of a site within the highway network. Highway agencies have adopted different location identifier systems for their inventory highway data and other data files. Five basic systems of location identifier information are used by most highway agencies. These basic location identifier systems include the following:

- Route/county/milepost
- Route/milepost
- Route/segment identifier/distance
- Segment identifier/distance
- Link/node

SafetyAnalyst can accommodate data files that utilize any one of these systems to link the respective data to a particular location within the highway network. GPS coordinate systems may also be used, but must include an underlying linear referencing system to represent locations in one of the five basic systems.

Location identifier data also fulfill one of three vital requirements for agency data to be sufficient for use with *SafetyAnalyst*. The first requirement is that accident data must be able to be exactly associated with one roadway segment, intersection, or ramp record. This assignment can be automated in *SafetyAnalyst* via the location identifier data. The second requirement is that each location to be analyzed has at least one year of traffic volume data. The final requirement is that there are enough inventory or characteristic data available to classify locations into one of the site type categories listed in Table 1. The ability to assign an SPF appropriate to the type of site directly affects the ability to employ procedures based upon the EB methodology.

To facilitate testing and demonstration of the *SafetyAnalyst* software package, a test dataset has been developed from actual data for state highways in a five-county area of a particular state. These data include urban and rural areas as well as data for all types of facilities.

OVERVIEW OF THE ANALYTICAL TOOLS

SafetyAnalyst is comprised of four analytical tools that, when packaged together, incorporate all stages of the highway safety management process:

- Network Screening tool
- Diagnosis and Countermeasure Selection tool
- Economic Appraisal and Priority Ranking tool
- Countermeasure Evaluation tool

The technical capabilities and functionality of these analytical tools are summarized in the following sections.

Network Screening Tool

The purpose of the Network Screening tool is to review the entire roadway network or portions of the roadway network, under the jurisdiction of a highway agency, and identify and prioritize those sites that have potential for safety improvement. Identification of a site by one of the network screening tools means that an opportunity to improve safety may exist at the site, but this does not necessarily indicate that there is a correctable safety problem at the site.

The network screening tool provides four types of screening to identify sites with potential for safety improvement:

- Basic network screening for high accident frequency
- Screening for a high proportion of specific target accident types
- Detection of safety deterioration
- Screening for extended roadway corridors

Each of these screening techniques is summarized below.

Basic Network Screening

The basic network screening methodology utilizes EB principles to estimate the potential for safety improvement (PSI) of a site. This is the only network screening methodology that uses EB concepts. In general terms, the basic network screening approach combines observed accident frequencies with predicted accident frequencies from regression relationships (i.e., SPFs) to estimate the expected accident frequency for a site. The *SafetyAnalyst* user may select whether the PSI is expressed in terms of (a) an expected accident frequency or (b) an excess accident frequency (i.e., amount by which the expected accident frequency exceeds a normal or typical frequency for that type of site). The EB-adjusted expected accident frequency is normalized on a per-mile or per-site basis for ranking and comparing the PSI among sites. Sites with higher expected or excess accident frequencies (i.e., PSI values) are ranked higher in terms of their potential for safety improvement.

The basic network screening methodology may be applied to all site types (i.e., roadway segments, intersections, and ramps). The EB calculations are similar for all site types, with slight variations. When the site list includes roadway segments, the user must choose to perform basic network screening from among two approaches: peak searching or sliding window.

In the peak searching approach, each roadway segment is divided into windows, which are incrementally increased in size until they cover the entire site. In the peak searching approach, the windows are always contained within a given roadway segment and never overlap adjacent roadway segments. For each window, the expected accident frequency (or excess accident frequency) is calculated on a per-mile basis. Based on the statistical reliability of the expected value, the maximum expected accident frequency (or excess accident frequency) across all windows within a roadway segment is used to rank the PSI of that site relative to the other sites in the site list.

In the sliding window approach, the unit of analysis for a roadway segment is a window of constant user-specified length. This window is incrementally moved along contiguous roadway segments in geographical sequence that makes up a route within the highway system. At any position along the route, the sliding window may overlap previous windows if the length of the user-specified increment by which the window moves forward is less than the user-specified window length. Since a window does not necessarily end at the end of a site, window locations may bridge contiguous roadway segments. At each window location, the expected accident frequency (or excess accident frequency) is calculated on a per-mile basis. The maximum expected accident frequency (or excess accident frequency) for any window position within or overlapping that roadway segment is used to rank the PSI of that segment for comparison to the other segments being evaluated.

High Proportions Test of Target Accidents

The objective of this screening method is to identify sites that have a higher-than-expected proportion of specific target accident types and to rank those sites based on the difference between the observed proportion and the expected proportion for that specific target accident type. A site is identified for further investigation if the probability that the observed proportion of the specific target accident type at a site is greater than the expected proportion for similar sites. This approach strives to identify locations with an overrepresentation of particular accidents, which may facilitate the selection of countermeasures and identify locations that are good candidates to be cost effectively treated. In particular, sites not identified by the basic network screening algorithm due to a low accident frequency, but having a well-defined accident pattern, would benefit from this type of screening.

Detecting Safety Deterioration

The objective of this screening methodology is to identify sites where the mean accident frequency has increased over time by more than would be attributed to changes in traffic volume or general trends. Two types of increases can be detected:

- A steady but gradual increase in mean accident frequency
- A sudden increase in mean accident frequency

Both steady and sudden increases in accident frequency are detected by a statistical test of significance for the difference between the means of two Poisson random variables.

With the steady increase approach, sites are flagged for further investigation if the slope to the regression model fit to data of observed accident frequency versus year is greater than a user-selected limiting value. In essence, this detects whether the average accident frequency for a site in recent years appears significantly larger than in the preceding years. The sudden increase approach is similar to screening for steady increases in mean accident frequencies, with one exception. Rather than specifying a limiting value for the slope, the user specifies a limiting value of a percentage increase in mean accident frequency from one year to the next that might be indicative of a safety concern.

Output for this screening methodology consists of the list of sites identified for their PSI. With the other network screening methodologies, all sites included in output report are ranked relative to the others as to their potential for safety improvement.

Corridor Screening

Screening for extended roadway corridors is a unique approach among the analyses that are performed within the Network Screening tool. All other types of analyses are performed on a site-by-site basis. However, within a corridor analysis, sites are aggregated to investigate the accident history for a group of roadway segments, intersections, and/or ramps. Thus, sites with a common corridor number are analyzed as a single entity.

The user has the option to rank corridors by one or both of two basic measures: accidents per mile per year or accidents per million vehicle miles of travel per year. The first measure is an accident frequency value expressed on a per-mile basis. Thus, this first measure does not fully consider traffic exposure within a corridor. The second measure is an accident rate that does take into consideration the traffic volume exposure within a corridor.

Diagnosis and Countermeasure Selection Tool

The purpose of the Diagnosis and Countermeasure Selection tool is to enable users to investigate the nature of safety concerns at specific sites through accident summary statistics, collision diagrams, and statistical tests for specific accident patterns. Through the use of an expert system, this tool also assists users in the diagnosis of specific accident patterns as well as the identification and selection of countermeasures that address these specific accident patterns.

The diagnosis of safety problems at a site begins by analyzing accident history data to identify accident patterns of interest. Three tools are provided within *SafetyAnalyst* for identifying accident patterns: accident summary reports, collision diagrams, and statistical tests of accident frequencies and/or accident proportions. An accident summary report presents the frequency distribution for various accident characteristics using observed accident history. The user may choose to view the results in tabular form, bar charts, or pie charts so that overrepresentation of accidents can be easily seen. Similarly, a collision diagram can be generated to provide a visual representation of accident patterns. A basic collision diagramming capability that is not interactive or extensively modifiable is included within *SafetyAnalyst*. However, links to commercially available collision diagramming software packages are provided for software vendors who chose to make their products available though *SafetyAnalyst*. The final accident pattern identification method, the statistical tests for frequencies and/or proportions, utilizes an EB-based methodology rather than observed accidents alone.

Having identified one or more accident patterns of interest, the diagnostic investigation of those patterns can be continued through an expert system tool. This tool guides the analyst through appropriate office and field investigations to identify particular safety concerns at a site by posing a series of detailed questions intended to identify countermeasures that could potentially ameliorate specific accident patterns of interest at the site. These diagnostic questions include both traditional engineering considerations as well as a strong human factors component. Based upon the user's responses to the diagnostic questions, recommended countermeasures will be identified for further consideration within *SafetyAnalyst*. The user has the option to modify the list of recommended countermeasures resulting from the diagnostic questions. The user also has the capability to select countermeasures for further consideration without going through the diagnostic questions.

The final selection of countermeasures as candidates for implementation will be made by the user, not by the software. The logic that identifies appropriate countermeasures will consider the accident patterns and related site conditions investigated in the diagnostic process. The user can then select one or more of the suggested countermeasures for further consideration or can add other countermeasures that they consider appropriate. To aid in this process, a BenefitCost (B/C) calculator is available to the user. The user enters a target percentage reduction in accidents, a countermeasure service life, a rate of return, a desired B/C ratio, and an analysis period; the B/C calculator will estimate the maximum cost for a countermeasure that would provide the desired B/C ratio. This tool is useful to decide the appropriate scale of safety investment at a site (e.g., low-cost, medium-cost, or high-cost countermeasures).

Economic Appraisal and Priority Ranking Tool

The purpose of the Economic Appraisal and Priority Ranking tool is to enable the analyst to conduct an economic analysis of a specific countermeasure or combination of countermeasures at a specific site. The tool also assists in cost-effective programming of safety countermeasures across a roadway network. Uses for economic appraisal results include the comparison of alternative countermeasures for a particular site as well as development of improvement priorities across sites. This tool also includes an optimization program that is capable of selecting a set of safety improvements that maximizes the safety benefits of a program of improvements within a user-specified improvement budget.

The user begins an economic appraisal by selecting the countermeasure (or countermeasures) to be considered for implementation at each site. Recommended countermeasures from the Diagnosis and Countermeasure Selection tool, user-specified countermeasures, and any other countermeasures, which are considered appropriate for a specific facility type, are available for the user to select. Economic analysis of specific countermeasures uses default values of accident modification factor, service life, and construction cost. The user can use these default values or provide values more appropriate for their own agency’s experience.

There are several economic criteria and ranking measures available to assess alternative countermeasures. Multiple criteria may be selected for consideration, which allows for the comparison of results from the different approaches. These criteria include the following:

- Cost-effectiveness: dollars spent per accident reduced (i.e., the present value of constructing the countermeasure divided by the total number of accidents reduced)
- EPDO-based cost-effectiveness: dollars spent per weighted number of accidents reduced (i.e., the present value of constructing the countermeasure divided by a weighted estimate of accidents reduced by severity type)
- Benefit-cost ratio: ratio of the monetary present value of the estimated annual accidents reduced to the present value of the construction cost of the countermeasure
- Net benefits: monetary present value of the estimated annual accidents reduced minus the present value of the construction cost of the countermeasure
- Construction costs: present value of the construction cost of the countermeasure
- Safety benefits: monetary present value of the estimated annual accidents reduced
- Total accidents reduced: number of total accidents reduced during the analysis period
- FI accidents reduced: number of fatal and injury accidents reduced during the analysis period

Additionally, the user can customize output to display and rank all countermeasures considered or only the highest ranked countermeasure for each site.

In addition to establishing the priority ranking of countermeasures for each site evaluated, the priority ranking tool enables the user to rank countermeasures across multiple sites when the net benefits economic criterion has been selected. This ranking is accomplished through a mathematical optimization technique called integer programming (IP).

IP is a linear programming technique that maximizes or minimizes an objective function, taking into account integer valued constraints. In the case of this tool, the total net benefits for all sites considered are maximized, subject to the following constraints:

- Only one countermeasure can be selected for each site, including the no-build alternative
- The total construction cost for the above selected countermeasures does not exceed the available budget

The available construction budget is entered by the user at the time of the analysis. The user may also elect to run the optimization with a nonlimiting budget value so that the countermeasures with the highest net benefits are determined for each site.

The computations involved for integer programming are quite laborious and repetitive. Therefore, algorithm options, such as the time to run, iterations used, and tolerance limits for comparisons, are available for the user to adjust within the priority ranking tool.

Countermeasure Evaluation Tool

The purpose of the countermeasure evaluation tool is to estimate the safety effect of countermeasures implemented at specific sites. The tool is capable of assessing the safety effectiveness of a single countermeasure at specific sites or the collective effectiveness of a group of countermeasures in which the same countermeasures were implemented at a specified list of sites. In most cases, the effectiveness measures are expressed as a percentage change (decrease or increase) in accident frequencies or in specific target accident types or severity levels. In other cases, the change of interest might be a shift in the proportion of specific collision types or in the distribution of accident severity levels.

The effectiveness of countermeasures is determined through statistical before-after evaluations. *SafetyAnalyst* performs two basic types of before-after evaluations of implemented countermeasures: (1) estimation of the percent change in accident frequencies due to an implemented countermeasure or (2) estimation of the change in proportion of a target collision type or accident severity level due to an implemented countermeasure. When multiple countermeasures are evaluated, they are analyzed as a single treatment. Thus, a single safety effectiveness value is calculated for the group of countermeasures as a whole. The safety effectiveness of the individual countermeasures is not calculated.

The primary before-after evaluation technique is to estimate the percent change in accident frequency due to an implemented countermeasure. This technique implements an EB approach to compensate for regression to the mean. This technique uses SPFs developed from a set of reference sites similar to the improved site(s) to estimate the change in accident frequency that would have occurred at the improved site(s) had the improvement not been made. The basic steps in the EB approach are as follows: (1) estimation of the number of accidents in the before period, (2) estimation of the number of accidents in the after period in the absence of a treatment, and (3) comparison of the observed number of accidents after the treatment is implemented to the estimated number of accidents in the after period in the absence of a treatment.

The countermeasure evaluation tool also provides the capability to test for shifts in the proportion of specific target collision types and testing for shifts in the proportion of specific target accident severity levels. For such evaluations, a nonparametric approach is used to assess whether the treatment (i.e., countermeasure) affected the proportion of accidents of the collision type under consideration. In statistical terms, this is done by calculating the average difference in proportions across all sites and a confidence interval around that difference at a pre-specified confidence level (e.g., 90%). The statistical test performed is the Wilcoxon signed rank test, a nonparametric test that does not require that the differences follow a normal distribution. This test is rather conservative; it is also relatively insensitive to outliers in the data.

The primary output results from a countermeasure evaluation include an estimate of the effectiveness, precision estimates of the effectiveness, and indication of statistical significance. For the nonparametric test, the estimated effectiveness is an estimated median treatment effect rather than mean difference in proportions, since the test only uses those sites with an observed non-zero change in proportion.

SUMMARY AND KEY ADVANTAGES OF *SAFETYANALYST*

The FHWA is developing a set of new computer software tools, known as *SafetyAnalyst*, which can be used by highway agencies for safety management of a roadway system. *SafetyAnalyst* focuses on identifying the need for improvements at specific highway sites, identifying the most appropriate improvements for those sites, and making cost-effective choices to set priorities among the potential improvements. *SafetyAnalyst* can also make reliable estimates of the safety effectiveness of countermeasures that are implemented by highway agencies. The key advantages of highway agencies using *SafetyAnalyst* include the following:

- *SafetyAnalyst* integrates all parts of the safety management process and automates portions of the process that have been performed manually in the past. This will increase the efficiency with which highway agencies can perform safety management, leading to better decisions at lower total costs.
- *SafetyAnalyst* applies state-of-the-art analytical procedures in the safety management process. Many of the current analytical procedures used by highway agencies as part of the safety management process are deficient and can be improved. *SafetyAnalyst* incorporates analytical procedures that are scientifically and statistically sound and that overcome many of the deficiencies of current practice. Thus, *SafetyAnalyst* provides results that lead to better selections of safety improvement projects.
- *SafetyAnalyst* has a strong cost-effectiveness component, allowing highway agencies to develop safety improvement programs that provide the maximum safety benefit within any given budget for safety improvements. This can provide assurance to highway agency management and the general public that safety improvement funds are being spent wisely.
- *SafetyAnalyst* provides safety engineers and planners with all of the information needed to make safety management decisions efficiently, but does not presume to make those decisions for them. While automating the entire safety management process, *SafetyAnalyst* does not make decisions that require technical expertise and judgment. Highway agencies retain full flexibility to choose projects and safety improvements that best meet the needs of the traveling public.

- *SafetyAnalyst* encourages collection and use of improved highway inventory data. Many highway agencies currently have the inventory data needed to apply *SafetyAnalyst* to roadway segments between intersections. *SafetyAnalyst* can also be applied to safety management of at-grade intersections and interchange ramps where inventories of these features are available. Many highway agencies will have access to improved inventory data in the future through ongoing development of geographic information systems and asset management inventories. *SafetyAnalyst* has the capability to allow highway agencies to take a broader look at the entire highway network during the safety management process than most agencies currently do. By providing a platform for effective use of newly collected data, *SafetyAnalyst* provides an incentive for highway agencies to develop expanded inventory databases for a more comprehensive approach to safety management.
- *SafetyAnalyst* software will be available from the FHWA at no cost to highway agencies. The FHWA will also provide training and technical support.

ACKNOWLEDGMENTS

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Critical Issues in Surface Transportation

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ABSTRACT

The surface transportation system is a major factor in the United States' economic health, domestically and globally, and in the quality of life of its citizens. The lives of hundreds of millions of people are affected daily. There are major issues affecting surface transportation that must be recognized and addressed to ensure a prosperous future and way of life.

The improvement of transportation safety and increased mobility for all users are common goals and are expressed as determinants of a successful surface transportation system, though the two goals are frequently conflicting. It is necessary to ask: How can we resolve this conflict of goals? Or is such a resolution possible? If the antagonist to mobility is congestion, and it is said that we cannot build our way out of it, what can we do?

Along with these seemingly insurmountable issues are the constraints of funding, energy, and institutional arrangements: what new funding mechanisms are needed, what sources of energy will be available, and how must institutions adapt to be successful? With all these challenges, we must also continue to safeguard the environment for future generations.

This presentation will offer some ideas to address these critical issues.

Key words: critical issues—economic concerns—surface transportation system

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Focus Group Participants' Understanding of Advance Warning Arrow Displays used in Short-Term and Moving Work Zones

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ABSTRACT

In long-term work zones on multilane highways and/or freeways, the Federal Highway Administration has interpreted the Manual on Uniform Traffic Control Devices (MUTCD) to mean that only one advance warning arrow display can be used to denote the closure of a single lane. Where two or more lanes are closed, a single arrow display is used for each lane to be closed. However, in short-term applications or for moving/mobile work convoys, the MUTCD allows the use of multiple arrow displays to indicate a single lane closure. These disparate uses for arrow displays create the potential for confusion by drivers.

This paper describes the results of four focus group interviews with Midwestern drivers. Participants were shown several mock images of shadow work vehicles with arrow displays and were questioned on how well they understood and/or interpreted the message conveyed by arrow displays, depending on the display type and quantity of displays used, and the researchers looked specifically for potential driver confusion.

Focus group participants generally considered panel displays that included motion (e.g., sequential arrows and sequential chevrons) as implying a more important situation and preferred their use over flashing versions (flashing arrows and flashing chevrons). While participants were receptive to the use of multiple arrow displays on multiple shadow vehicles, a minority indicated that this conveyed a need to move over more than one lane. Participants also indicated that staggering sequential shadow vehicles from the shoulder into the closed lane provided useful information as to the number and location of the closed lanes.

Key Words: arrow display—focus group—traffic control—work zone

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INTRODUCTION

In work zones where the traffic control plans are relatively static, such as at long-term work zones, the use of a single advance warning arrow display, as shown in Figures 1 and 2, to indicate a single closure appears well-standardized and well-understood by the arriving public. In fact, the Federal Highway Administration (FHWA) has interpreted the Manual on Uniform Traffic Control Devices (MUTCD) as meaning that only one arrow display is to be used for each lane closed in long-term situations (FHWA 2003). However, in work zones that are moving or of a very short duration, there is often a desire by highway departments and contractors to use multiple arrow displays to indicate that a single lane is closed. Because these types of work zones often have fewer visual reinforcements to the lane closure message (e.g., no cones, barrels, or other channelizing devices), this desire seems understandable.

In short-term applications or for moving/mobile work convoys, the MUTCD does allow the use of multiple arrow displays to indicate a single lane closure (FHWA 2004). This results in a situation where the same traffic control device (e.g., an arrow display) is used in a slightly different manner depending on the nature of the work zone: in a long-term work zone, two arrow displays mean two lanes are closed, but in a short-term, mobile, or moving work zone, two arrow displays may mean only a single lane is closed. These disparate uses for arrow displays create the potential for confusion by drivers. This research was conducted to determine the extent to which typical drivers are able to understand these uses of arrow displays, to explore any confusion resulting from these different uses, and to make suggestions on how to improve lane changing information to drivers in short-term, mobile, and moving work zones.

Figure 1. Typical advance warning arrow display at a long-term work zone (arrow displayed)

Figure 2. Typical advance warning arrow display at a long-term work zone (chevron displayed)

METHODOLOGY

Focus group meetings were conducted in order to gain a better understanding of the views and opinions of the driving public as to how well they understand the information conveyed from the various displays on an arrow panel, which methods were preferred, and what alternatives might be useful for the driving public. Focus groups have advantages over other survey methods in that they are able to cover a topic in more depth and, due to the open-ended nature of the discussions, the potential exists for innovative concepts to be suggested by participants (University of Texas 2007). Areas of emphasis that were discussed during the focus groups included the following:

- What the various arrow displays mean to the driving public
- How participants interpreted the difference between a single arrow display and multiple arrow displays
- What the various caution displays mean to the driving public
- The driving actions that participants believe they would take when confronted with various arrow panel displays

Additionally, participants were asked what they would prefer to see changed with respect to advance work zone traffic control applications in short-term, mobile, and moving freeway operations.

Four focus groups were conducted in four cities in three Midwestern states in order to provide a diverse group of participants. Focus groups were conducted in the following locations:

- Kansas City, Missouri
- Lawrence, Kansas
- Overland Park, Kansas
- West Des Moines, Iowa

Participant Demographics

Focus group participants were recruited with the goal of having a diverse population of licensed drivers. Requirements for individual participants included having a valid driver license, driving at least 8,000 miles per year, and driving on a freeway at least once per month. Additionally, it was desired to match as closely as possible the demographics of the overall driving population with respect to gender, age, and level of education. Table 1 shows the ideal demographic percentages obtained from the U.S. Census Bureau and U.S. Department of Transportation, FHWA, for the population education level and the age of licensed drivers in the three states where focus groups were held (FHWA 2007; U.S. Census Bureau 2007). For the demographic category of age, all of the percentages were calculated based on state licensed drivers instead of the total state population.

Table 2 shows the actual demographic percentages of the 39 participants that took part in the focus groups. The targeted demographic distribution and the overall distribution of actual participants compared reasonably well, although an exact match between them was not achieved due to last-minute cancellations by some participants. Additionally, finding some demographic groups, such as participants with less than a high school education and participants for the oldest demographic group, was problematic. Overall, the actual demographic results indicated that a good cross section of the driving public was achieved; this is an encouraging indication that the comments provided can be considered representative for the areas where the focus groups were conducted.

Focus Group Study Design

Each focus group consisted of five main parts. The first part served as an orientation, where the research team explained the research goals of the project, how the focus group would be conducted, and an explanation of any questions that the participants had. The second, third, and fourth parts of the discussions consisted of explorations of participants' opinions and understanding of the following:

- Individual arrow displays when mounted on a single work vehicle
- Multiple arrow displays when mounted on several work vehicles
- Individual caution displays when mounted on a single work vehicle

The final part of the focus groups consisted of an open-ended discussion about what participants thought could be a way to change the advance warning area traffic control layout for short-term, mobile, and moving work zones.

Table 1. Targeted focus group participant demographics by location

		Kansas City, Missouri	Lawrence, Kansas	Overland Park, Kansas	West Des Moines, Iowa	Average
Gender	Male	49%	49%	49%	49%	**49%**
	Female	51	51	51	51	**51**
	Total	100	100	100	100	**100**
Age	Under 25	15	16	16	15	**15**
	25 – 39	26	26	26	24	**25**
	40 – 64	44	42	42	44	**43**
	Above 65	15	16	16	17	**16**
	Total	100	100	100	100	**100**
Education Level	No High School Degree	15	12	12	11	**13**
	High School	34	30	30	36	**32**
	Some College	29	32	32	31	**31**
	College Degree	22	26	26	22	**24**
	Total	100	100	100	100	**100**

Source: U.S. Census Bureau and the Federal Highway Administration.

Table 2. Actual focus group participant demographics by location

		Kansas City, Missouri	Lawrence, Kansas	Overland Park, Kansas	West Des Moines, Iowa	Average
Gender	Male	43%	43%	44%	45%	**44%**
	Female	57	57	56	55	**56**
	Total	100	100	100	100	**100**
Age	Under 25	14	28	33	0	**19**
	25 – 39	43	43	45	64	**49**
	40 – 64	43	29	11	18	**25**
	Above 65	0	0	11	18	**7**
	Total	100	100	100	100	**100**
Education Level	No High School Degree	0	0	0	18	**2**
	High School	14	0	33	0	**14**
	Some College	14	57	22	27	**30**
	College Degree	72	45	45	55	**54**
	Total	100	100	100	100	**100**

The arrow displays presented as part of this research are shown in Figure 3. Each arrow display was presented within a conceptualized image of a rural freeway scene showing one or more work vehicles meant to represent the advance warning shadow vehicles for a short-term, mobile, or moving work convoy equipped with arrow displays. The images were intentionally conceptualized to prevent participants from seeing extraneous levels of detail that would inevitably occur if real photographs had been used. The images were intended to show the vehicles just upstream from the crest of a hill, and this was explained at length to the participants; this meant that participants had to rely solely on the advance warning to know what lay ahead. Participants were told that they were approaching the work vehicle(s) from the right lane of a six-lane freeway. The images were displayed to the participants on a projection screen for several minutes while discussions about the specific arrow display(s) took place. The arrows and chevrons in the images flashed and moved sequentially as appropriate when presented to the participants.

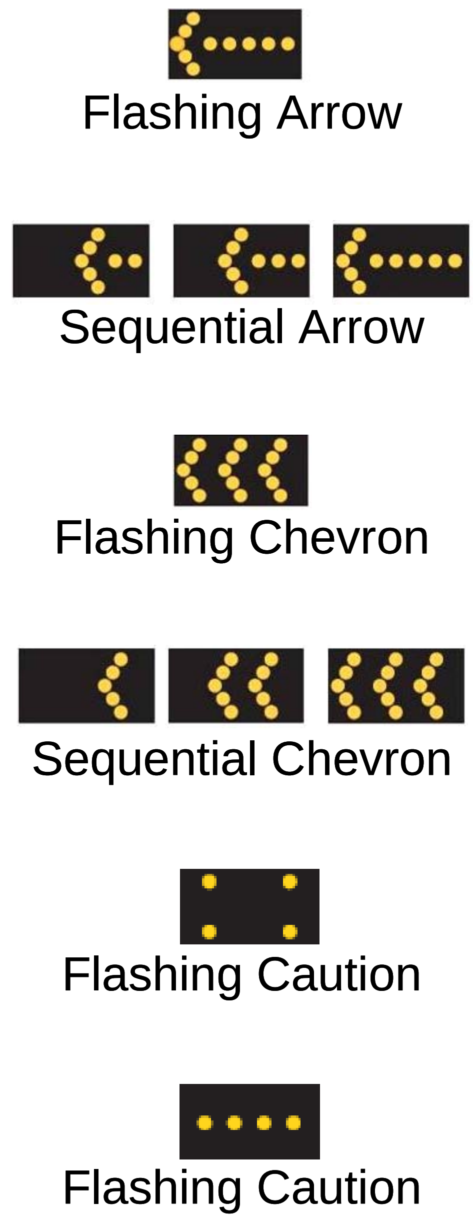


Figure 3. Advance warning arrow and caution displays presented to focus groups

KEY FINDINGS

Individual Arrow Displays when Mounted on a Single Work Vehicle

Figure 4(a) shows the first image shown to the participants: a representation of a shadow vehicle equipped with a flashing arrow display and moving slowly on the shoulder of a rural freeway. Participants were asked what they thought was happening and what they were being told to do by the traffic control devices. The same questions were asked of participants when the same image was shown with a sequential arrow (Figure 4(b)), flashing chevron (Figure 4(c)), and sequential chevron arrow (Figure 4(d)) displayed instead. These cases were designed to examine whether the driving public would associate the specific image on the arrow display with a closed lane, as well as the participants' preference.

Participants' Understanding of Single Arrow Displays

Participants generally understood that they were being directed to move over when shown the image in Figure 4(a). This did not change when participants were shown subsequent images of the sequential arrow (Figure 4(b)), flashing chevron (Figure 4(c)), and sequential chevron displays (Figure 4(d)). However, when asked specifically how many lanes, participants did not universally understand that they were to move over one lane. A minority of participants in each focus group indicated that they were being told to move all the way to the left.

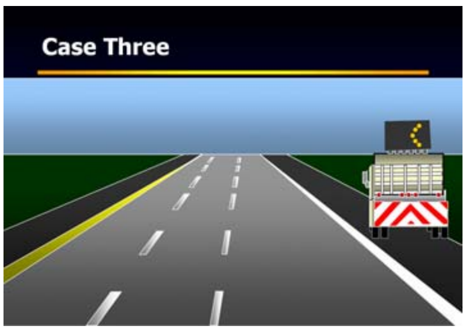
Some participants indicated that, even through they were being told to move over one lane, they were not likely to comply for varying reasons.

- A few participants indicated that they would move all the way to the left lane, not because of the arrow display, but because they were uncomfortable driving near a large work vehicle.
- A small minority of participants in each focus group also admitted that they would not move from the right lane regardless of the arrow display unless they were presented with additional information or were forced to do so.

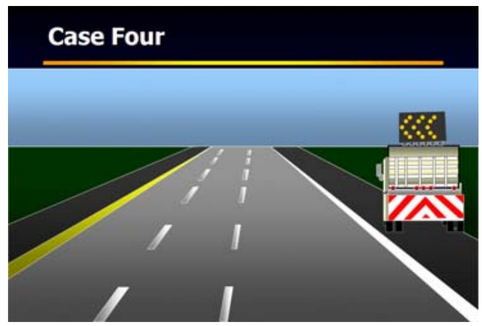
Interestingly, in all four focus group locations participants indicated that the sequential displays seemed to indicate a more important or critical situation due to the movement within the display.

(a) Flashing arrow

(b) Sequential arrow
(first of three arrows shown)



(c) Sequential chevron
(first of three chevrons shown)



(d) Flashing chevron

Figure 4. Focus group images showing single work vehicles equipped with arrow display

Comparison of Sequential Chevron Display with Curve Warning Chevrons

In two of the focus group locations, participants pointed out that the sequential chevron display (Figure 4(c)) seemed to be indicating that there was a sharp curve just over the hill rather than a lane closure. This potential discrepancy in meaning is a point of concern for the use of chevron displays compared to arrow displays. Participants did not have this reaction from the flashing chevron display or either of the arrow-type displays.

Participants' Preference of Arrow Display Based on Perceived Effectiveness

Participants were also asked to rate which arrow display they preferred to see based on which one they believed was most effective in conveying a lane closure. The participant preferences included the following:

- Sequential arrow (49% considered this display the most effective)
- Sequential chevron (28%)
- Flashing arrow (19%)
- Flashing chevron (2%)

As noted above, participants were more likely to rate sequential displays as more effective than their flashing counterparts.

Multiple Arrow Displays when Mounted on Several Work Vehicles

Participants were shown several images of two or three work vehicles in various configurations. Each of these images is shown in Figure 5. These images were shown one at a time to participants for several minutes, starting with the image shown in Figure 5(a) and ending with Figure 5(e).

Generally, participants agreed that they were approaching a larger and/or more extensive work zone operation when two work shadow vehicles could be seen instead of one. At each of the four focus group locations, participants noted that for the images shown in Figure 4 they thought it possible that the entire work zone might consist of just the one truck that was visible and could be a single worker collecting trash from the roadside, for example; the presence of two trucks meant that this was not the case. This finding could mean that when a convoy crosses the crest of a vertical curve there may be some advantage in leaving more than a single vehicle behind to alert traffic that they are approaching a work zone. Other findings are discussed in the following subsections.

Participants' Understanding of Multiple Arrow Displays

When shown Figures 5(a) and 5(b), the majority of participants indicated that they would move over at least one lane.

- When asked whether they were being asked to move over one lane or two, most indicated they were being directed to move one lane; a minority indicated two lanes.
- Among those who believed that one lane was being closed, some also commented that they thought this would be an ineffective way of closing two or more lanes.
- As with the images showing a single shadow vehicle, a small minority of participants indicated that they desired to move over two lanes regardless of what was being shown because they disliked driving next to large work vehicles.
- A small portion of participants misinterpreted this to indicate multiple lanes were closed.

These responses remained consistent when shown all of the images shown in Figure 5 for all four focus groups. This indicates that most motorists are likely to correctly interpret multiple arrow displays in a moving convey as indicating that one lane is closed.

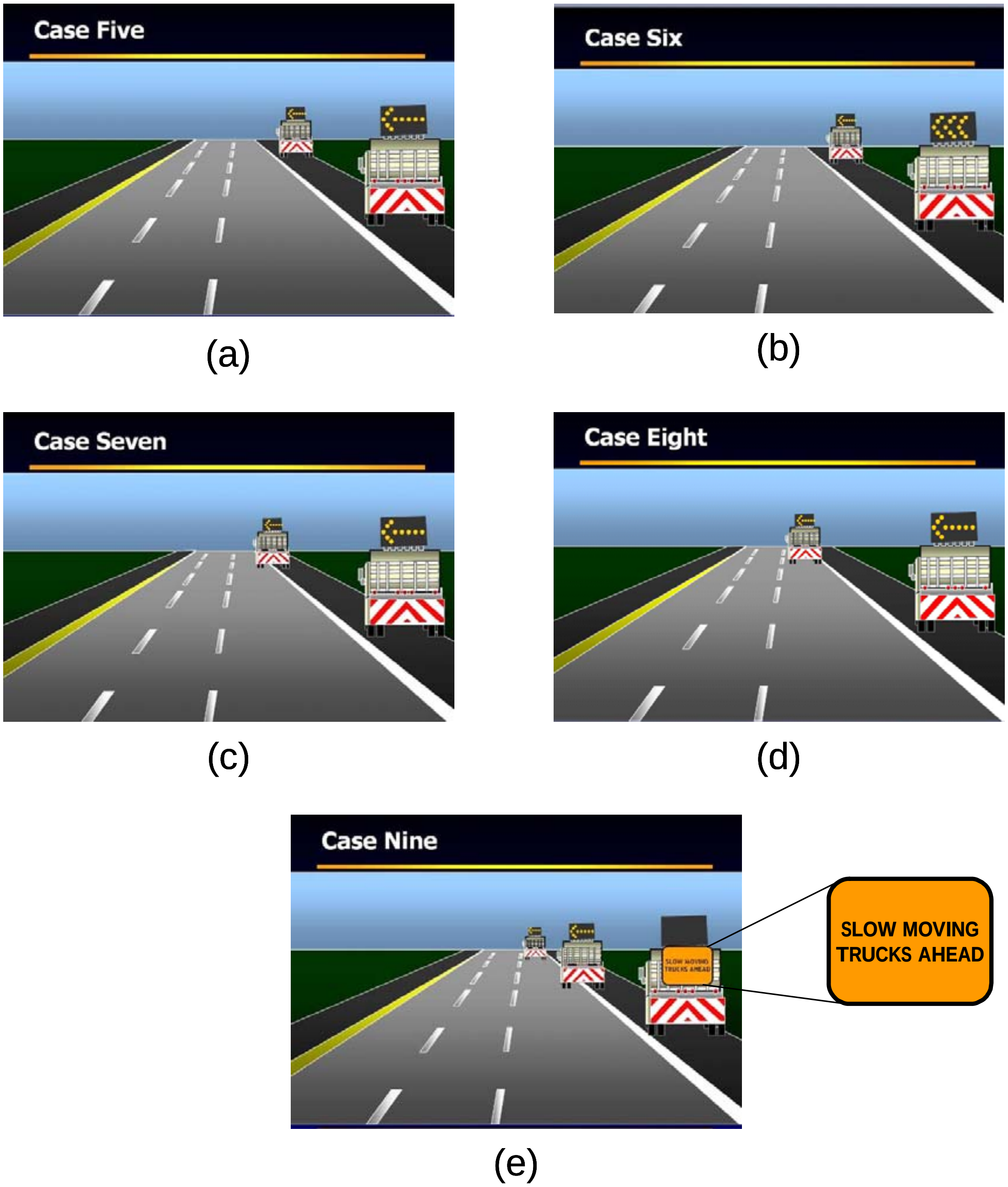


Figure 5. Focus group images showing multiple work vehicles equipped with arrow displays

A small minority of participants in each focus group indicated that they would not vacate the right-hand lane, as they would prefer to drive over the crest of the vertical curve and see for themselves that the lane is actually closed. As expected, this was not an issue for the images shown as Figures 5(c), 5(d), and 5(e). Indeed, participants generally approved of this staggered approach, indicating that this gave positive information regarding the lane closure and would get compliance from even aggressive drivers that would have preferred to remain in the right-hand lane.

Participants' Opinions on Mixing of Multiple Arrow Displays

When shown Figure 5(b), participants were universal in their dislike of displaying different arrow types on two vehicles.

- Example comments by participants were that if they saw this on a real highway they might be inclined to wonder if these two vehicles were part of the same operation or if they were two unassociated individual vehicles that just happened to be in the same vicinity.
- Many participants also commented that at a minimum it appeared that the two truck operators were "not on the same page," and that this reduced the credibility of the message they were conveying to drivers.

These are important statements because in short-term, mobile, and moving work zones the limited traffic control provided by the shadow vehicles is often the only effective safety device protecting workers. If the credibility of this message is degraded, it could correlate to an increased safety risk to workers. While presenting multiple arrow types in a single work area is not disallowed by the MUTCD, many individual states have policies stating that only one arrow type be used statewide or that only one type be used for any given work zone.

Static Signing on Work Vehicles

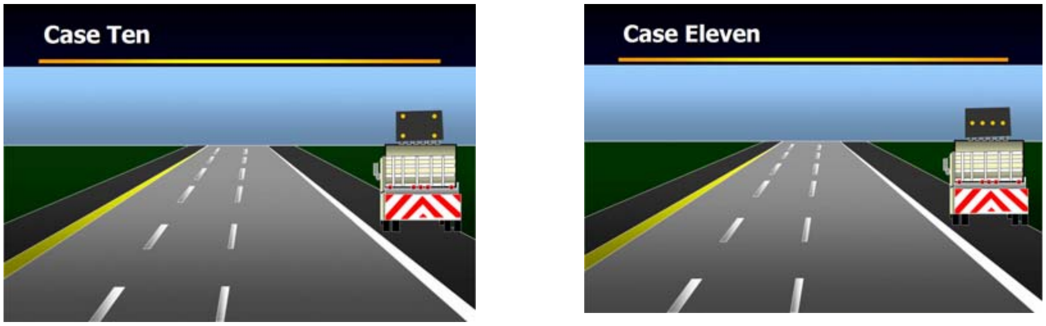
One final issue discussed in this section of the focus groups was the sample text sign shown in Figure 5(e). Participants were generally in favor of having additional information about the work zone they were approaching. When asked what information would be appropriate to show on such a sign, participants were divided in their opinions at each of the focus group locations.

- One group of participants wanted to know what activity was taking place. These participants were interested in what was going on with the work: were they approaching a painting operation that might spray paint on their vehicle? Were they approaching a work area where workers would be out of vehicles and near the traveled lanes?
- Another group of participants wanted to know what they were supposed to do. These participants were less interested in what was happening and stated a preference for positive directions, such as "MOVE OVER," "RIGHT LANE CLOSED," or "MOVE LEFT."

Individual Caution Displays when Mounted on a Single Work Vehicle

Participants were shown two images of single work vehicles with caution displays. These images are shown in Figure 6. These images were shown one at a time to participants for several minutes. Participants were unsure when asked what information the displays were conveying to drivers, and indeed initial reactions by several participants was that the first display shown (Figure 6(a)) did not mean anything. Upon further consideration, the participants began to believe that the message indicated that they would approach the situation with caution, just as if the vehicle were sitting on the shoulder with hazard lights or a flashing amber light. While this is the correct answer, the participants admitted to being less confident in their answers than in earlier sections of the focus group. When told that these were caution displays, comments were given by participants, included the following:

- If the work vehicle has a flashing amber light, is the caution display even necessary?
- There was the possibility that the flashing caution display shown in Figure 6(b) could be confused with an arrow display with several light bulbs burned out



(a) Flashing caution display (b) Flashing caution display

Figure 6. Focus group images showing single work vehicles equipped with caution displays

CONCLUSIONS

Several interesting findings were uncovered through the focus group meetings. These include the following:

- A few subjects indicated that the sequential chevron display looked more like a curve ahead warning than a lane closure warning.
- A large majority of participants indicated that they preferred sequential displays over flashing displays. Reasons given were that the sequential movement of these displays indicated a more important or critical situation compared to the flashing alternatives.
- The majority of focus group participants understood that multiple arrow displays in a work convoy indicate a single lane closure. Only a small minority of participants misunderstood the message to mean multiple lane closures.
- Participants were generally favorable of staggering work vehicles into the closed lane (as shown in Figures 5(c) through 5(e)), indicating that this provided positive reinforcement of the lane closure message.
- Participants were universal in their disapproval of mixing arrow displays within the same work convoy. The reaction of the participants to mixing arrow display types seems to reinforce state-level decisions in place to prevent this and indicates that national-level consideration of such a prohibition is needed.
- Participants liked the idea of including additional information in the form of static signing on the back of the shadow vehicles, but there was no agreement on the nature of the information. Some participants wanted to know more about the nature of the work being performed, while others were only interested in directions on what they were supposed to do.
- Caution displays were ultimately understood by participants, but participants were less confident in their responses than for other focus group questions, and there was some discussion about whether such displays were even needed if other devices such as flashing amber lights were present.

Each of these findings represents avenues for additional field research into improving the use and understanding of arrow displays. Improved driver understanding of these devices could result in a beneficial improvement in worker safety and is worthy of additional effort.

ACKNOWLEDGMENTS

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A Clearly Defined Methodology for Analyzing the Crash History for a Given Location or Corridor

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ABSTRACT

Good crash analysis is not a simple process. In order to adequately determine if a correctible problem exists, the analyst may need to follow many steps, make multiple calculations, recognize specific patterns, and compare multiple rates and ratios. These steps can seem overwhelming to most analysts, and many of the steps may be unnecessary, depending on the level of detail needed and the intermediate findings. With the increasing availability of crash data to engineers and technicians, there has followed an increased need to define a sound crash analysis procedure. Most practitioners who are new to crash data analysis are eager to include safety in their projects and decisions, but they have many questions and would benefit greatly from clear guidance. Multiple sources are available that cover several aspects of crash analysis; however, they have not been organized and condensed into a practical methodology for the majority of crash data users. The purpose of this paper is to clearly define a methodology for investigating the crash history for a corridor or spot location. This methodology will follow a logical sequence and provide guidance regarding the level of detail needed to determine whether the crash history indicates the potential to reduce the risk of future crashes at a given location. Some of the topics and guidance that will be defined include the number of years of crash history required, length of segment considerations, use of frequencies, rates and comparables, consideration of animal-related crashes, and some of the intricacies involved with freeway and grade-separated interchange analysis. The result will be a flexible, sequential methodology for crash analysis that will not be dependent on any particular crash data software or lookup tool. The flexibility will allow the analyst to focus on areas of concern and skip unnecessary steps. The methodology will lead the analyst through the process to a statistically sound result that can be used in decision making and resource allocation. The ultimate result will be a product that enables more engineers, technicians, and analysts to efficiently and effectively use crash data as a factor to improve roadway safety.

Key words: crash analysis—crash reduction—safety

Developing Guidance for Use of Lighting on Rural Roadways

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ABSTRACT

Nighttime driving has proven to be particularly challenging. For example, the U.S. Department of Transportation reports that 45% of all fatalities occur during dark conditions. Another study shows that the nighttime fatality rate is three times the daytime fatality rate. Crashes during dark conditions are more common than during light conditions. In an effort to limit the dark conditions at intersections, roadway lighting may be installed. This strategy can be effective, as long as the lighting levels and configuration allow the light to be useful.

Overhead lighting is often requested by the public whether or not it is warranted. While lighting at rural intersections has been shown to provide a positive safety benefit, it can be costly for rural agencies to install and maintain. As a result, in some cases agencies install the lowest cost configuration, such as a single light on a utility pole some distance from the intersection. Less-than-optimum lighting configurations may have little or no safety or lighting benefit because they may not illuminate the areas of the intersection where lighting is beneficial.

This paper details the development of guidelines to assist agencies in Iowa in deciding when good lighting is the appropriate countermeasure to address safety concerns at rural isolated intersections. The guidelines define what "good" lighting is; provide examples of common lighting configurations used in Iowa, with an assessment as to whether they provide adequate lighting using TarVIP (an overhead luminaire configuration program); and present other nonlighting countermeasures to rural intersection crashes.

Key words: lighting configuration—roadway lighting—TarVIP

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Field and Laboratory Study of the Mn/DOT Precast Slab Span System

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ABSTRACT

The Minnesota Department of Transportation (Mn/DOT) Precast Slab Span System was initially designed by Mn/DOT with input from University of Minnesota researchers and local fabricators. The bridge system consisted of a series of precast, prestressed concrete inverted tee bridge elements which also served as stay-in-place formwork for the cast-in-place portion of the deck placed in the field. One of the Mn/DOT implementations, located in Center City, MN, was instrumented. The bridge has been monitored for reflective cracking and continuity over the piers since the deck was cast. Transverse load distribution was evaluated with a static truck test. In addition, a two-span test specimen was constructed to investigate effects of variations in flange thickness, bursting reinforcement, horizontal shear reinforcement, and flange surface treatment. The data obtained from the field study indicated that cracking had initiated in the bridge at the locations of some of the gages at midspan and near the support. The cracking was determined to be the result of environmental loads and shrinkage rather than due to vehicular loads. The data from the truck tests indicated that the design assumption of a monolithic slab system was valid for the determination of load distribution factors. The results of the laboratory study showed that positive restraint moments developed in the precast system for which continuity was made at a young age (i.e., seven days), and that these moments could be reasonably well predicted by existing models. It has also been found that current American Association of State Highway and Transportation Officials bursting requirements require unnecessary transverse reinforcement in the end zones of slab span systems.

Key words: bursting reinforcement—composite—precast—restraint moment—slab span

Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

PROBLEM STATEMENT

The need to accommodate increasing volumes of traffic while replacing the aging infrastructure has resulted in a need to implement new construction techniques. Quality control, safety issues, and environmental concerns associated with onsite concrete casting, have prompted wider acceptance for the use of precast elements. Rural areas, where many short to intermediate span bridges are located, have additional motivation for precast construction due to limited specialty contractors for post-tensioning and formwork. A team of engineers who participated in a 2004 Federal Highway Administration International Scanning Tour of Prefabricated Bridge Elements and Systems identified a number of systems considered for implementation. A variation of the precast Poutre Dalle slab span system, originally developed in France, has been implemented by the Minnesota Department of Transportation (Mn/DOT). The goal of this research was to better understand the performance of the system to improve design guidelines and develop standard details.

RESEARCH OBJECTIVES AND METHODOLOGY

The Mn/DOT Precast Slab Span System was initially designed by Mn/DOT with input from the University of Minnesota (U of MN) researchers and local fabricators. The bridge system consisted of a series of precast prestressed concrete inverted tee bridge elements which also served as stay-in-place formwork for the cast-in-place (CIP) portion of the deck placed in the field. A typical cross section is shown in Figure 1, where precast depths of 12 to16 in. have been used for spans ranging from 22 to 45 ft. One of the Mn/DOT implementations located in Center City, MN was instrumented by the U of MN researchers. The bridge has been monitored for reflective cracking and continuity over the piers since the deck was cast September 19, 2005. Transverse load distribution was evaluated with a static truck test.

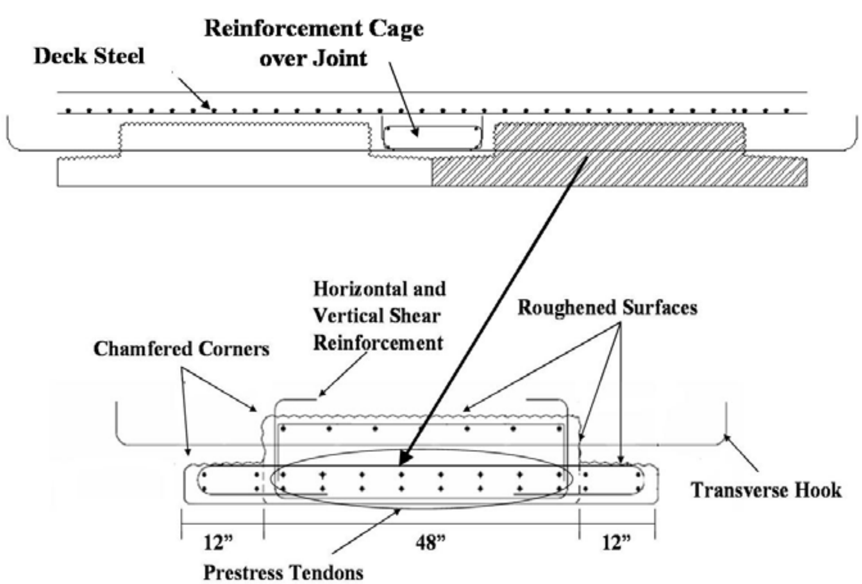

Figure 1. Conceptual cross section of Mn/DOT Precast Slab Span System

In addition, a two-span test specimen was constructed in the U of MN Structures Laboratory to investigate effects of variations in flange thickness, bursting reinforcement, horizontal shear reinforcement, and flange surface treatment. Each span employed two precast panels that incorporated combinations of these parameters. One of the four precast sections was identical to those used in the Center City Bridge. The precast inverted tee beams were cast September 6, 2006, and the CIP deck was cast seven days later to maximize the positive restraint moment that develops in precast systems made continuous by a CIP deck. The restraint moment was monitored for 250 days after the continuity pour.

Implementation considerations for the Mn/DOT Precast Slab Span System included the potential for development of reflective cracking that may occur due to stress concentrations between flanges of adjacent precast sections and over web corners of individual precast sections, and design issues such as restraint moments, bursting reinforcement requirements, continuity over the pier and load distribution factors.

RESTRAINT MOMENT

When bridge systems are made continuous with a CIP deck, restraint moments need to be considered which develop as a result of variations in time-dependent effects between the precast and CIP elements. As part of the laboratory study, the two-span test bridge was monitored for restraint moment development. These results were compared to the original design calculations and methods found in the literature (Freyermuth 1969; Peterman and Ramirez 1998). The design considerations of NCHRP 519, which have been incorporated in American Association of State Highway and Transportation Officials (AASHTO) LRFD 2007, were also investigated.

Methodology of Restraint Moment Study

As mentioned above, the CIP deck was cast on relatively young (seven-day-old) precast inverted tees to maximize the effects of the positive restraint moments over the pier. Because younger precast beams will have increasingly similar shrinkage development as the CIP deck, the creep due to prestress dominates the behavior and greater positive moments are developed. The differential age of seven days was chosen as a reasonable minimum for practical construction limitations. Positive restraint moments negate to some degree the benefit of continuous bridge construction as the positive midspan moment is increased. These moments are difficult to address in design because positive moment connections with capacities beyond $1.2M_{cr}$, where M_{cr} is the cracking moment of the diaphragm, are not efficient as they may increase the positive restraint moment by attracting more moment. Consequently, it is recommended that steps be taken to reduce the positive moment if necessary (Miller et al. 2004).

The development of restraint moments was monitored by placing load cells under the supports of the ends of the laboratory specimen. Equilibrium was used to calculate the corresponding restraint moments using the center of bearing length shown in Figure 2.

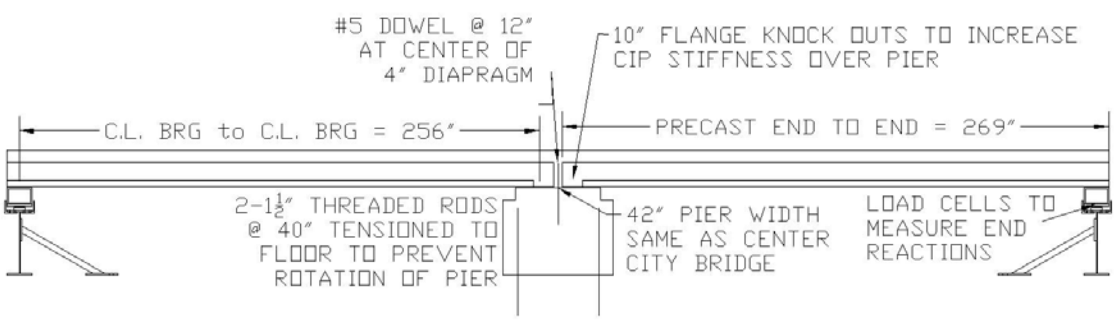

Figure 2. Elevation of laboratory specimen

Key Findings

Results from the laboratory study were compared to the Portland Cement Association (PCA) Method (Freyermuth 1969) and the P-Method (Peterman and Ramirez 1998). The primary differences between these methods were the modeling assumptions at the pier and the values assumed for creep and shrinkage. In the P-Method, the diaphragm is modeled as a small span between the bearing supports of adjacent beams, whereas the PCA Method uses a single support at the center of the pier. For the Center City Bridge, the P-Method predicted restraint moments 14% larger than the PCA Method due to the modeling assumption at the piers. For creep and shrinkage, the PCA Method provides charts from which values were scaled. The P-Method provides no creep and shrinkage values, so AASHTO LRFD (2004) Section 5.4.2.3 was used. The creep and shrinkage values from the PCA Method were much higher than those from the AASHTO LRFD. Using the laboratory specimen as an example, the values assumed for maximum differential shrinkage and the ultimate creep coefficient were of 135 $\mu\varepsilon$ and 2.66, respectively, for the PCA Method and 88 $\mu\varepsilon$ and 1.01, respectively, for the AASHTO LRFD. The change in strain at the center of gravity of the strands in the web of each of the four precast sections in the laboratory specimen has been monitored since the beams were cast and shows better agreement to the combined creep and shrinkage predicted by the models in AASHTHO LRFD (2004) than the PCA Method as shown in Figure 3. For the models, Young's modulus was calculated using AASHTO LRFD (2004) C5.4.2.4-1 using the precast and CIP 28 day strengths of 12.9 ksi and 4.3 ksi, respectively. Other required parameters included the age at prestress transfer (1 day), the cure time for the precast beams (1 day) and CIP deck (8 days), and the average measured relative humidity (40%).

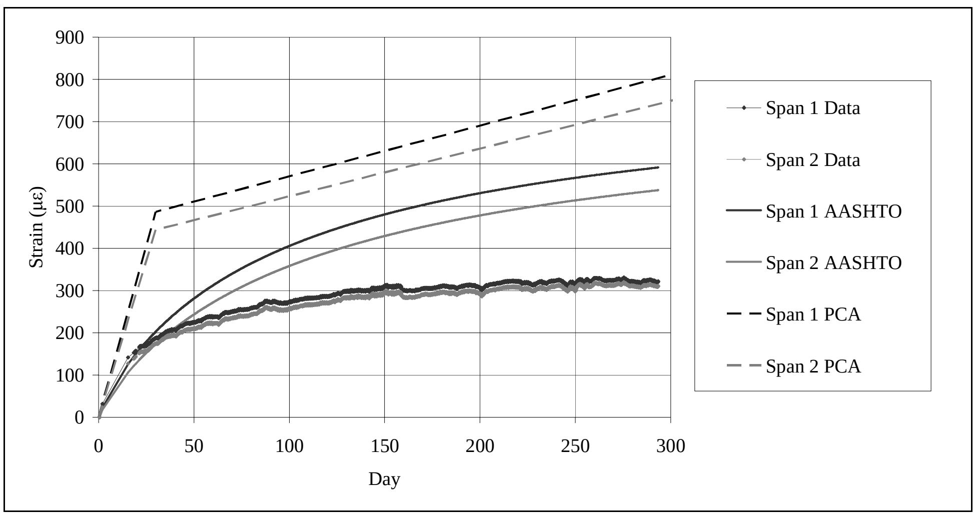

Figure 3. Combined creep and shrinkage strains from center of gravity of prestress strands

The two methods were also applied to two of the Mn/DOT Slab Span bridges, assuming continuity at 14 and 90 days, which predicted positive and negative restraint moments, respectively. In these cases, the PCA method was found to predict slightly smaller positive restraint moments and much larger negative restraint moments than the P-Method as shown in Table 1. The main reason for the difference in negative restraint moment between the methods was a factor to account for the shrinkage restraint due to the longitudinal stiffness of the deck steel and precast section that was used in the P-Method but not in the PCA Method. Creep of the precast and differential shrinkage predictions were both higher for the PCA Method for continuity at 14 days, but the resultant was similar to the P-Method prediction. For the Center City Bridge, the shrinkage restraint factor reduced free shrinkage by 64% and 70%, respectively, using design and measured strengths to calculate relative stiffnesses. Stiffnesses were calculated using AASHTO LRFD C5.4.2.4-1 where the design strengths were 6.5 and 4.0 ksi for the precast beams and

CIP deck, respectively, and the measured 28 day strengths from the laboratory specimen were 12.9 and 4.3ksi for the precast beams and CIP deck, respectively. The PCA method has been found to over-predict negative restraint moments in other studies as well (Peterman and Ramirez 1998; Miller et al. 2004).

Table 1. Predicted restraint moments for Mn/DOT Slab Span bridges (ft.-kips)

	Positive restraint moment (Continuity at 14 days) (Predicted at 20 years after continuity)		**Negative restraint moment (Continuity at 90 days) (Predicted at 100 days after continuity)**	
	PCA Method	**P-Method**	**PCA Method**	**P-Method**
Center City	78.5	106	198	48.4
Bridge 04002	142	149	266	101

The AASHTO LRFD (2004) Section 5.14.12.2.7c requirement for considering a fully effective continuous joint was found to be extremely restrictive for the Center City Bridge. This provision essentially required the positive restraint moment to be smaller than the sum of the superimposed dead load moment and one half the live load moment. Because the span lengths for the Center City Bridge were only 22, 27, and 22 ft., the applied moments were small, and the positive restraint moment was limited to only 71 ft-kips or $0.46M_{cr}$. This appeared too conservative, and it seems reasonable to limit the positive restraint moment to the cracking moment of the diaphragm. However, because the cracking moment may be over predicted due to insufficient bond at the ends of the precast sections, a reduction factor may be required.

Figure 4 shows the calculated restraint moments for the P-Method and PCA Method using measured material and geometrical parameters for each span of the laboratory specimen, with the creep and shrinkage models for the respective methods described above. The predictions obtained from the PCA Method were piecewise linear due to the limited data points for creep over time; whereas the P-Method results were obtained using a daily time step. Cross sections for both spans are given in Figure 5. The predicted strand stresses in the 16 web strands of each precast section immediately after transfer were used as the prestress force in the restraint moment calculations, and they were 194 and 195 ksi, respectively for Spans 1 and 2 using measured properties. The strands in the flange were neglected because they were only nominally stressed. A line at $0.6M_{cr}$, half the recommended limit from NCHRP 519, is shown for scale. The P-Method predicted different restraint moments for the two spans that reflect the design changes in the specimen (e.g., reduction in precast concrete area). There was not a similar change for the PCA method because the modeling assumption of a single roller support at the pier resulted in a single value for the restraint moment at the pier that combined the effects of the two spans. The data from the west span is missing from the 14th to the 161st day due to an error in monitoring setup. The daily fluctuations in the data were due to temperature changes which became more pronounced towards the end of the monitoring period when work related to other projects began requiring the doors to the structures laboratory to be opened.

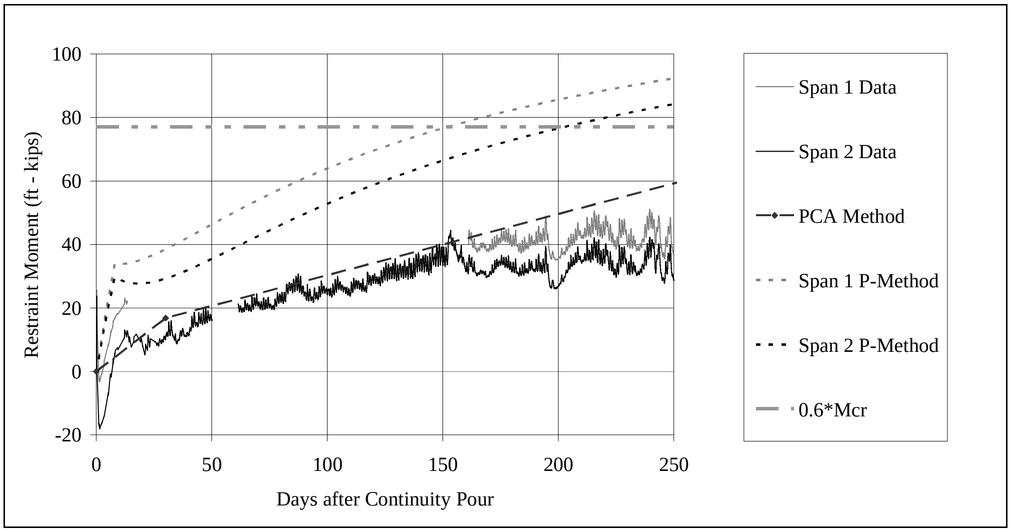

Figure 4. Restraint moments from laboratory specimen

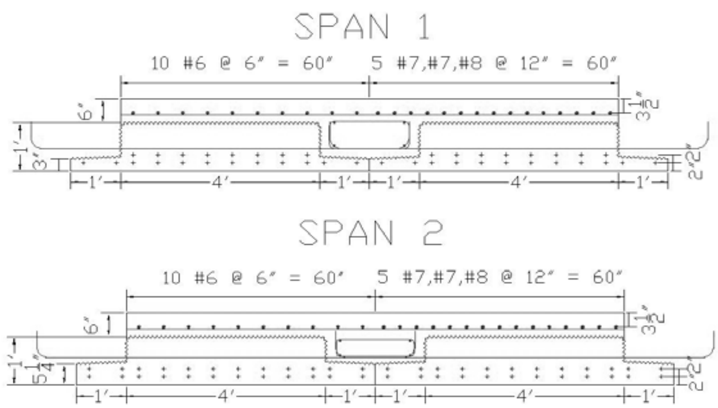

Figure 5. Laboratory specimen cross sections

The measured data were expected to be less than the predictions because the models assumed roller(s) at the pier where in reality the pier support should have behaved as a rotational spring as can be seen in the pier connection in Figure 2. The negative restraint moment developed over the first several days was likely due to the cooling of the CIP deck after casting which is not considered by either of the models, and this consideration would allow for an upward shift in the measured results to an unknown degree. Considering the high degree of uncertainty due to the variability of material properties and imperfect modeling assumptions, both methods provided reasonable estimates of the positive restraint moment developed over the first 250 days. But this was somewhat of a coincidence as competing differences within the models canceled out. For example, when the time of continuity was changed from 7 to 28 days, the P-Method predicted restraint moments of -3 and 63 ft-kips at 30 days and 20 years after continuity, respectively, whereas the PCA Method predicted -182 and 17 ft-kips, for the same time frames. In these cases, the differences in the predictions would greatly affect both positive and negative moment design at the pier. Although the PCA Method appeared to provide a better estimate of the restraint moment, the creep and shrinkage results from the laboratory specimen in Figure 3 show that the PCA Method over predicted the strains that drove positive restraint moment development. In contrast, the P-Method predictions had similar relations to the measured data in both Figures 3 and 4, which infers that the shrinkage restraint factor, and creep and shrinkage models assumed for the P-Method provided a better model of the behavior than the PCA Method. Without the large creep coefficient predicted from the PCA

Method, there would not have been enough positive restraint moment from prestress creep to counter the large negative restraint moment due to the assumption of unrestrained shrinkage.

Figure 6 shows the two models carried out to the 20-year service life of the bridge. It is unknown whether the positive restraint moment would have reached 120 ft-kips as it appeared to have tapered off closer to 40 ft-kips. This is consistent with the readings of the strain gages at the center of gravity of the strand shown in Figure 3, where the creep and shrinkage strains in the precast sections that drove positive moment development leveled off compared to either of the model predictions.

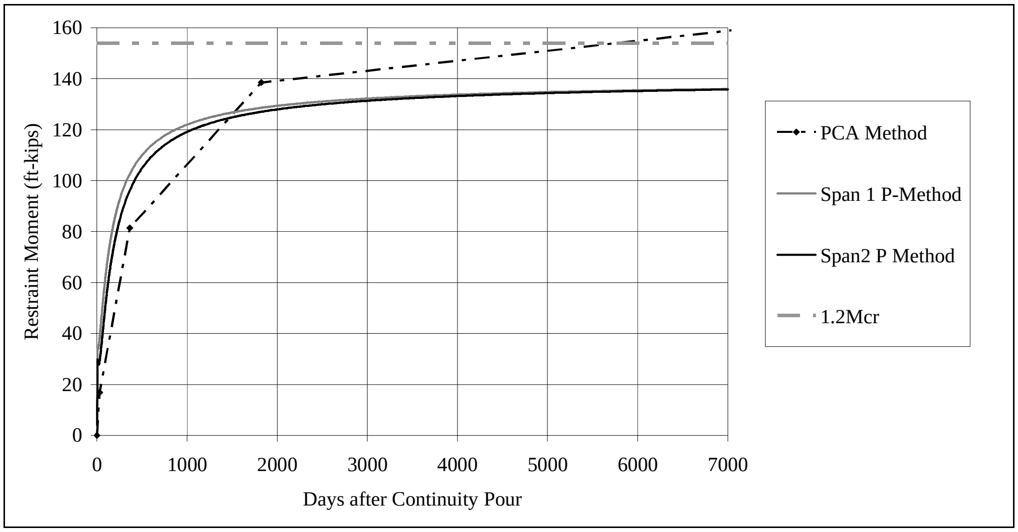

Figure 6. Laboratory specimen predictions carried out to 20 years

BURSTING

Horizontal cracks frequently form in the end zone of prestressed members when the prestress force is transferred to the concrete. These cracks, known as bursting cracks, result from vertical tension created by the transfer of forces from the prestressing strand to the concrete. If unrestrained, these cracks can extend into the member and decrease strength and durability. Previous studies have determined these cracks cannot be eliminated, but vertical reinforcing steel can limit crack widths and propagation (Fountain 1963). AASHTO's 1961 Interim Specifications introduced a minimum vertical reinforcement requirement for the end zones of pretensioned members, which has since remained almost unchanged. Meanwhile, pretensioning use has increased along with developments in cross section shape, higher strength materials, and increased strand sizes allowing for greater prestressing forces within members. Despite these advances, AASHTO has not made significant modifications to bursting steel requirements. The purpose of this investigation is to determine if the current AASHTO bursting requirements are applicable to the precast slab span system.

Development of AASHTO Bursting Requirements

The 1961 Interim AASHTO Specifications introduced bursting requirements for I-beams stating, "Vertical stirrups acting at 20,000 psi to resist 4% of the prestressing force shall be placed within *d*/4 from the end of the beam with the end stirrup placed as close to end of beam as practicable," where *d* is the effective depth. By 1969 the above requirements were made applicable to end zones of all prestress beams, not just I-beams. The only difference between the 1969 provisions and the current, 2007 AASHTO LRFD Bridge Design Specifications is that vertical steel can now be placed within *h*/4 from the end face, where h is the member height, instead of $d/4$. It is generally assumed the original AASHTO provisions were developed as a result of experimental testing performed on I-beams by Marshall and Mattock (1962). This suggests the original AASHTO requirements, developed for I-beam members, are not necessarily applicable to other cross sections, particularly slab span systems.

The current AASHTO bursting provisions can require a large amount of vertical steel to fit in a small area. For slab span systems and other sections small in height, fitting the required vertical steel within one quarter of the height from the end face causes congestion and complicates prestressing strand and concrete placement. For the laboratory slab span specimen, with a height of 12 in., all bursting steel was required to fit within 3 in. from the end. Allowing 2 in. for clear cover, only 1 in. was available for bursting steel placement, which was not feasible.

Marshall and Mattock (1962) used experimental tests on I-beams to develop a design equation for the area of bursting steel necessary to limit bursting crack width and propagation. The first series of tests were conducted on ten 22.5 in. tall I-beams with no vertical tension steel. Members varied by strand configuration, web thickness and strand surface condition. Maximum tensile strains were found to occur on the end face near mid-depth. The strains decreased quickly from the end face to reach zero at a distance no further than one-quarter of the height from the end face. The second series of tests were conducted on 25 I-beams of 22.5 and 25 in. heights with two sizes of vertical steel. These beams varied by strand size and location and magnitude of prestress force. Results indicated the total vertical reinforcement force was proportional to the prestressing force and inversely proportional to strand transfer length. Using this relationship and experimental data, a design equation was developed for the area of vertical steel needed to limit crack width and propagation.

$$A_s = 0.021 \frac{P}{f_s} \frac{h}{L_t}, \text{ if } \frac{h}{L_t} \leq 2 \qquad (1)$$

where T is the total prestress force (kip), h is the height of the section (in.), L_t is the transfer length (in.), and f_s is the allowable working stress of the transverse reinforcement (ksi).

Equation 1 was developed for members with non-uniformly spaced strands in the top and bottom of the member. The AASHTO requirements are equal to the above equation, if h/L_t is set equal to 2.

In an attempt to better quantify the location and magnitude of bursting forces, authors have used finite element models (Gens et al. 2005), equilibrium analyses (Gergely et al. 1963), and strut and tie models (Castrodale et al. 2002). In the Gergely and Sozen (1963) approach, equilibrium analysis was applied to the end zone of a section. In the model, cracking was assumed to occur along the end face at the height of maximum moment. The crack was conservatively assumed to extend a length equal to the height of the member. A free body diagram of the section below the crack is shown in Figure 7. To put the free body into equilibrium, a moment was applied at the top of the free body. This moment was used to determine the tensile force along the face of the member. Although originally developed for post-tensioned systems, this theory is also applicable for pretensioned systems. In a study performed at the University of Nebraska-Lincoln (Tadros et al. 2003), the Gergely-Sozen model was compared to experimental results from Marshall and Mattock's study on pretensioned I-beams. The Gergely-Sozen method was shown to provide a conservative estimate of vertical tensile force in nearly all cases, and ranged between 0.95 and 3.58 times the experimental result.

Table 2 provides a comparison of the different predicted tensile forces from AASHTO (2007), the Marshall and Mattock equation, and the Gergely-Sozen model. These are compared to the experimental results gathered from literature for an I-beam (Marshall et al. 1962), inverted tee (Tadros et al. 2003) and the laboratory specimen. Note the large difference between experimental values and AASHTO values. A 2003 study on pretensioned systems found that changing AASHTO specifications by decreasing the required tension force to be resisted by the reinforcement from 4% to 3% of the total pretension force would still be conservative (Tadros et al. 2003).

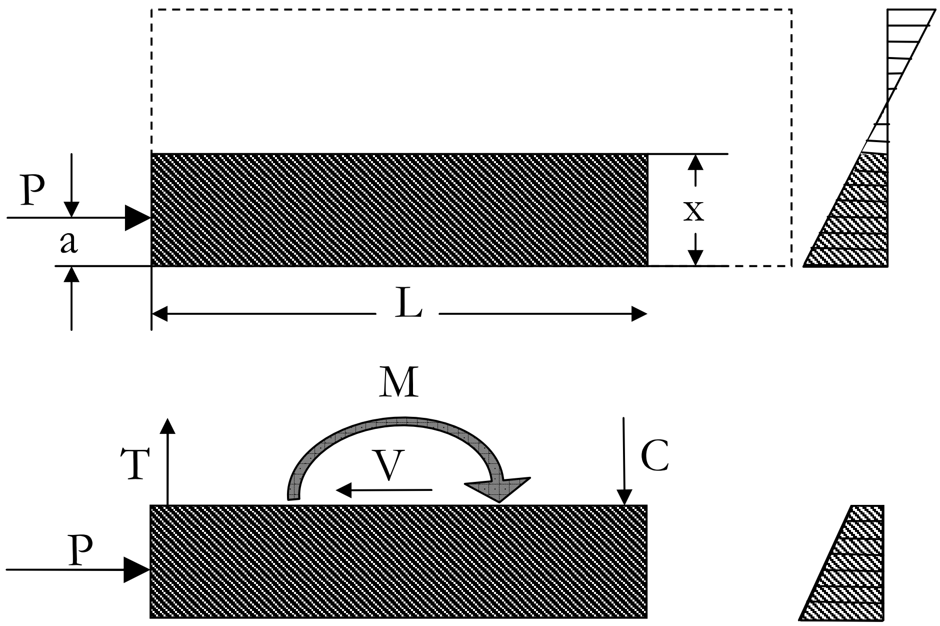

Figure 7. Gergely-Sozen Model (Gergely et al. 1963)

Table 2. Comparison of Predicted Tension Force to Experimental Tension Force Results (kips)

	AASHTO	Marshall and Mattock	Gergely - Sozen	Experimental
I-beam	11.6	12.2	11.2	5.2
Inverted Tee	19.8	9.8	20.8	12.2*
Laboratory Specimen	19.8	5.0	1.6	0.0

*Average vertical tensile force from 3 studies on identical inverted tee specimens. Results ranged from 10.3 kip to 14.5 kip (Tadros et al. 2003).

Methodology of Bursting Study

Both ends of each precast section for the laboratory specimen were instrumented with steel and concrete strain gages to measure strain in the section. Rosettes were used on the side face of each section, located at mid-height, two in. into the section. Two strain gages were attached to each vertical stirrup placed in the end zones. Four different vertical steel configurations were used for the four precast beams, as shown in Table 3. Two of the configurations used less than half of the area of steel required by AASHTO (2007), and the sections that met the required area of bursting reinforcement were not able to meet the placement requirements (i.e., extended further than three in. from the end into the section).

Table 3. Bursting Steel Configurations for Laboratory Specimen

	Slab 1		Slab 2		Slab 3		Slab 4		AASHTO
Stirrup Location (in. into beam)	2	4	2	4	2	4	2	4	
Vertical Stirrup Size	#3	#3	#4	--	2 - #5	2 - #5	#5	#5	
Total Stirrup Area (in.^{2})	0.44		0.40		2.48		1.24		0.99

Key Findings

In the laboratory specimen, the concrete and steel strain gages were monitored at transfer. The measured strains were negligible, and there were no visible signs of cracking. The concrete had adequate strength to resist tensile stresses induced at the time of transfer. Therefore, the quantity of vertical reinforcement in the end zone did not affect the results. Further investigation is being performed to develop recommended changes for bursting requirements.

FIELD BRIDGE STUDY

Methodology of Field Bridge Study

The Center City Bridge was monitored to investigate the effects of environmental and vehicular loads since the deck was cast September 19, 2005. The focus was to watch for the development of reflective cracking. In the center span, three joints were instrumented at midspan as shown in the inset of Figure 8 to monitor the transverse strains over the joint between the precast sections, over the web corners, and in the mild reinforcement that crossed above the joint between the precast sections. In addition, gages were placed on longitudinal mild reinforcement as shown in Figure 8. Load distribution, continuity over the piers, and effects of live load on reflective cracking were investigated by performing static truck tests April 18, 2007. Results were compared to a simple finite element model.

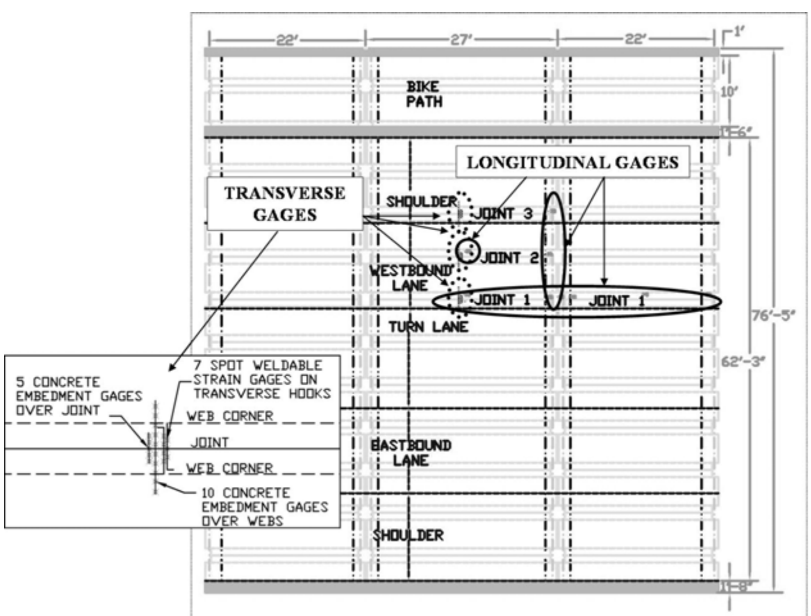

Figure 8. Layout of Center City Bridge

Key Findings

Results from monitoring the bridge for 21 months showed satisfactory behavior. There were, however, sufficient strain readings in two of the three monitored joints and over the pier that indicated some cracking had occurred. Daily strain fluctuations of 165 and 260 $\mu\varepsilon$ in the transverse direction at midspan of the middle span in Joints 1 and 3, respectively, were observed. The readings from the adjacent mild reinforcement fluctuated as well, indicating that it was effectively spanning the crack. Stresses in the mild steel reduced along the reinforcement away from the maximum over the joint between the precast sections indicating that the crack was centered on the joint. The absence of similar behavior in the embedment gages above the web corners indicated that the crack had not propagated to that level, although there were visible longitudinal shrinkage cracks on the deck that ran the entire length of the bridge. At the pier, strain readings indicated a positive moment crack. Daily strain changes of greater than 700 $\mu\varepsilon$ were recorded on the mild steel of the positive moment connection at mid-depth of the section. As with the transverse cracking, there was no unusual activity observed in the instrumentation directly above indicating that cracking had not propagated to the top of the CIP. The cracking appeared to be driven by the thermal gradient due to the top of the bridge absorbing solar radiation. This caused the bridge to camber, which generated positive restraint moments. All of the activity appeared to have begun abruptly on April 23, 2006. Because the pier developed a positive moment crack, it is unlikely that this was caused by a vehicular load, and, moreover, it is unlikely that a truck could have caused the crack at the pier as indicated by the small strains observed during the truck test. The cracking was most likely due to a combination of shrinkage and temperature effects that reached a critical level.

The truck test performed on April 18, 2007 confirmed that Joints 1 and 3 had reduced stiffness, likely due to cracking. A wheel load placed directly above the joint caused transverse strains of 19 and 32 $\mu\varepsilon$, respectively, in the concrete embedment gages immediately above Joints 1 and 3, compared to 8 $\mu\varepsilon$ in Joint 2, which had not indicated any cracking. These small strain readings also show how small the effects of vehicular loads were compared to environmental loads. A truck would require an axle weight of more than 60 tons to generate the same transverse strains that temperature effects caused on any given day. Reflective cracking over the web corners was also a concern for the system, but no load position caused a tensile stress in the embedment gages over any of the three monitored joints.

The truck test was also used to evaluate the load distribution factor used by Mn/DOT to design the Center City Bridge. Because none of the categories in AASHTO LRFD 4.6.6.2 directly addressed the Mn/DOT Precast Slab Span System, the equation for effective width of a monolithic concrete slab-type bridge given in AASHTO LRFD (2004) 4.6.2.3 was used for design. Due to the gap between flanges of adjacent precast sections, a reduction in transverse stiffness may cause the effective width to be smaller than that of a monolithic system. Load distribution was evaluated by loading the center span at midspan with a single truck at six locations across the width of the bridge. Midspan curvatures were calculated using gages welded to longitudinal reinforcement in Joints 1 and 2. Figure 9 shows the truck layout for one of these six positions. For the midspan tests, the truck faced west, but the front axle load was neglected since it was near the pier.

Curvatures were calculated by fitting a line through the strains from gages 9in, 12.5 in, and 15.5 in from the bottom of the 18 in. composite section. Results for the six tests are shown in Figure 10 along with the results of a finite element model of an isotropic flat plate with a smeared stiffness calculated by combining the stiffnesses of the precast sections, CIP deck, and mild deck reinforcement and assuming no cracking. The same stiffness calculations, based on the 28-day strengths of the laboratory specimen components, discussed in the restraint moment findings were used to calculate the smeared isotropic stiffness as it was assumed that the bridge components would have similar stiffnesses to that of the laboratory specimen since the precast sections were made with the same mix from the same precast plant and both deck mixes followed the Mn/DOT 3Y33 specification. The parapets were included in the model, but only had an effect on the results close to the parapet which allowed for the superposition of the data for the six tests onto a single plot. In the model, the boundary condition at the pier was assumed to be a single roller at the center of the pier. The midspan curvature of the center span corresponding to the same load assumed to be carried over the effective width from the AASHTO equation is also given.

The results generally followed the trend of the isotropic model, from which it can be observed that a design equation for a monolithic slab system is a valid assumption for this system. It can be assumed that longer spans would behave similarly because they will have a deeper overall section, and the height of the gap at the flange tips will be unchanged, so longer span bridges should behave more like monolithic systems than shorter ones. The isotropic model was also used to predict strains near the pier in the middle span where another series of gages was located when the bridge was loaded at midspan. Because the gages were within one depth of the bearing, the assumption of beam theory was not valid, but the strains in the deck steel were compared to those predicted in the isotropic model at the same depths as the gages. The results for both the positive and negative reinforcement are both given in Figure 11. The model predicted negligible strain for the positive moment steel because it was located at mid-depth of the 18 in. composite section (i.e. neutral axis in the model). Again, the results generally followed the trend of the isotropic model, except that the data appeared to be slightly less distributed, but the assumption of a monolithic system appeared to be reasonable. It can also be seen that the strains in the positive moment reinforcement at mid-depth were compressive, which corroborates the existence of positive moment cracking at the pier as the concrete below mid depth must have a reduced stiffness.

Continuity over the pier was also evaluated by analyzing the truck test data from instrumentation in Joint 1. When the center span was loaded at midspan, the midspan deck steel strain in the adjacent span due to negative bending was 1.2 $\mu\varepsilon$ for a single truck and 1.5 $\mu\varepsilon$ for two trucks, compared to 1.9 and 3.7 $\mu\varepsilon$ as predicted by the isotropic model. The loads in these tests where centered on Joint 1 to maximize the results. The discrepancies were most likely due to moment transferred into the substructure at the piers and abutments of the bridge where perfect rollers were assumed in the model. Because the midspan curvatures from the truck test were smaller than those predicted by the isotropic model, the assumption of full continuity appeared to be conservative so long as the benefit of the moment connections at the abutments and piers was ignored. The midspan curvature for the case without continuity at the pier was also plotted in Figure 10 to confirm that the continuity assumption was valid.

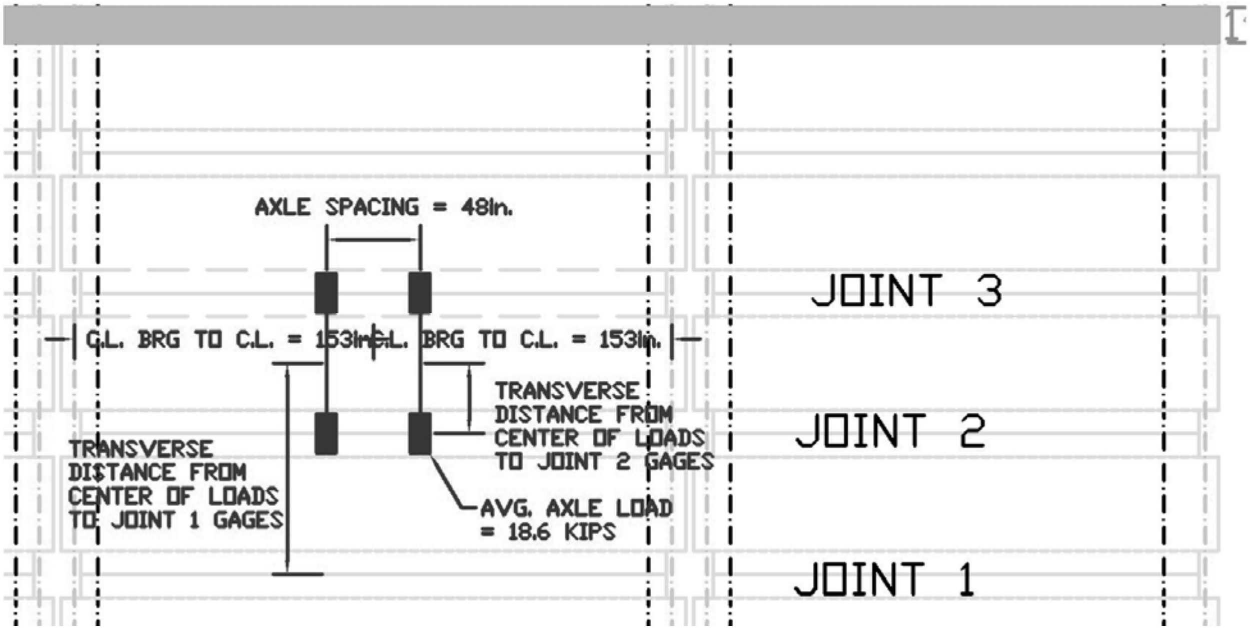

Figure 9. Typical truck test position layout

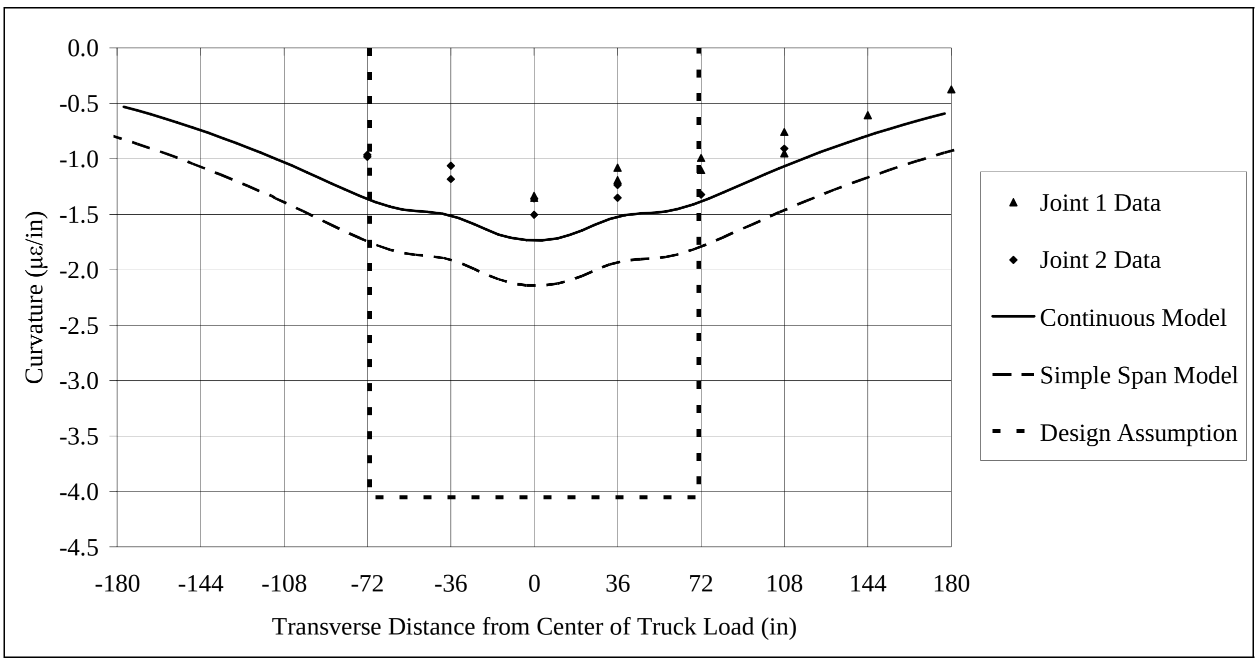

Figure 10. Curvatures at midspan due to a single truck

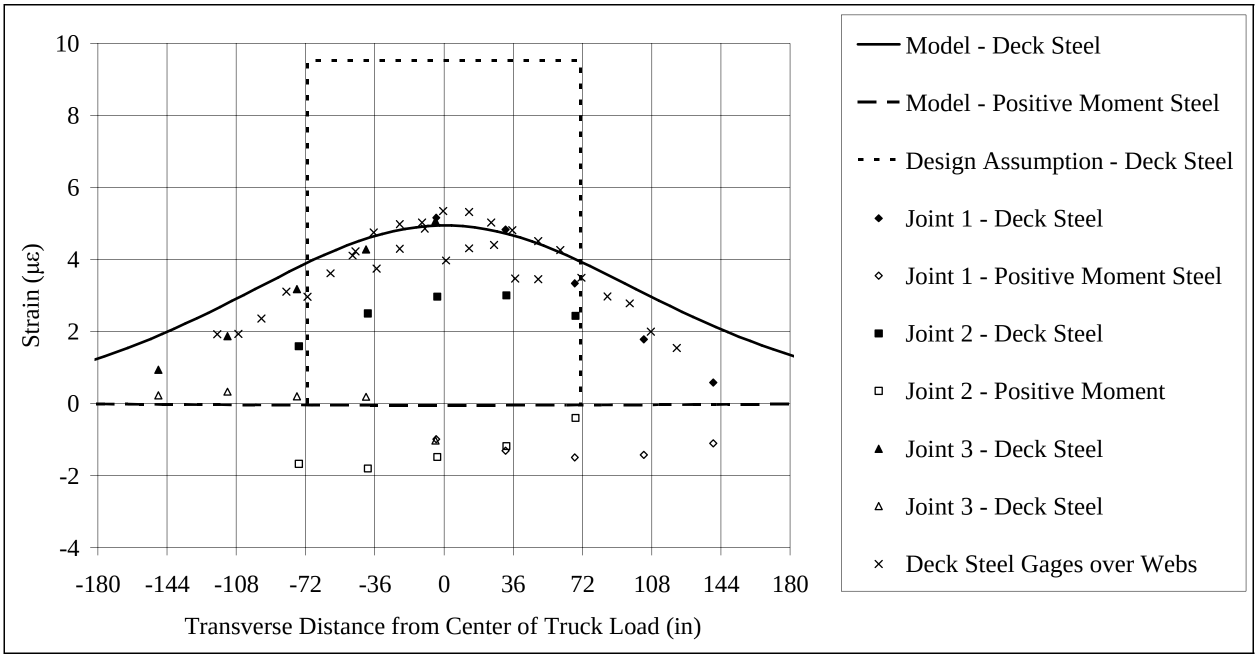

Figure 11. Strains near pier due to a single truck

CONCLUSIONS

The results of the laboratory study showed that both the PCA Method and P-Method provided a reasonable prediction of positive restraint moment development. The creep and shrinkage values were better predicted by AASHTO LRFD (2004) 5.4.2.3 than the charts provided with the PCA Method, and with the absence of the large creep strains predicted by PCA Method to offset the unrestrained shrinkage, the assumption of shrinkage restraint appears to be valid. It has also been found that current AASHTO bursting requirements do not take into account cross section shape and require unnecessary transverse reinforcement in end zones of slab span systems.

The results of the first 21 months of the field study showed that no reflective cracks have propagated above the web corners, and all of the strain fluctuations observed appeared to be initiated and driven by environmental loads and shrinkage. The results from the truck test showed that wheel loading immediately above the joint resulted in strains less than 20% of those observed from environmental loads, even in joints where cracking had been observed. The isotropic plate model reasonably predicted the load distribution of the Center City Bridge due to vehicular loads from which it can be inferred that the assumption of using a monolithic slab system to determine the load distribution factors for design was valid. The assumption of full continuity appeared to be valid so long as the benefit of the moment connections at the abutments and piers was ignored.

ACKNOWLEDGMENTS

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Cooperative Wetland Mitigation Clearinghouse

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ABSTRACT

The Iowa Department of Transportation (Iowa DOT) currently performs wetland mitigation on a project by project basis. While the Iowa DOT is mitigating projects on a case by case basis, other agencies such as the Iowa Department of Natural Resources and the Natural Resource Conservation Service are performing wetland restoration projects, and counties and cities are mitigating wetland losses as well. The Iowa DOT desired to determine whether state and local resources may be utilized cooperatively in developing shared wetland mitigation projects in ways that will benefit both Iowa agencies and local governments.

The research objectives were to conduct a survey on wetland mitigation activities in Iowa to see if there is an interest and a demand to develop a wetlands clearinghouse concept, develop instruments for cooperative wetland mitigation, and exchange and publish the information discovered in the project.

The research results included development of a conceptual framework for cooperative wetland mitigation, reviewing the existing inventories available for wetlands mitigation, developing a template project outline and flow chart of the proposed Iowa Wetland Mitigation Clearinghouse concept, creating a template cooperative umbrella instrument, and raising the awareness of and appreciation for the value of local wetlands and their responsible use through educational materials, technical assistance, and workshops.

In a national-level review, the research did not discover other processes utilizing a clearinghouse concept for wetlands mitigation. As a result, a model was defined in the research.

Key words: clearinghouse—Iowa Wetland Mitigation Clearinghouse—wetlands inventories—wetlands mitigation

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PROBLEM STATEMENT

The Iowa Department of Transportation (Iowa DOT) currently performs wetland mitigation on a project by project basis. While the Iowa DOT is mitigating projects on a project by project basis, other agencies such as the Iowa Department of Natural Resources (Iowa DNR) and the Natural Resource Conservation Service (NRCS) are performing wetland restoration projects, along with counties and cities that are mitigating wetland losses as well. The Iowa DOT desired to determine whether state and local resources may be utilized cooperatively in developing shared wetland mitigation projects in ways that will benefit Iowa agencies and local governments.

RESEARCH OBJECTIVES

The purpose of this project is to develop a framework for an Iowa Wetland Mitigation Clearinghouse (IWMC) and showcase typical inventories that will serve agencies and communities involved with wetland mitigation.

- Developed a conceptual framework for cooperative wetland mitigation that utilizes the concept of an IWMC.
- Reviewed the inventories available for wetlands mitigation in Iowa.
- Developed a template project outline and flow chart of the proposed IWMC concept.
- Created a template cooperative umbrella instrument (CUI) to be used in the approval process.

This paper will discuss the conceptual framework for cooperative wetland mitigation, the inventories currently available to identify and categorize wetlands along with flow charts for the proposed IWMC concept.

RESEARCH METHODOLOGY

The research methodology utilized several diverse activities. First, the results of the Iowa Highway Research Board (IHRB) project TR-526 was reviewed and used as a guide for the research team. The IHRB project recommended developing a framework for an IWMC. To assure themselves that this recommendation would meet the regulations included in the Clean Water Act of 1977, the team evaluated the Act and researched other processes across the nation currently used for wetlands mitigation.

The evaluation led the team to the conclusion that the concept of the IWMC would meet the requirements. The IWMC concept was developed along with the advantages, potential products that could be developed, and barriers that may need to be removed, managed, or overcome. The duties of the IWMC included developing manageable inventories, providing quality assessment and oversight, and completing the documentation when the site is closed. Finally the IWMC activities were developed as they would pertain to a typical roadway building project..

Iowa Highway Research Board (IHRB) Project TR-526

At the beginning of this research project there was a desire to assess the potential for collaborative development of wetland mitigation projects in Iowa. The IHRB initiated project TR-526 that completed a study of the interest to participate in wetlands banking. The study was conducted by the Center for Transportation Research and Education (CTRE), a center of Iowa State University. The Iowa DOT and the Iowa DNR were interviewed, and a survey of Iowa cities and counties was conducted.

The findings of the IHRB study indicate that most wetland mitigations conducted by the Iowa DOT in the last five years have constituted small acreages that are well under the 25-acre minimum adopted by Iowa's Mitigation Banking Review Team (MBRT) for considering wetland banking. The Iowa DOT staff in the Office of Location and Environment have a successful process in place for complying with the National Environmental Policy Act and for obtaining 401 and 404 permits when needed. Project delays do not appear to be associated with obtaining 404 permits. Accordingly, there is not a strong need for the Iowa DOT to change the current permitting process or to engage in wetland banking.

Moreover, wetland banking incurs financial risks. The Iowa DOT would have to spend state highway funds up front to build mitigation banks and hope to recoup the funds with project funds when the bank sells credits in the future. There are several cases around the country where a withdrawal of credits has not been approved after a bank was built, and thus the Iowa DOT should move very cautiously with regard to banking. An opportunistic approach is recommended. If a road building project requiring mitigation is in an area where partners can be found, the Iowa DOT should be open to banking or other collaborative actions. This concept opens the door for considering a clearinghouse process to facilitate collaborative action.

The Iowa DNR procures land annually for wetland restoration under the Prairie Pothole Joint Venture with the U.S. Fish and Wildlife Service. There is potential for the Iowa DOT to collaborate using this project, but only in the north central part of the state and under special location circumstances. The NRCS also purchases wetlands from landowners under the Wetland Reserve Program (WRP) and temporarily rents land under the Farmable Wetland Program (FWP). By viewing the NCRS as a local entity and by considering its activities in wetlands acquisition, there is potential for a clearinghouse function to be located with the NRCS.

The survey of Iowa cities and counties, included in the IHRB study, revealed that mostly very small mitigation activities (fewer than five acres) have occurred in the last five years. Cities and counties report administrative difficulties and expenditure of time and money on the 404 permit process. Based on the survey, counties, cities, and county conservation boards are willing to collaborate as partners if the conditions are right. However, only a few counties report mitigation projects that would support wetlands banking.

Two of the recommendations of the IHRB study focused on a wetlands clearinghouse process to facilitate collaborative actions.

1. *A site identification clearinghouse involving the NRCS should be established*. The current problems with mitigation include the following:
    a. Obtaining wetland property for mitigation requires the Iowa DOT to purchase approximately four times the acreage required due to the real estate market.
    b. The Iowa DOT does not want to manage wetlands or own excess property.
    c. The sustainability of mitigation sites in Iowa, although good by national standards, could be improved.

    Because of these problems, the IHRB report recommends an IWMC centered at the NRCS to help identify landowners willing to sell wetlands. The Iowa DOT should request that the NRCS contact applicants to the WPR and landowners exiting the FWP, in affected HUC 8 districts, to aid in obtaining mitigation sites. The NCRS would then bring the landowner and the Iowa DOT together for negotiations.

2. *A partnership clearinghouse should be established*. To help identify potential site managers or other agencies with mitigation needs, this report recommends an IWMC. While the partnering process currently happens on a project-by-project basis, the Iowa DOT should routinely contact cities, counties, the DNR, the Farm Bureau, the County Conservation Commission, and others as early as possible in the mitigation process to see whether others have mitigation or restoration needs in the area or are willing to consider a management contract.

NATIONAL REVIEW SUMMARY

The research looked into the regulations that guide the wetland mitigation process, the Clean Water Act of 1977 (the Act). Two collaborative methods that have been used to obtain compliance with these regulations in other locations across the nation are wetland mitigation banks and in-lieu fee mitigation. Both of these methods are discussed, and their shortfalls are identified. The research team did not find any indications that others are using or considering a concept similar to the IWMC for their collaborative wetlands mitigation efforts.

The U.S. Environmental Protection Agency (EPA) defines wetlands as follows:

> As used in this regulation, [wetlands] shall include those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Historically, wetlands had been drained for farmland or developments because wetlands were considered unproductive and a nuisance. Advances in understanding the ecological function of wetlands led to placing a high value on preserving them, culminating in the Act.

Two sections of the Act require permits for projects with impacts on wetlands. Section 401 of the Act gives the EPA authority to regulate activities that have the potential to adversely affect water quality. Included in this category are projects that impact natural and artificial wetlands. When permitted activities disrupt wetlands, the Clean Water Act requires compensatory mitigation to offset the loss.

Section 404 gives the United States Corps of Engineers (Corps) the jurisdiction to grant permits for construction activities within waterways and wetlands. This section requires any construction project that may require the loss of an acre or more of wetlands to notify and apply for a permit from the Corps. The Iowa DOT and any other entity building a roadway project that impacts a wetland must file for a permit.

Section 404 of the Act regulates the “discharges” of “dredged or fill material” into waters of the United States. Since wetlands have water, they fall under this section of the Act. The section also states that the wetlands program goal is “no net loss of wetlands.” The Act does contain some wetland exemptions and allows some types of projects, such as highway building, to automatically receive general permits.

Wetland Mitigation Banks

The 1991 Intermodal Surface Transportation Efficiency Act specifically addressed the use of wetland mitigation banks and authorized the use of federal funds for this type of wetland remediation. In 1993, President Clinton released his wetlands protection plan called “Protecting America’s Wetlands: A Fair, Flexible, and Effective Approach.” This plan attempted to balance the needs of landowners with the need to prevent further wetland losses. This plan endorsed the increased use of mitigation banking. Further direction was given to using wetland mitigation banks in a 1995 Corps Memorandum to the Field, titled "Federal Guidance for the Establishment, Use, and Operation of Mitigation Banks."

The EPA and the Corps established the National Wetlands Mitigation Action Plan in 2002. The goal of that plan was to provide no net loss of the nation's wetlands. The following guiding themes are integral to the mission:

- Provide a consistent voice on compensatory mitigation matters
- Focus guidance, research, and resources to advance ecologically meaningful compensatory mitigation
- Provide information and options to those who need to mitigate the losses of wetland functions
- Provide technical and research assistance to those who undertake the work of mitigation

As part of this plan, the EPA and the Corps provided guidance for compensatory mitigation projects and appropriate use of preservation and vegetative buffers as a component of compensatory mitigation.

Iowa has established its MBRT as required under the terms of the 1995 Corps memorandum. The MBRT prepared "Mitigation Banking in Iowa," a draft document that establishes procedures for creating a mitigation bank in Iowa.

In-Lieu Fee Mitigation

In-lieu fee mitigation occurs in circumstances where a permittee provides funds to an in-lieu fee sponsor instead of either completing project-specific mitigation or purchasing credits from a mitigation bank approved under the Banking Guidance as referenced in the Federal Register 2000.

In-lieu fee mitigation, or other similar arrangements wherein funds are paid to a natural resource management entity for implementing either a specific or general wetland or another aquatic resource development project, is not considered to meet the definition of mitigation banking because these projects do not typically provide compensatory mitigation in advance of project impacts. Moreover, such arrangements do not typically provide a clear timetable for the initiation of mitigation efforts.

The Corps, in consultation with other agencies, may find circumstances for which such arrangements are appropriate, as long as the arrangements meet the requirements that would otherwise apply to an offsite prospective mitigation effort and provide adequate assurances of success and timely implementation. In such cases, a formal agreement between the sponsor and the agencies, similar to a banking instrument, is necessary to define the conditions under which its use is considered appropriate.

In summary, when reviewing the current legislation, the collaborative wetlands mitigation processes don't work very well for road building projects. Neither wetlands mitigation banking nor in-lieu fee mitigation meets the needs of agencies trying to provide small mitigation sites along a roadway corridor. The legislative review supports the recommendations of the IHRB study to provide another mechanism, the IWMC, to allow all levels of government and all agencies the opportunity to collaborate on wetlands mitigation efforts.

Defining the IWMC Concept

Following the lead from the IHRB study to develop a concept for the IWMC, the research team was interested in a management organization that is already established, working with all levels of government and agencies and was positioned to carry out the duties of the IWMC. They focused on the NRCS and its local arm in central Iowa, the Prairie Rivers Resource Conservation and Development (Prairie Rivers RC&D).

Considering Prairie Rivers Resource Conservation and Development

The research team included the Iowa Division of the Federal Highway Administration (FHWA), the Iowa DOT, CTRE, and the Prairie Rivers RC&D. The Prairie Rivers RC&D is located in central Iowa and services the counties of Webster, Hamilton, Hardin, Boone, Story, and Marshall. Prairie Rivers RC&D was a logical research partner choice because it is located in Story County, close to CTRE, it has a board of public officials that provide oversight, it currently receives funding from the NRCS, and it has an existing staff available. Please refer to Figure 1. These elements were thought to be critical to the success of a pilot wetlands mitigation project in central Iowa.

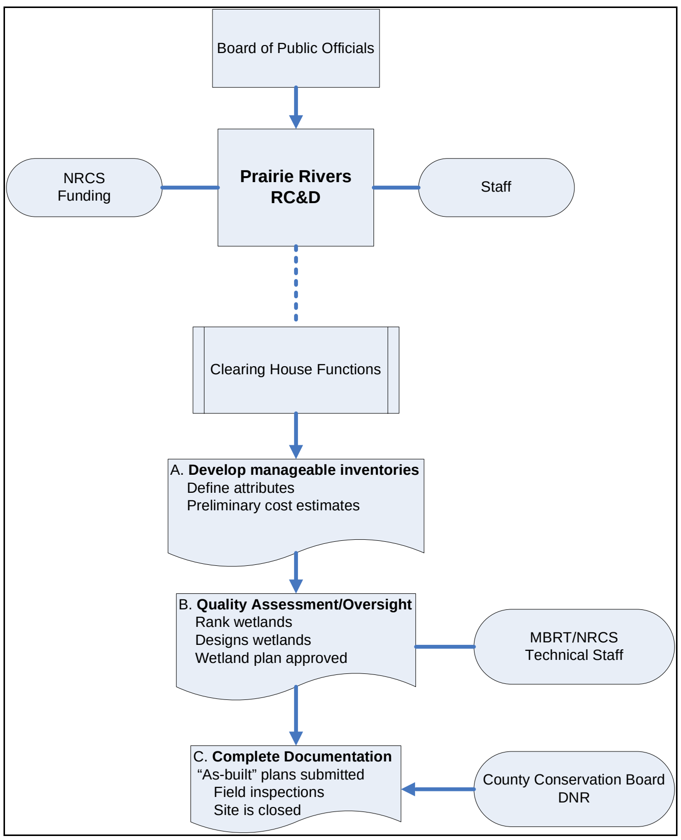

Figure 1. The IWMC concept

Advantages, Potential Products, and Barriers

As the research team evaluated Prairie Rivers RC&D as an organization to manage the IWMC they identified the advantages to the clearinghouse concept, the potential products that could be developed, and the barriers that may need to be removed, managed, or overcome.

Advantages

- This concept serves the NRCS in-agency mitigation needs and would fast-track mitigation procedures for the Iowa DOT, the Iowa DNR, along with cities and counties.
- This concept should result in better mitigation projects in general and an overall coordinated effort.

Potential Products

- Manageable inventories of potential banking sites and their attributes would be developed and would be envisioned as preservable and restorable.
- Preliminary cost estimates for restoration would be developed. The supply of banking sites would be priced and available for evaluating alternatives.
- A programmatic agreement would be developed and a banker agreement negotiated.
- Long-term monitoring would be available.

Barriers

- Funding would be required for the initial mapping exercise and for developing engineering cost estimates.
- Long-term funding would be required for quality assessment and administration duties.
- The price for mitigation sites may need to be moved into private ownership and out of government oversight.
- Developing and agreeing upon the determination of these wetland sites.

IWMC Responsibilities

The IWMC is envisioned to carry out tasks that include developing manageable inventories, providing quality assessment and oversight, and to be responsible for completing the documentation to close the wetland mitigation site activities. Please refer to Figure 1.

Manageable Inventories

Wetlands are classified by the vegetation that is growing in the area, the soil classifications, and the wetland hydrology. These are the three inventories that should considered in the wetlands evaluation efforts.

To evaluate the vegetation, it is most common to visit the site and do an inventory of the plant species growing in the wetland area. The survey would attempt to document a prevalence of vegetation typically adapted for life in saturated soil conditions. On the other hand, recorded data is frequently available for the soil classification and the wetland hydrology.

The soils that are present in a wetland area have been classified as hydric, or they possess characteristics that are associated with reducing soil conditions. These soils consist of unconsolidated natural materials that support, or are capable of supporting, plant life. Hydric soils are classified into two broad categories: organic and mineral.

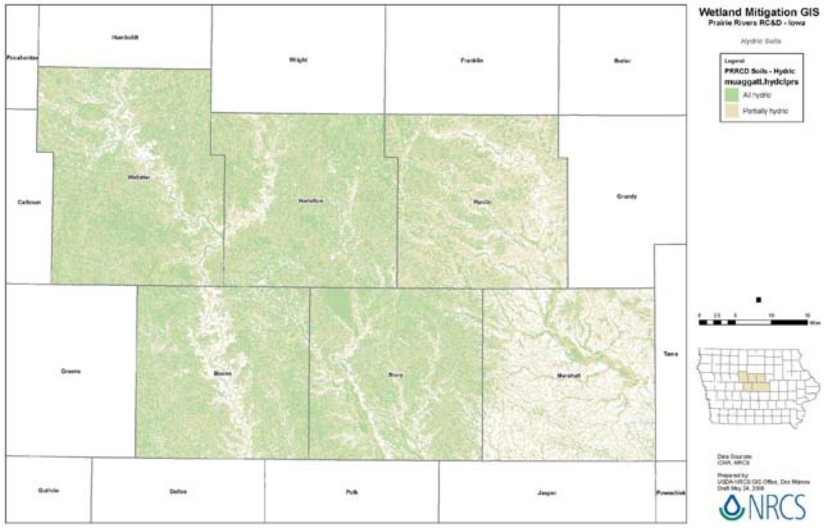

Figure 2. Hydric soils, Prairie Rivers RC&D District

Figure 2 is a soils map is of the six-county Prairie Rivers District, and Figure 3 was printed at the Marshall County–level. They are prime examples that illustrate the soils information that is already in existence and available as a resource for wetlands soil classification.

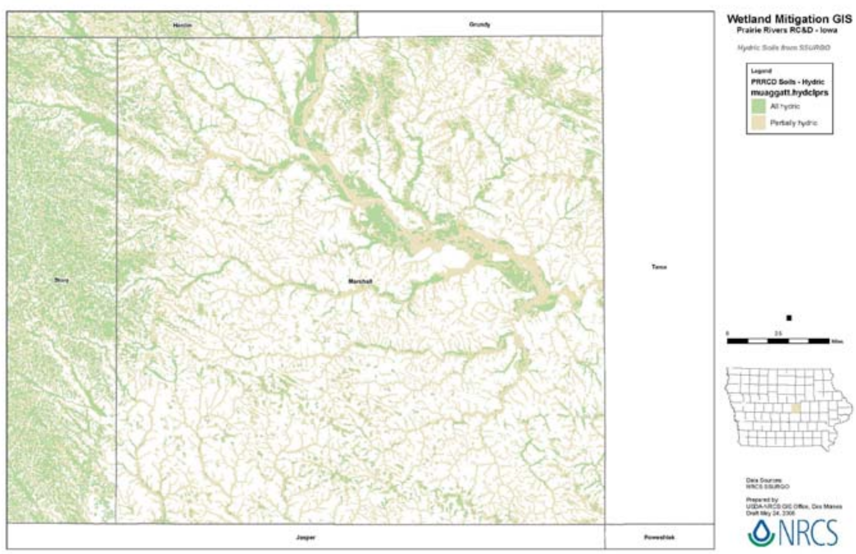

Figure 3.Hydric soils, Marshall County

Wetlands hydrology encompasses all hydrologic characteristics of areas that are periodically inundated or contain soils that are saturated to the surface at some time during the growing season. Numerous factors can influence the wetness of an area, e.g., precipitation, topography, soil permeability, and plant cover. Indicators of wetland hydrology may include, but are not limited to, drainage patterns, drift lines, sediment deposition, watermarks, stream gage data and flood predictions, historic records, visual observation of saturated soils, and visual observation of inundation. Figure 4 is a map of the six-county soil drainage classifications and illustrates the hydrological inventory that is available for wetlands determination.

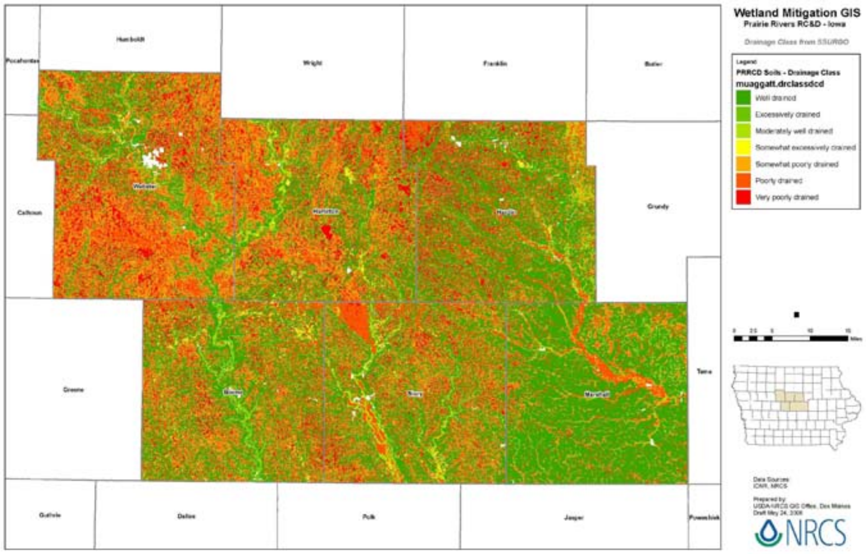

Figure 4. Soil drainage classification, Prairie Rivers RC&D District

Once the soil and hydrology facts are known in an area, the next step would be to determine the ownership of the land. There are several sources of this data and some of it has been mapped on geographic information system (GIS) based maps.

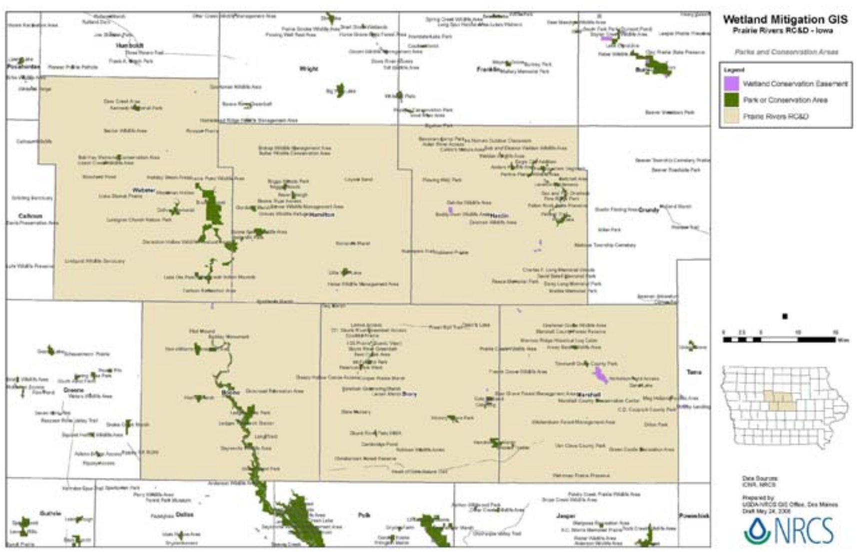

Figure 5. Wetlands ownership, Prairie Rivers RC&D District

Shown in Figure 5 are the six Prairie Rivers District counties that identify whether the land ownership is a wetland conservation easement, a park or conversation area, or a Prairie Rivers RC&D holding. Figure 6 is a Story County map showing parks and conservation areas ownership.

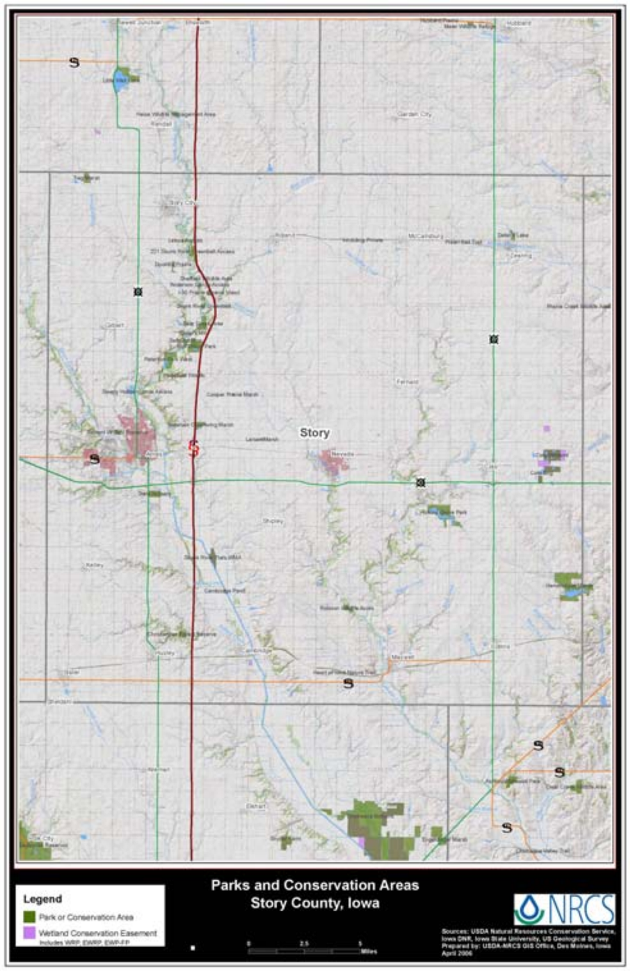

Figure 6. Wetlands ownership, Story County

In summary, the research team found the following to be true of the Iowa inventories that would be available for wetlands determination.

- Some data sets are not available for every Iowa county, or some data sets are incomplete: those of interest would be the remapped National Wetlands Inventory, drainage districts, and wetlands determination maps.
- It appears logical to select potential sites based on proximity to existing wetlands or natural areas.
- In order to narrow the hydric soils, it is logical to use drainage classification and ponding frequency.
- Nonfunded WRP applications may be a source of landowners who may have an interest in developing a wetlands mitigation project.
- After finding willing landowners, GIS could be used for field-scale analysis.

Complete Wetland Site Documentation

The documentation that is envisioned for the IWMC includes ranking the candidate wetlands mitigation sites, the development and approval of a wetland restoration and maintenance plan, overseeing the construction and maintenance activities, providing “as-built” plans, and finally closing the site at the end of its use. The concept also includes an annual report to the MBRT.

IWMC Mitigation Activities

The approach used for wetlands mitigation will vary, based on the area in question. There are two basic approaches: routine and comprehensive.

1. *Routine approach*. The routine approach will normally be used in the vast majority of determinations. The routine approach requires a minimal level of effort, using primarily qualitative procedures. This approach can be further subdivided into three levels of required effort, depending on the complexity of the area and the amount and quality of preliminary data available. The following levels of effort may be used for routine determinations:
    a. Level 1. Onsite inspection unnecessary
    b. Level 2. Onsite inspection necessary
    c. Level 3. Combination of levels 1 and 2

2. *Comprehensive approach*. The comprehensive approach requires application of quantitative procedures for making wetland determinations. It should seldom be necessary, and its use should be restricted to situations in which the wetland is very complex and/or is the subject of likely or pending litigation. Application of the comprehensive approach requires a greater level of expertise than application of the routine approach, and only experienced field personnel with sufficient training should be used.

Figure 7 shows how the IWMC mitigation activities would be accomplished. A customer requiring mitigation first contacts Prairie Rivers RC&D to view options about site inventory. The customer then chooses options for the 401/404 permit based on site, location, and customer-specific goals. When the customer's permit is approved by the appropriate regulatory agencies, the customer pays mitigation fees to Prairie Rivers RC&D, the contract is finalized between the two parties, and Prairie Rivers RC&D begins the mitigation process. Prairie Rivers RC&D then hires a contractor (private, NRCS, or DNR) to design and construct the wetland, and the wetland management plan is created and approved. The wetland is then constructed and maintained according to the agreement with the regulatory agencies. "As-built" plans and specifications are submitted to the Corps. The wetland is monitored as per the wetland management plan specified in the agreement. Finally, the site is closed.

While the Prairie Rivers RC&D is completing the above process with the customer, they complete an agreement with the inventory site holder, and the site is removed from the inventory list of potential wetland mitigation sites.

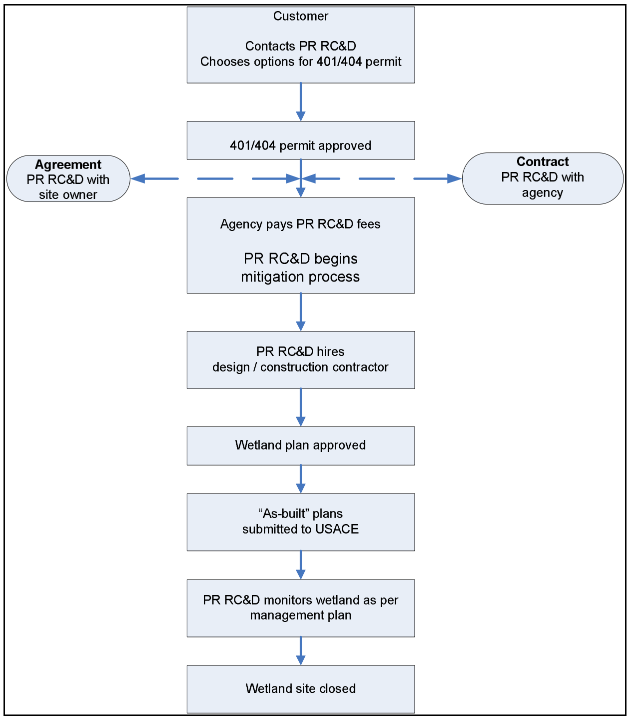

Figure 7. IWMC mitigation activities

KEY FINDINGS

The research team found the following to be the key findings from the research efforts:

- To facilitate collaborative wetland mitigation actions, the IHRB study established the desire to establish a wetlands clearinghouse process.
- The IHRB study recommended a partnership wetlands clearinghouse be established.
- Wetlands mitigation banks and in-lieu fee mitigation may not be the best options for meeting the requirements of the Clean Water Act of 1977.
- The research team did not discover another partnership clearinghouse concept while reviewing activities across the nation.
- Local arm of the NCRS, the Prairie Rivers RC&D provides basic structure.
- The duties of the IWMC are defined.
- Most inventories used for wetlands determination are available.
- A CUI was developed and is available for review in the research final report.

SUMMARY RECOMMENDATIONS

The following are the summary recommendations for this research project.

- Complete the pilot project for the purpose of testing the IWMC concept.
- Utilize the local arm of the NRCS, Prairie Rivers RC&D, as the mitigation agent; they have paid staff available; they are governed by a public official board, and they are established with government agencies in central Iowa.
- The Iowa DOT should contract with Prairie Rivers RC&D for a mitigation pilot project.
- The Iowa DOT should use Prairie Rivers RC&D as a statewide model.
- The Corps should work with the DNR on service area definitions.
- The Iowa DOT should develop a user's handbook for those new to using the RC&D in the wetlands mitigation process.

ACKNOWLEDGMENTS

The author would like to thank Mike LaPietra of the Federal Highway Administration, Iowa Division, and Scott Marler of the Iowa Department of Transportation, Office of Location and Environment, for sponsoring this research. Jim Cooper, Resource Conservation and Development (RC&D) Coordinator for the Prairie Rivers of Iowa RC&D District, was also a prime player in visualizing the concept of this research and in providing documentation that illustrates our concept. Finally, the authors thank Gregg Hadish of the United States Department of Agriculture, Natural Resource Conservation Service, for providing the mitigation banking maps used in this research.

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Iowa Department of Transportation's Statewide DMS Deployment Program

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ABSTRACT

In 2004, the Iowa Department of Transportation started a statewide dynamic message sign (DMS) deployment program. Initially, 101 locations across Iowa were identified as potential locations for DMS. This number was narrowed down to 13 locations for year one deployment in FY 2005. Another 15 locations were planned for FY 2006–2008. This will add to the existing 21 locations selected before the program started. The deployment plan keeps a five-year outlook on potential sites. Procurement procedures, software, communications, agreements, maintenance access/responsibilities, and operational guidelines were other aspects that were updated, reevaluated, or established. The deployment program has evolved to include DMS for rest areas, side-mount, portables, and DMS in school speed zones.

Key words: dynamic message signs—Iowa

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Scheduling Specification for the Rehabilitation and Reconstruction of I-235

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ABSTRACT

On behalf of the Iowa Department of Transportation (Iowa DOT), Iowa State University has developed a review process to monitor contractor compliance with the Iowa DOT construction scheduling specification. The process also involved review of schedule updates in comparison to actual work performed. Records of the reviews and project progress photos have been retained using a redundant system of both hard copies and electronic backup files. Specifically, the system has been implemented on the I-235 corridor reconstruction project stretching from the downtown Des Moines area to the north I-35/I-80/I-235 system interchange. This paper provides a case study that focuses on a project having a contract value of approximately $93 million and that is being constructed under an average traffic volume of between 115,000 and 120,000 vehicles per day in peak traffic areas. The paper presents a detailed description of the schedule review system, a discussion on the lessons learned from the implementation of the system, and recommendations for those who may need to institute a similar system in the future.

Key words: construction—I-235—Iowa Department of Transportation—specification compliance—schedule

INTRODUCTION

On October 25, 2005, the Iowa Department of Transportation (Iowa DOT) let its largest contract to date in the amount of $93,118,005.65. It was let as several tied or combined projects, with IM-NHS-235-2(269)7 – 03-77 designated as the prime project. Herein, the project is referenced as (269), the short version of the project number used by most project participants. The objective of (269) was to reconstruct I-235 from Cottage Grove Avenue to East 16th Street in Des Moines, Iowa. The magnitude and visibility of the project prompted the Iowa DOT to develop a Special Provisions for Progress Schedule by Critical Path Method (SPEC), which amends the Standard Specifications, Series 2001. The SPEC states, “The contractor shall submit a progress schedule of construction activities based on the critical path method (CPM) of scheduling….The CPM progress schedule shall be used for coordination and monitoring of all work under the contract, including all activities of subcontractors, vendors, and suppliers. The Engineer will review the CPM progress schedule and forward comments to the contractor” (Iowa DOT 2005).

The Iowa DOT contracted Iowa State University (ISU) to review contractor compliance with scheduling specification and to offer assistance. The SPEC states, “The Engineer’s review comments will neither bind the Contracting Authority nor constitute acceptance of any portion of the schedule” (Iowa DOT 2005). Throughout the duration of the contract, ISU has reviewed the contactor-submitted schedules, compared the schedules with actual progress in the field, and attended regular meetings where scheduling issues were discussed. ISU has developed recommendations for an efficient SPEC compliance review system.

THE I-235 CORRIDOR RECONSTRUCTION PROJECT

The (269) project was part of a multiyear, multiple contract effort to rebuild the entire I-235 corridor through the Des Moines metro area. The corridor includes 13.83 miles of freeway from the east system interchange of I-80/I-35/I-235 to the west system interchange of I-80/I-35/I-235 through the heart of Des Moines, Iowa (Figure 1). From 2002 to 2007, contracts for approximately $429 million of construction will have been completed in this area (Iowa DOT 2007a). Work consisted of the rehabilitation and reconstruction of numerous utilities, bridges, and miles of interstate paving, all of which were constructed under traffic. Currently the estimated traffic count is 115,000 to 120,000 vehicles per day. The purpose of the corridor reconstruction project is to “improve safety, update the facility to current roadway design standards, reduce congestion, and improve mobility” (Iowa DOT 2007b).

Figure 1. Map of I-235 corridor (Iowa DOT 2007c)

SCHEDULE SPECIFICATION

Formal scheduling efforts with the assistance of ISU started in 1999, when a program-level schedule was developed for the entire I-235 corridor (Chen, Jahren, and Canales 2003). The program-level schedule provided information on each construction contract and each major utility conflict as separate activities. Logical relationships between these activities and a schedule were also provided. This process was helpful because it identified projects that were critical for the timely completion of the entire corridor. When such projects were identified, the Iowa DOT took several actions to ensure that the project completion did not delay corridor completion. Possible actions included starting the project early in the corridor reconstruction process, budgeting to accommodate appropriate bids for a tightly scheduled project, and setting up incentive/disincentive schemes. From 2000 to 2005, project-level schedules for each contract were provided by the contractor and reviewed by the Iowa DOT according to Section 1110 of the Iowa DOT Standard Specifications (e.g., see GS 1101, 2006).

Currently, the Iowa DOT funds approximately $400 million of highway construction per year. This is generally made up of small, rural, standardized projects that are familiar to the construction industry and the Iowa DOT. For routine projects such as these, the risk and consequences of a poor schedule is less in comparison to a large, urban, unique project. The (269) project was sufficiently large enough to possibly draw out-of-state contractors to bid on the project. Therefore, (269) was going to be either constructed by an in-state contractor who had never taken on a project of this magnitude or an out-of-state contractor who was not familiar with Iowa DOT business practices. In either case, a higher degree of project-level scheduling in comparison to past projects was deemed appropriate to ensure smooth and timely progress on the schedule. In addition, (269) had an incentive of $70,000 per calendar day, up to a maximum of $2.1 million for 30 days prior to the final completion date, and a disincentive of $30,000 per calendar day beyond the final completion date.

In addition to the (269) project, other projects were concurrently under construction along the I-235 corridor that had similar levels of importance and that were also conducted under the SPEC. In order to provide focus for this paper, the authors primarily refer to the (269) project. However, it is worth noting that the other important projects existed, because concurrent execution of several critical projects was part of the rationale for using SPEC.

The SPEC states, “The contractor shall submit a progress schedule of construction activities based on the critical path method (CPM) of scheduling….The CPM progress schedule shall be used for coordination and monitoring of all work under the contract, including all activities of subcontractors, vendors, and suppliers. The CPM progress schedule shall include provisions for traffic control, staging, and other events necessary to complete all work involved in the contract. This schedule shall be the Contractor’s intended working schedule and shall be used to plan, organize, and execute the work; record and report actual performance and progress; and forecast remaining work” (Iowa DOT 2005).

After the contractor has turned in a progress schedule, “the engineer will review the CPM progress schedule and forward comments to the contractor within 7 calendar days” (Iowa DOT 2005). The contractor was thus required to develop a computer-generated schedule displaying the following: activity descriptions, durations, dollar values, major crews, and equipment for each activity. In addition, a cost curve showing cumulative expected progress payments vs. time was required.

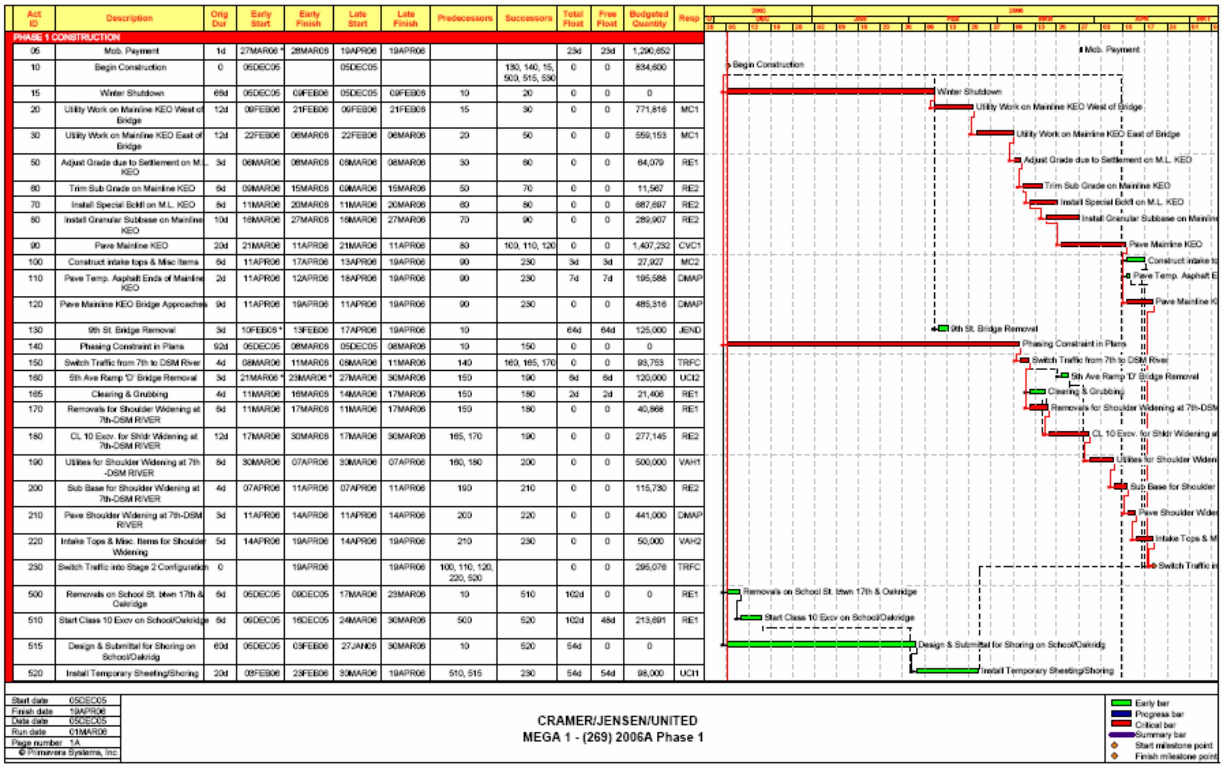
The CPM progress schedule was also used in construction operations. The contractor was required to “conduct weekly job site meetings with the Engineer to verify CPM progress schedule accuracy” (Iowa DOT 2005). Furthermore, the contractor was required to update the schedule as required, which was determined to be every other week, to reflect the actual progress of work.

REVIEW SYSTEM

The duty of ISU was to develop a system to review contractor compliance with regard to the SPEC. This was accomplished by reviewing the schedule, conducting weekly field visits to compare actual progress with scheduled progress, and attending weekly meetings to coordinate activities. Other services included taking weekly progress photos, providing data backup, and maintaining a project website that was available to the contractor, subcontractor, and Iowa DOT.

Specification Compliance of Schedule

During the initial stages of schedule development, ISU reviewed schedule submissions as the contractor worked to develop a baseline schedule (Figure 2). For each version of the pre-baseline schedule submitted, ISU completed a schedule review checklist (Figure 3). The completed schedule checklist was submitted to the Iowa DOT’s Resident Construction Engineer and the contractor’s project scheduler. In an iterative process, the schedule was revised until it reached compliance with the SPEC, at which point it was accepted as the “baseline” schedule. According to Hinze (2004), “The baseline schedule provides a measuring stick for comparing the as-built schedule. It is used not only as a management tool for determining the accuracy of planning efforts, but also as a basis for any construction delay claims.” The contractor’s project scheduler would then provide schedule updates, typically on a biweekly basis.



Act ID	Description	Orig Dur	Early Start	Early Finish	Late Start	Late Finish	Predecessors	Successors	Total Float	Free Float	Budgeted Quantity	Resp
PHASE 1 CONSTRUCTION												
05	Mob. Payment	1d	27MAR06 *	28MAR06	19APR06	19APR06			23d	23d	1,290,652	
10	Begin Construction	0	06DEC05		05DEC05			130, 140, 15, 500, 515, 530	0	0	834,600	
15	Winter Shutdown	66d	06DEC05	09FEB06	05DEC05	09FEB06	10	20	0	0	0	
20	Utility Work on Mainline KEO West of Bridge	12d	09FEB06	21FEB06	09FEB06	21FEB06	15	30	0	0	771,816	MC1
30	Utility Work on Mainline KEO East of Bridge	12d	22FEB06	08MAR06	22FEB06	08MAR06	20	50	0	0	559,153	MC1
50	Adjust Grade due to Settlement on M.L. KEO	3d	06MAR06	08MAR06	08MAR06	08MAR06	30	60	0	0	64,079	RE1
60	Trim Sub Grade on Mainline KEO	6d	09MAR06	15MAR06	09MAR06	15MAR06	50	70	0	0	11,567	RE2
70	Install Special Bckfl on M.L. KEO	8d	11MAR06	20MAR06	11MAR06	20MAR06	60	80	0	0	687,697	RE2
80	Install Granular Subbase on Mainline KEO	10d	16MAR06	27MAR06	16MAR06	27MAR06	70	90	0	0	289,907	RE2
90	Pave Mainline KEO	20d	21MAR06	11APR06	21MAR06	11APR06	80	100, 110, 120	0	0	1,407,232	CVC1
100	Construct intake tops & Misc Items	6d	11APR06	17APR06	13APR06	19APR06	90	230	3d	3d	27,927	MC2
110	Pave Temp. Asphalt Ends of Mainline KEO	2d	11APR06	12APR06	18APR06	19APR06	90	230	7d	7d	195,588	DMAP
120	Pave Mainline KEO Bridge Approaches	9d	11APR06	19APR06	11APR06	19APR06	90	230	0	0	485,316	DMAP
130	9th St. Bridge Removal	3d	10FEB06 *	13FEB06	17APR06	19APR06	10		64d	64d	125,000	JEND
140	Phasing Constraint in Plans	92d	06DEC05	08MAR06	05DEC05	08MAR06	10	150	0	0	0	
150	Switch Traffic from 7th to DSM River	4d	08MAR06	11MAR06	08MAR06	11MAR06	140	160, 165, 170	0	0	93,753	TRFC
160	5th Ave Ramp 'D' Bridge Removal	3d	21MAR06 *	23MAR06 *	27MAR06	30MAR06	150	190	6d	6d	120,000	UCI2
165	Clearing & Grubbing	4d	11MAR06	16MAR06	14MAR06	17MAR06	150	180	2d	2d	21,406	RE1
170	Removals for Shoulder Widening at 7th-DSM RIVER	6d	11MAR06	17MAR06	11MAR06	17MAR06	150	180	0	0	40,868	RE1
180	CL 10 Excv. for Shldr Widening at 7th-DSM RIVER	12d	17MAR06	30MAR06	17MAR06	30MAR06	165, 170	190	0	0	277,145	RE2
190	Utilites for Shoulder Widening at 7th -DSM RIVER	8d	30MAR06	07APR06	30MAR06	07APR06	160, 180	200	0	0	500,000	VAH1
200	Sub Base for Shoulder Widening at 7th-DSM RIVER	4d	07APR06	11APR06	07APR06	11APR06	190	210	0	0	115,730	RE2
210	Pave Shoulder Widening at 7th-DSM RIVER	3d	11APR06	14APR06	11APR06	14APR06	200	220	0	0	441,000	DMAP
220	Intake Tops & Misc. Items for Shoulder Widening	5d	14APR06	19APR06	14APR06	19APR06	210	230	0	0	50,000	VAH2
230	Switch Traffic into Stage 2 Configuration	0		19APR06		19APR06	100, 110, 120, 220, 520		0	0	295,076	TRFC
500	Removals on School St. btwn 17th & Oakridge	6d	06DEC05	09DEC05	17MAR06	23MAR06	10	510	102d	0	0	RE1
510	Start Class 10 Excv on School/Oakridge	6d	09DEC05	16DEC05	24MAR06	30MAR06	500	520	102d	48d	213,691	RE1
515	Design & Submittal for Shoring on School/Oakridg	60d	06DEC05	03FEB06	27JAN06	30MAR06	10	520	54d	0	0	
520	Install Temporary Sheeting/Shoring	20d	03FEB06	23FEB06	30MAR06	19APR06	510, 515	230	54d	54d	98,000	UCI1

Figure 2. Sample baseline schedule

Contractors sometime submit post-bid value engineering (VE) proposals to the Iowa DOT in accordance with GS 1105.15 (Iowa DOT 2006). During the process of construction, delays and setbacks are very common. If a VE proposal was submitted, or if a delay or setback occurred, ISU would review the schedule impacts and provide comments. This was helpful for the Iowa DOT when it was necessary to obtain approvals for the changes.

I-235 Schedule Checklist

Date Received: ______	Schedule:	
# of paper copies recv'd ______ (8 1/2 x 11 required)		
Computer schedule recv'd?	Program used	PDF recv'd?

Item	Y or N	Comments
Activity Node		
Activity Description		
Activity Duration		
Logic		
Shop drawing submittals & approvals		
Fabrication activity		
Responsibility		
Total Cost (within 2%)		
Total Float		
Early Start		
Predecessors		
Successors		
Expected Payment vs. Time curve		
Less than 10 days float		
Work Days, shifts & hours		
Major crews (equipment and # of workers)		

Reviewed by: ____________________ (print name)

Date reviewed: __________ Signature: ______________________________ (signature of reviewer)

Figure 3. I-235 schedule checklist

Weekly Field Visits

After the "baseline" schedule was accepted and a schedule update was submitted, an ISU team member visited the construction site weekly to perform crew checks on critical activities. This was done to compare as-planned crew sizes to observed crew sizes. Additionally, the progress of the work was checked by comparing the updated schedule to the observed progress. A field verification sheet was developed to streamline this process (Figure 4); it was submitted to the Iowa DOT's Resident Construction Engineer weekly.

Project Number:	___________________			
Date:	___________________			
Observer Name:	___________________			
On *(enter date here)* Iowa State University conducted a field visit to check on the progress of work at *(enter number of areas checked here)* areas of (*enter project number here*): (*enter areas visited here*). The CPM schedule showed the following as planned activites:				
Activity ID#:	**As Planned Activity Description:**		**Observed Activity Description:**	
In addition to checking the progress of work Iowa State University performed crew checks at *(enter number of areas checked here*) areas of *(enter project number here)*: *(enter areas visited here)*. The CPM schedule showed the following crews.				
Activity ID#:	**As Planned Activity:**	**Responsibility:**	**As Planned Crew:**	**Observed Crew:**
Comments:				
Authored By:				
	(print name here)			
	(sign name here)			

Figure 4. Field verification sheet

Weekly Progress Photos

During the weekly field visits, digital progress photos of multiple areas of construction along I-235 were taken to document the progress of work in each area. These photos were submitted via CD-ROM to the Iowa DOT's Resident Construction Engineer on a monthly basis and were utilized on the project website. Photos were also taken prior to and at the end of each phase of construction, where disincentives might be assessed.

Weekly Progress Meetings

The Iowa DOT conducted weekly progress meetings with contractors and subcontractors involved in the I-235 project. ISU attended each progress meeting and offered comments on the schedule. The meeting served as a time for ISU to exchange compiled data with the Iowa DOT and receive schedule updates from the contractors. The weekly progress meetings allowed all participants to become better informed of the following: traffic, utility, and environmental impacts and any contract modifications, change orders, extra work, and activities happening in and around Des Moines that could impact the project.

Data Backup

All project information obtained by ISU including the following: plan sets, staging scrolls, specifications, schedules, meeting minutes, progress photos, schedule reviews, field reviews, and any correspondence between ISU, the Iowa DOT, or the contractors. This information was backed up electronically every week and was stored in hard copy format immediately. Hard copies and electronic copies are stored in separate areas of the building in case of fire. The redundant backup system has been in place since October of 2005.

Project Website

ISU has developed and maintained a secure website for multiple projects of the I-235 project (ISU 2007). The website was password-protected to allow access only to project personnel. The website has been updated on a weekly basis and contains the following information: preliminary schedules, baseline schedule, schedule updates, project contact information, plan sets, staging scrolls, specifications, addendum letters, progress photos, meeting minutes, and VE proposals. A screen shot of the project website is provided in Figure 5.

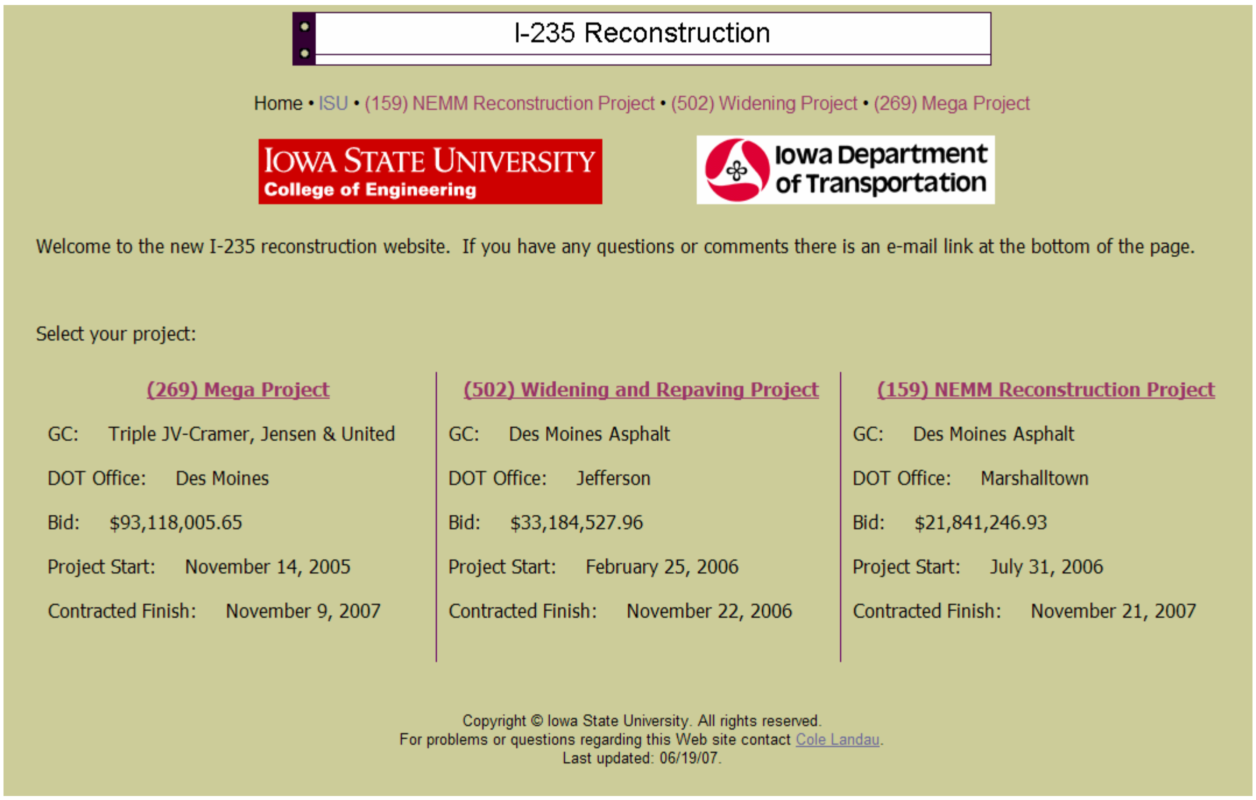


Figure 5. Screen shot of project website

STAKEHOLDER INTEVIEWS

During June 2007, the second author conducted interviews with project participants to obtain information about how the schedules were used, what parts of the scheduling process went well, and what parts could be improved. The results are still being analyzed. However, the following preliminary observations can be shared at this writing:

- The schedule was used primarily by higher level managers for the Iowa DOT, prime contractors, and major subcontractors.
- Most respondents reported the following:

  - A more complete planning effort was undertaken with greater communication amongst stakeholders as a result of applying the SPEC to this project.
  - Including the schedule as an agenda item in the weekly progress meetings encouraged effective communication and problem solving on scheduling issues.

- Iowa DOT respondents indicated the following:

  - The schedules allowed them to quickly and confidently respond to questions on the schedule from public officials, utility companies, and concerned citizens. It was especially helpful for providing updates to and answering questions from upper level managers.
  - The process of schedule updating encouraged contractors to reexamine and refine their plans on a regular basis.
  - The information from the schedule provided justification in an efficient manner to obtain necessary approvals for contractor-generated VE proposals.

- Contractors reported that the schedule provided a basis for agreements amongst subcontractors on when to work overtime and extra shifts in order to meet schedule commitments.
- It was challenging to develop agreements regarding the amount of detail necessary and how to represent logical relationships, especially the first time that a contractor group submits a schedule to the Iowa DOT. Maintaining the balance between providing required information to the contracting agency and allowing the contractors to use their own means and methods for planning and scheduling required considerable effort.
- Some scheduling tasks that were perceived as useful to the Iowa DOT were not perceived as useful by the contractors
- Some scheduling tasks that were contemplated as necessary and useful at the beginning of the project turned out to have limited usefulness during the project.

RECOMMENDATIONS

The knowledge gained by project participants in using this scheduling process may be used by others wishing to implement a similar system on future projects. Based on the experiences described in this paper, the following recommendations are made:

- Continue to use the SPEC on challenging projects where schedule compliance, coordination, and stakeholder communications are deemed especially important.
- Prioritize tasks required by the specification in terms of benefits to project stakeholders and consider adding or deleting tasks. On future project, modify the specification according to the needs of the project.
- Plan to expend considerable effort at the beginning of the scheduling effort to communicate contracting agencies’ needs and contractors’ preferences regarding the scheduling effort.
- Develop standard procedures to monitor scheduling efforts and retain records or refine the ones described in this paper.

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Several Iowa Department of Transportation, Iowa State University, and contractor employees worked closely with the authors in this scheduling effort. Not all of them can be mentioned here; however, the following from each organization were most closely involved with the material developed in the paper:

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Regulatory Compliance and Ecological Performance of Mitigation Wetlands in an Agricultural Landscape

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ABSTRACT

The success of wetland mitigation projects nationwide is typically assessed by comparing the total number of wetland mitigation acres attained to the total number of mitigation acres required by Section 404 permits. In the absence of performance measurements on mitigation wetlands, the success of compensatory mitigation in replacing the ecological value of impacted wetlands is increasingly questioned by wetland scientists. This study focuses on evaluating regulatory compliance and ecological performance of mitigation wetlands in Iowa. Regulatory compliance was determined by comparing delineated wetland areas to permitted losses and by evaluating completeness of permit conditions at 24 randomly selected Iowa Department of Transportation wetland mitigation sites. In a separate study, intensive biological inventories were used to evaluate ecological performance at 12 mitigation and 3 reference wetlands. Species richness and abundance data were collected on algae, protozoa, aquatic invertebrates, butterflies, amphibians, reptiles, birds, and mammals at each site. Species richness and diversity at mitigation sites and reference sites were compared to determine whether mitigation wetlands are performing differently than reference wetlands in Iowa. The results are valuable for building and expanding the tools and knowledge necessary to effectively assess and manage the ecological performance of compensatory mitigation wetlands and improve the ecological effectiveness of wetland mitigation.

Key words: agricultural landscape—ecological performance—regulatory compliance—wetland mitigation

An Evaluation of the Relationship between the Seat Belt Usage Rates of Front Seat Occupants and Their Drivers

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ABSTRACT

Death as a result of ejection of unrestrained occupants from the vehicle is the highest cause of fatalities in a motor vehicle crashes. In 2002, in the United States, about 70% of unrestrained occupants fatal motor vehicle crashes were ejected and killed. Data from the National Highway Safety Administration (NHTSA) show that overall seat belt usage rates in the United States are showing an increasing trend. However, a review of the literature did not reveal any prior studies regarding whether the seat belt usage of a driver affects the seat belt usage of passengers in the same vehicle. This paper provides a summary of an analysis to identify the seat belt usage rates of front seat occupants in a vehicle based on the seat belt usages of their drivers for three vehicle types: sedans/station wagons, pick-up trucks, and vans/SUVs. The results show that irrespective of the vehicle type, the seat belt usage rates of front seat occupants change significantly with the driver's seat belt usage characteristics. When the drivers use seat belts, there is a significant increase in the overall usage rates of front seat occupants, whereas when the drivers do not use seat belts, there is a substantial decrease in front seat occupants' seat belt usage rates. However, in general, the seat belt usage rates by drivers have a bigger impact on the seat belt usage rates of passengers in pick-up trucks than in sedans or vans. This indicates that media campaigns and enforcement campaigns could be effective by focusing on drivers' seat belt usage, and this would increase the seat belt usage rates of front seat occupants as well. Equally, strategies to increase seat belt usage by occupants (e.g., education) are likely to lead to increases in seat belt usage by drivers. This result also means that by observing seat belt usage rates of drivers, the seat belt usage pattern of front seat occupants could be estimated.

Key words: occupant protection—safety—seat belt

INTRODUCTION

Records for the year 2002 from the National Highway Traffic Safety Administration (NHTSA 2005) indicate that motor vehicle crashes are the 8th leading cause of death among all ages that year. However, crashes were ranked first among the different causes of death for every age from 3 through 33 (NHTSA 2002). Seat belts are intended for use by passengers as a restraining device and aimed at reducing the severity of injury to occupants of a vehicle involved in a crash. Proper use of seat belts also helps reduce the potential for ejection from the vehicle and to reduce the impact of occupant contact with the vehicle interior or other objects. Evans (1987) showed that unbelted driver involvement rates in fatal crashes were 28% to 86% higher than those for belted drivers for seven types of traffic accidents/crashes. Steptoe et. al (2002) described seat belt usage as one of the most effective methods of reducing injury in motor vehicle crashes. Depending on the type of vehicle and seating position, the proper use of seat belts can significantly improve the chance of surviving a potential fatal crash (Blincoe et. al. 2002). NHTSA and state offices of traffic safety invest significant resources to improve seat belt usage rates by occupants of motor vehicles. Data from NHTSA show that overall seat belt usage rates in the United States show an increasing trend (NHTSA 2007) However, a review of the literature shows a need to evaluate the relationships between seat belt usage of driver and that of passengers in the same vehicle. This paper summarizes findings of a study to identify the relationship between seat belt usages of drivers and passengers for two conditions: when the driver is wearing a seat belt and when the driver is not using the seat belt. Seat belt usage rates for four years from 2003 to 2006 at 50 sites across the state of Nevada are used for this study.

OBJECTIVE

The objective of this paper is to identify whether a relationship exists between seat belt usage rates of the driver and passengers in the same vehicle. In other words, it aims to determine if any differences exist in the seat belt usage pattern of front seat occupants when drivers (a) use a seat belt and (b) do not use a seat belt. The results of this study would greatly benefit safety advocates and decision makers by focusing on drivers and passengers separately in education and enforcement campaigns in order to attain higher seat belt usage rates.

METHODOLOGY

The study that forms the basis of this paper used observational surveys to obtain data regarding seat belt usage rates of drivers and passengers in vehicles. Trained observers made field observations regarding seat belt usage rates at locations where traffic speeds were slow enough to make these observations. The data collection and site selection aspects of the study follow.

Data Collection

Data required for the study were collected by field observations at 50 locations across the state of Nevada. The data were collected for the years 2003 to 2006. At each site, a minimum of 400 vehicles were observed for seat belt usage rates of drivers and front seat passenger. In total, over 20,000 observations were made each year. On an average, 33.6% of the observed vehicles had occupants present in the front seat. The data were further categorized based on gender and age group of passengers, vehicle type, and area type.

Site Selection

The sites for data collection include 50 locations across the state of Nevada. The locations of these sites were determined using NHTSA's guidelines for "State Observational Surveys of Seat Belt Use" (NHTSA 2000). They were distributed based on functional classification proportionate to the statewide annual vehicle miles of travel. The 50 sites are at locations where stopped or slow moving traffic can be observed.

Types of Analyses

In order to determine relationship of seat belt usage between drivers and passengers, the drivers and passengers are identified based on their gender. Vasudevan and Nambisan (2005) show that the seat belt usage rates change based on vehicle type. Three types of vehicles are considered: sedan/station wagon, pick-up truck, and van/SUV. Analyses are performed based on vehicle type to identify relationships of seat belt usage between drivers and passengers of these vehicle classifications.

SUMMARY OF ANALYSES

Vasudevan and Nambisan (2003, 2004, 2005, and 2006) showed that the seat belt usage patterns of drivers and front seat passengers vary based on vehicle category and area type. Therefore, the analyses presented herein include the relationships of seat belt usages for drivers and front seat passengers for the following vehicle types: sedan/station wagon, pick-up truck, and van/SUV. The analyses are divided into two major sections: when drivers use a seat belt and when drivers do not use a seat belt. They are described in detail in this section. Table 1 shows the seat belt usage rates for different categories of front seat occupants, viz. male and female passengers for different vehicle type, without considering driver's seat belt usage. Figure 1 compares seat belt usages of passengers to that of drivers for the years 2003 to 2006. Figure 1 shows that, in general, the seat belt usage rates of passengers are higher than the usage rates of drivers. Figure 2 summarizes the seat belt usage rates of passengers in comparison to drivers' seat belt usages. This figure clearly shows that the front seat occupants' seat belt use is dependent on seat belt usages of drivers.

Table 1. Average seat belt usage rates for front seat passengers

Year	2003			2004			2005			2006		
Category	Passengers With SB	Total # Passengers	% SB Usage	Passenger With SB	Total # Passenger	% SB Usage	Passenger With SB	Total # Passenger	% SB Usage	Passenger With SB	Total # Passenger	% SB Usage
Sedans/Station Wagons												
Male	740	966	76.6%	795	917	86.7%	1,031	1,112	92.7%	1,594	1,661	96.0%
Female	1,730	2,031	85.2%	1,764	1,923	91.7%	1,999	2,068	96.7%	1,687	1,779	94.8%
All	2,470	2,997	82.4%	2,559	2,840	90.1%	3,030	3,180	95.3%	3,281	3,440	95.4%
Pick-ups												
Male	469	752	62.4%	458	605	75.7%	595	655	90.8%	651	792	82.2%
Female	631	799	79.0%	634	701	90.4%	837	872	96.0%	521	563	92.5%
All	1,100	1,551	70.9%	1,092	1,306	83.6%	1,432	1,527	93.8%	1,172	1,355	86.5%
Vans/SUVs												
Male	519	672	77.2%	480	548	87.6%	929	970	95.8%	845	915	92.3%
Female	1,069	1,237	86.4%	1,154	1,239	93.1%	2,043	2,071	98.6%	1,584	1,651	95.9%
All	1,588	1,909	83.2%	1,634	1,787	91.4%	2,972	3,041	97.7%	2,429	2,566	94.7%
All Vehicles												
Male	1,728	2,390	72.3%	1,733	2,070	83.7%	2,555	2,737	93.4%	3,090	3,368	91.7%
Female	3,430	4,067	84.3%	3,552	3,863	91.9%	4,879	5,011	97.4%	3,792	3,993	95.0%
All	5,158	6,457	79.9%	5,285	5,933	89.1%	7,434	7,748	95.9%	6,882	7,361	93.5%

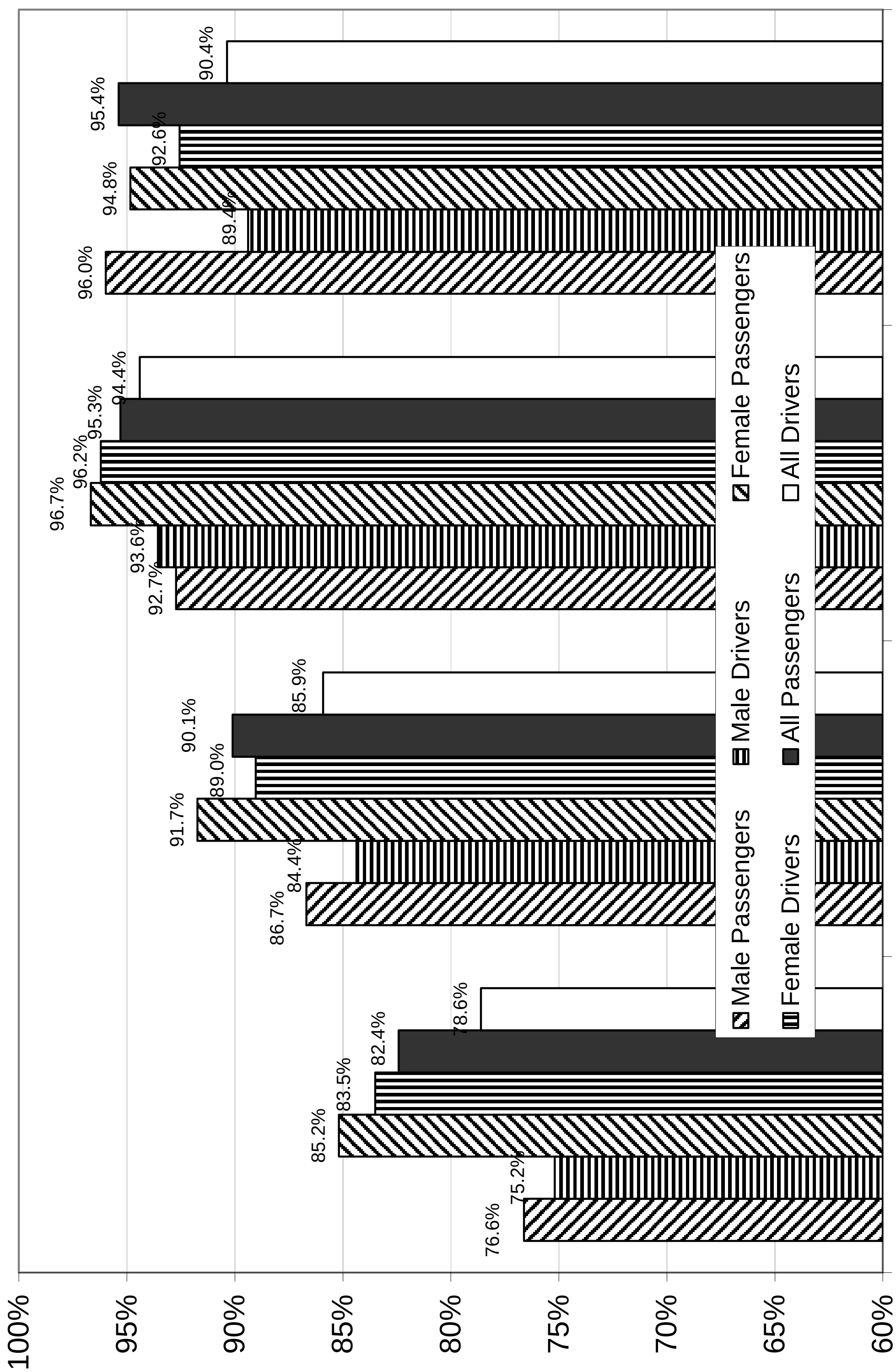

Figure 1. Comparing overall seat belt usages of passengers and drivers (2003–2006)

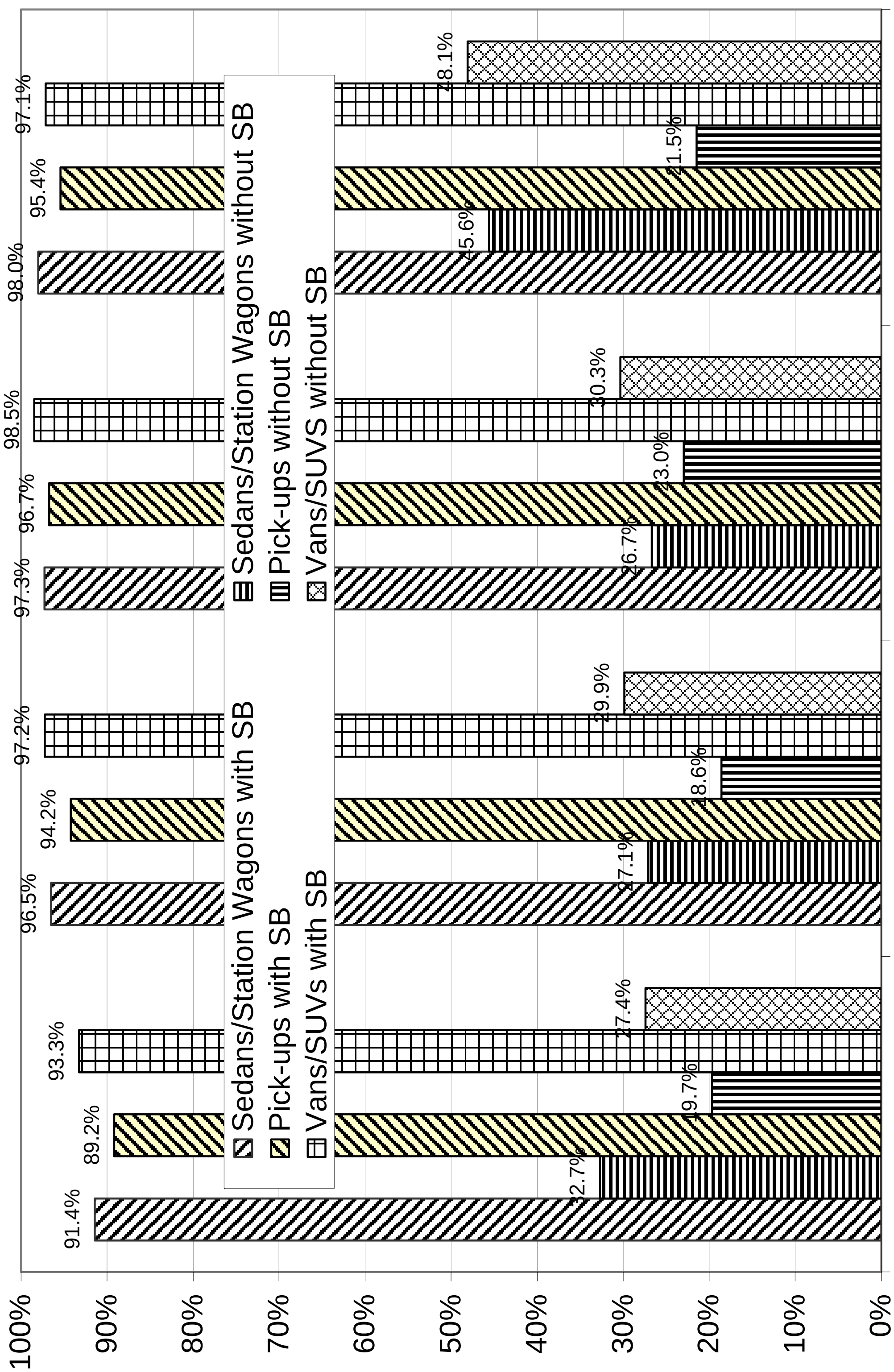

Figure 2. Comparing seat belt usage rates of front seat occupants based on drivers' seat belt usage

Occupants' Seat Belt Usage Based on Drivers' Seat Belt Usage

Next, data pertaining to the seat belt usage of occupants are compared to that of drivers when the drivers wear seat belts. These data are again divided based on vehicle types.

For All Vehicles: Statewide Data

Figure 3 shows the summary of seat belt usage rates of front seat passengers when drivers use and not use seat belts, respectively, for all vehicles types for the years 2003 to 2006. Here, it is seen that when drivers use seat belts, the seat belt usage rates for front seat occupants over the four years is over 95%, except for the year 2003 when it was 91.5%, whereas when the drivers do not use seat belts, the corresponding seat belt usage rates of passengers decrease considerably to about 26%, except for the year 2006 when it was 37.8%.

For Sedans/Station Wagons

Figure 4 summarizes seat belt usage rates of passengers when drivers use and do not use seat belts, respectively, for sedans and station wagons for the years 2003 to 2006. Here, it is seen that when drivers use seat belts, the seat belt usage rates for front seat occupants for the four years is over 95%, except for the year 2003 when it was 91.4%, whereas when the drivers do not use seat belt, the corresponding seat belt usage rates decrease considerably to about 30%, except for the year 2006 when it was 37.8%.

For Pick-up Trucks

Figure 5 shows the summary of seat belt usage rates of passengers when drivers use and do not use seat belts respectively for pick-up trucks for the years 2003 to 2005. It is noted that when drivers use seat belts, the seat belt usage rates for front seat occupants for the four years is over 95%, except for the year 2003 when it was 89.2%. Conversely, when the drivers do not use seat belts, the corresponding seat belt usage rate by passengers drops noticeably to about 18% to 23%.

For Vans/SUVs

A summary of seat belt usage rates of passengers based on the drivers' use of seat belts for sedans and station wagons for the years 2003 to 2006 is shown in Figure 6. It can be seen that when drivers use seat belts, the seat belt usage rates for front seat occupants for the four years is over 95%, except for the year 2003 when it was 93.3%. As in the cases of sedans/station wagons and pick-up trucks, the seat belt usage rates of passengers drops significantly to about 30% when the drivers do not use seat belts (except for the year 2006 when it was 48.1%).

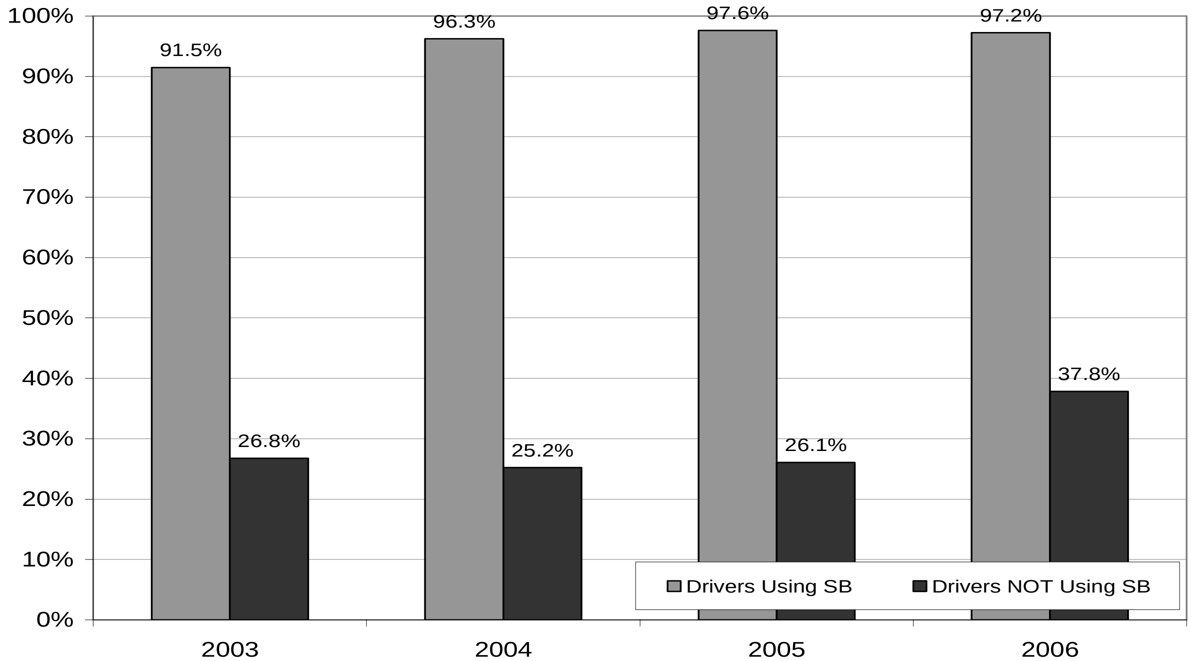

Figure 3. Comparison of seat belt usages of passengers based on drivers' seat belt usage (all vehicles)

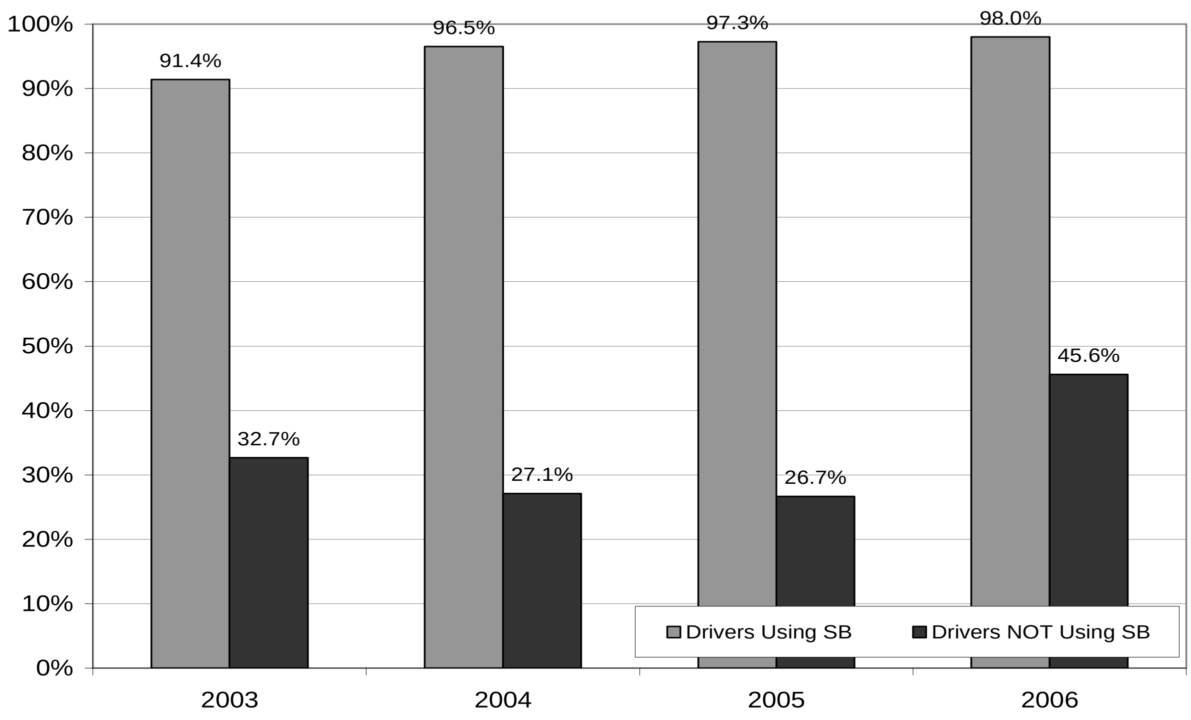

Figure 4. Comparison of seat belt usages of passengers based on drivers' seat belt usage (sedans/station wagons)

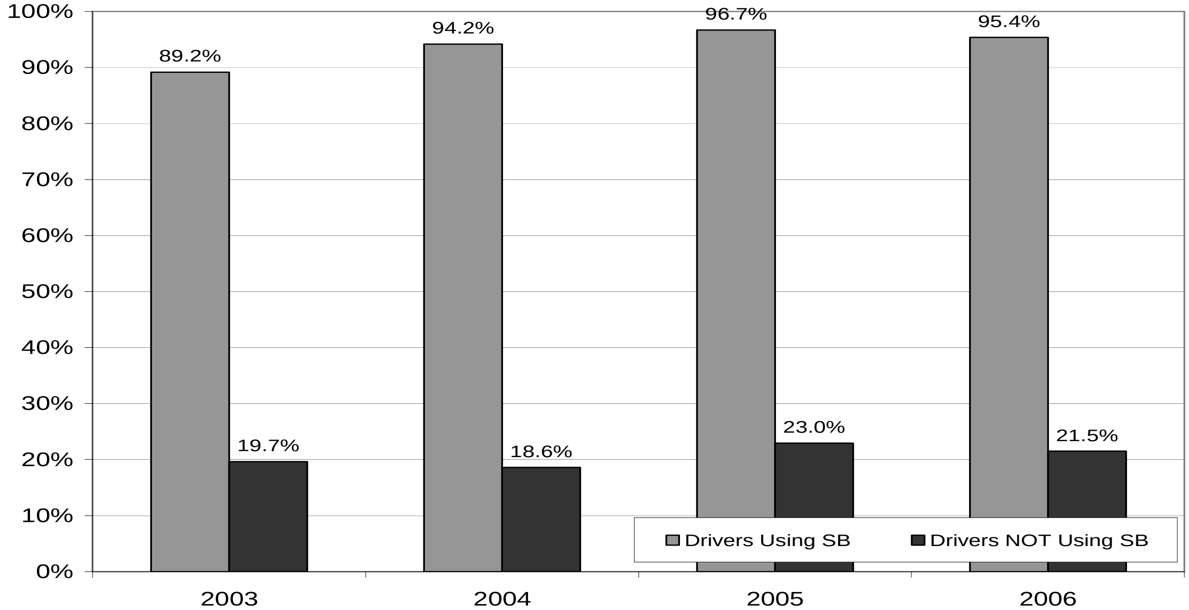

Figure 5. Comparison of seat belt usages of passengers based on drivers' seat belt usage (pick-up trucks)

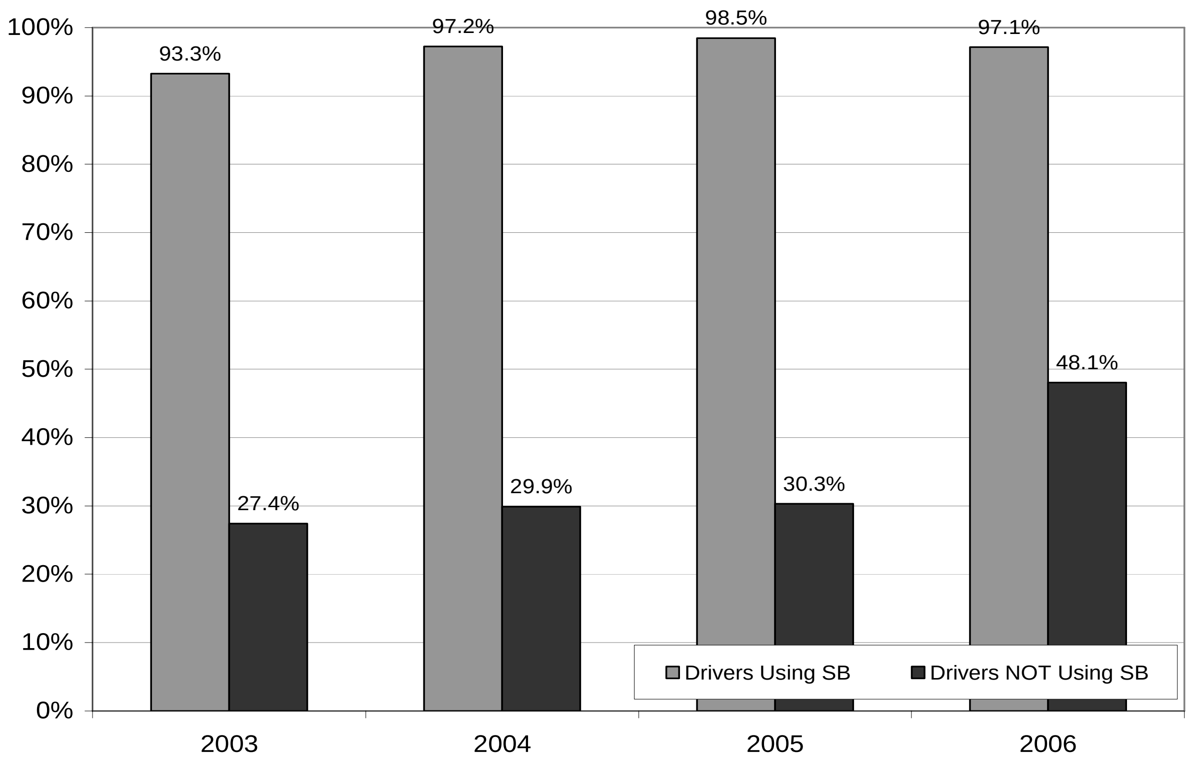

Figure 6. Comparison of seat belt usages of passengers based on drivers' seat belt usage (vans/SUVs)

DISCUSSION

Figure 2 to 6 show that the seat belt usage rates of front seat occupants are closely linked to the use of seat belts by their respective drivers. This finding is very important, since it shows that if the drivers do not wear seat belts, the chance that the front seat occupants also do not wear seat belts is significantly high. This means that strategies to enhance seat belt usage would benefit significantly by focusing on drivers. Conversely, education strategies that encourage occupants to wear seat belts (e.g., in elementary and middle schools) could have positive influences on the use of seat belts by drivers. This correlates very well with anecdotal evidence of children asking their parents to buckle up before they start driving. It also indicates that by observing the seat belt usage of drivers, front seat occupants' seat belt usage behaviour could be accurately estimated.

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Using Crash Data as a Measure of Effectiveness in Evaluating Signal Timing Optimizations along Corridors

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ABSTRACT

Orth-Rodgers and Associates (ORA) has been working with the Freeway and Arterial System of Transportation of the Regional Transportation Commission of Southern Nevada (RTC FAST) on corridor signal timing optimization projects. The traditional measure of effectiveness (MOE) for the performance of corridor optimization projects is the changes to travel time, delay time, number of stops, and average speed along the corridor. These measurements are quantified through calculations of cost savings to the drivers in terms of time and maintenance and on the reduction of air pollutants. The MOEs are determined by conducting and comparing before and after travel time and delay runs.

ORA and RTC FAST are conducting research to determine if crash data analysis could be used as an additional MOE. The research will compare crash data for similar time periods before and after the signal timing optimization project improvements were installed on the corridor. The corridor projects selected for this research will be several miles in length, will have ten or more signals, and will have had the new signal timing plans operational for at least two years, but preferably three years.

This presentation will discuss the results of the crash analyses conducted on at least four corridors in the Las Vegas metropolitan area. Included in the presentation will be brief overviews of these corridors and the performance evaluations included with the project report. This information is useful for further justifying corridor-level signal timing optimization projects.

Key words: corridor signal coordination—crash mitigation—traffic safety

Monitoring the Performance of Timber Bridges over the Long Term

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ABSTRACT

Timber bridges are often viewed by engineers as less durable than steel or concrete structures. However, they remain a durable and economical option along secondary roads in many rural areas of the country. With well over 75,000 bridges listed by the Federal Highway Administration's National Bridge Inventory as having timber superstructures as of 2002, they represent about 15% of the nation's highway bridges. Many of these timber bridges have been in service for more than 50 years. Knowledge of their long-term performance characteristics will allow maintenance and repair practices to be optimized and for future designs to incorporate improved design details for extending the service life of timber bridges. This paper will discuss long-term data collection goals, overview current work, and outline sensor types/features needed for obtaining reliable data required to successfully monitor timber bridges.

Key words: bridge—moisture—performance—sensors—timber—vibration—wood

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INTRODUCTION

The Federal Highway Administration (FHWA) recently studied the effectiveness of the bridge inspection program within the National Bridge Inventory (NBI) (Phares et al. 2001). The NBI bridge condition data have been largely based upon visual assessment techniques which have been shown to be highly variable and subjective. These visual bridge inspection techniques have served as the basis for prioritizing bridge rehabilitation and replacement funds for the past 35 years. With the nation's highway bridges getting older and their deteriorated condition diminishing their level of service, focus has shifted towards repair and strengthening techniques that extend service life. When bridge replacements are considered, there is a need for greatly improved bridge condition data to support the greater allocation of funds.

Subsequently, the FHWA initiated its Long-Term Bridge Performance Program (LTBPP) in order to gain a better understanding of the relative deterioration rates for highway bridges. The introduction of the LTBPP program signals a major shift from the routine NBI (2-year interval) inspections towards a more proactive approach employing continuous data collection of in-service bridge structures. In order to generate the required health-monitoring data, key bridge performance parameters will be tracked using state-of-the-art sensing technologies on hundreds of instrumented bridges over a 20-year period. The goal is to generate more quantitative and more reliable condition and performance data (over extended periods of service) that can better support decisions for bridge repair and replacement.

The focus of this paper will be data collection strategies to support the long-term monitoring aspects of timber highway bridges. Timber highway bridges comprise about 15% of the approximately 575,000 highway bridge in the FHWA NBI. They provide vital transportation links mainly along secondary roadways in rural areas of the country. This paper will discuss data collection goals, overview past field monitoring work, and propose data collection strategies for long-term performance monitoring of timber highway bridges.

LONG-TERM FIELD MONITORING GOALS

Acquiring reliable quantitative data pertaining to the condition of the main structural components of timber bridge systems can potentially provide insight into the performance and deterioration characteristics of the components with respect to load and environmental conditions. Knowledge of these performance and deterioration characteristics can lead to forecasting the decay and wear of the structure as well as the degradation of load capacity. If structural integrity weakening can be forecast, then preventative maintenance and inspection schedules can be developed to identify and correct the deficiencies of the timber bridge structure prior to major deterioration and/or catastrophic failure.

Sensor Attributes

Some of the key attributes for data sensors suitable for long-term timber bridge monitoring are as follows:

Low Cost. The unit cost of sensors should be relatively low to support usage on a variety of timber bridge structures, including long-span trusses/arches and shorter span timber bridges. Cost will be a factor in embedding a number of sensors in a single structure also.

Environmentally Durable. The sensors must be sufficiently rugged to withstand the rigors of outdoor exposure including effects of temperature, precipitation, and sunlight.

Reliable and Accurate. The sensors must maintain their reliability and remain sufficiently accurate in measuring various performance parameters under exposed conditions. The sensors will need to be designed to undergo a large range of temperatures and precipitation. Also, the sensors will need to have a high signal to noise ratio. The primary goal for sensors is that they remain reliable and accurate for at least 20 years.

Placement of sensors will also be an important factor in developing a successful timber bridge monitoring system. Initially, a limited number of sensors will be placed in specific locations the earliest signs of distress typically occur. A secondary goal is development of miniaturized sensors that can be embedded into bridge members built of engineered wood composites. A large array of embedded sensors will provide a global (vs. localized) indication for each data parameter. In order to achieve this goal, the sensors will need to have the following attributes in addition to those listed above.

Toughness. Sensors will need to be rugged enough to withstand manufacturing methods and be compatible with waterproof adhesives.

Power Consumption and Communication. Sensors should have very low-power requirements and be capable of transmitting data to an onsite computer control system using wireless technologies.

Key Data Parameters

The following data variables should be the primary focus of a successful timber bridge monitoring system. These monitoring data sets should be able to provide an early warning of bridge deterioration due to physical or biological factors.

Moisture Content. The moisture content of wood is the best predictor for conditions conducive to incipient (early) decay. There is a threshold level of wood moisture (approximately 22% to 24%) below which decay cannot sustain itself. Brown rot is one type of decay organism that is commonly found in timber bridge structures and can significantly reduce load carrying capacity of bridge structural members prior to exhibiting visual signs of deterioration. For glulam bridges, where the initial moisture content is less than 15%, the development of sensors that will trigger warnings as the threshold moisture content is surpassed. For sawn lumber bridges, many structures are installed with internal moisture contents in excess of 30%, and sensors should be designed to trigger warnings as they drop below fiber saturation levels.

Deflection/Strain. The behavior of a timber bridge under heavy trucks, whether static or dynamic loading, gives the best system performance data. When typical service loads are used, deflection and strain data collected a various locations can provide an indication of load distribution levels.

Vibration Characteristics. Measuring the vibration characteristics of timber bridges can provide a measure of the overall stiffness of the bridge system, without the time delay of live load testing. Forced and free vibration techniques isolate the key frequencies that can be correlated to bridge superstructure stiffness.

Bridge Distortions. Because timber bridge materials are hygroscopic, they shrink and swell as their moisture content changes. When exposed to the elements, they can undergo significant moisture-related distortions that can adversely affect the overall structural performance.

Prestressing Bar Forces. The structural integrity of stress-laminated bridges is predicated on the maintenance of sufficient force in the prestressing bars. If bar forces drop below minimum design levels, various modes of failure can potentially take place. Active monitoring of these bar forces will give a key indication of the structural performance of the bridge. In addition, some rehabilitated longitudinal deck bridges employ stress-laminated techniques which increase load capacity and these prestressing bar forces deserve similar attention.

PREVIOUS FIELD MONITORING EFFORTS

Previous efforts with regard to performance monitoring of timber bridges focused primarily on two timber bridge superstructure types: stress-laminated and sawn-timber-girder timber bridges.

Automated Data Acquisition Systems

During the 1990s, the National Timber Bridge Monitoring Program was conducted by the U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory (FPL) in conjunction with the FHWA. Nearly 100 timber bridge structures were evaluated in the field during their initial two to five years in-service. Most of the field bridges were of the new stress-laminated deck design and the field data helped support the acceptance of a guide specification by the American Association of State Highway Transportation Officials (AASHTO). A subset of approximately 12 bridges from FPL/FHWA monitoring program were instrumented with automated data acquisition systems (Table 1). A remote data acquisition system was installed at each bridge site to monitor key performance parameters for stress-laminated decks that included temperatures, relative humidity, and stress-laminating compression forces. Field data was continuously collected over 24- to 48-month periods using a Campbell-Scientific datalogger and various sensors (Figure 1). A Vaisala temperature and humidity probe was used to measure the ambient conditions beneath the superstructure. Thermocouple wires were used to measure ambient and internal bridge deck thermal variations. Hollow-core steel load cells were custom manufactured at FPL and then encapsulated within an environmentally rugged PVC casing.

Table 1. Timber bridges instrumented with automated data acquisition systems for two- to five-year monitoring periods

State	Bridge name	Nearest City	Year built
WI	Pine River	Richland Center	1991
	Moose River	Clam Lake	1996
MN	Ciphers	Warroad	1989
IA	Deans Bottom	Moulton	1993
OH	McCurdy Road	Butler	1995
PA	Dutch Hill Road	City of Titusville	1992
	Brookston Road	Howe	1992
	Laurel Run	Jackson	1992
	Jacobs	Todd	1991
	Millcross Road	East Lampeter	1992
	Dogwood Lane	West Brunswick	1993
	Birch Creek	Cherry	1992

The field performance data from many of these instrumented bridges has been reported previously. From 1991 through 1993, the Ciphers Bridge was documented in the extreme weather conditions in Northern Minnesota (Wacker et al. 1998). From 1993 through 1995, the Deans Bottom bridge performance was monitored in southern Iowa. From 1997 to 2002 the FPL acquired data from seven hardwood demonstration timber bridges that were constructed by the Pennsylvania Department of Transportation using hardwood bridge materials (Wacker et al. 2004). The comprehensive data set from all 12 stress-laminated decks further characterized the trend in bar force levels in these bridges.

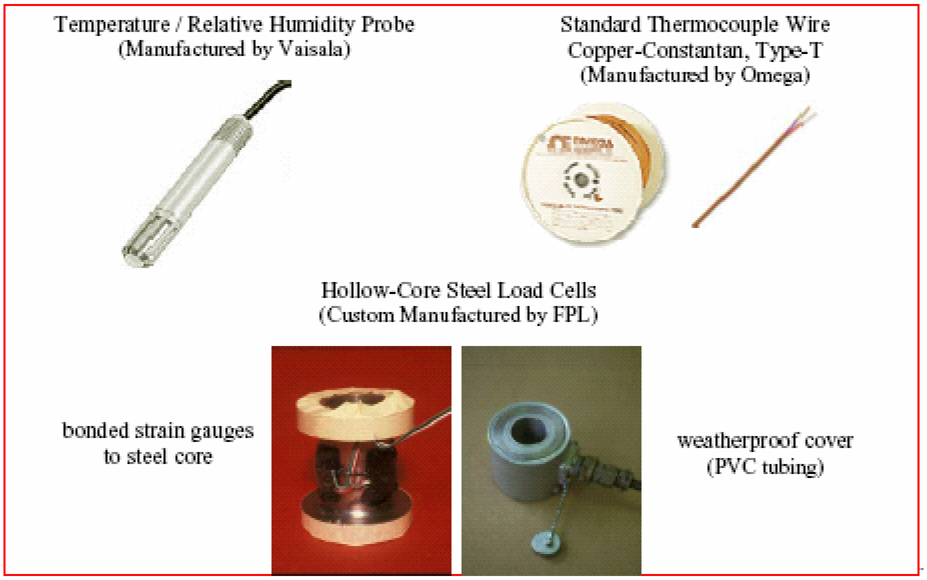

Figure 1. Typical sensors used in previous two- to five-year field monitoring studies

This data also was used to characterize any effects of cold (winter) temperatures on their structural performance. The sensors that were used in these field studies yielded acceptable data but required periodic recalibrations and other adjustments during the monitoring period.

Forced Vibration Inspection Technique

Several single-span timber bridges comprised of longitudinal sawn timber stringers with a transverse plank deck were evaluated in the field: 5 timber bridges from the Ottawa NF in Michigan's Upper Peninsula (Wang et al. 2005) and 12 timber bridges from St. Louis County, MN, (Wacker et al., in progress) were evaluated in service using dynamic testing techniques and compared with static live load test results.

A forced-vibration technique was used to identify the first bending mode frequency of the bridge structures. This method is a purely time domain method and was proposed because it eliminates the need for modal analysis. An electric motor with a rotating unbalanced wheel is used to excite the structure which creates a rotating force vector proportional to the square of the speed of the motor. Placing the motor at mid-span ensured that the simple bending mode of structure vibration was excited. A single piezoelectric accelerometer, also at mid-span, was used to record the response in the time domain. To locate the first bending mode frequency, the motor speed was slowly increased from rest until the first local maximum response acceleration was located. The period of vibration was then estimated from ten cycles of this steady-state motion. Because the primary goal of this work was to relate the vibration characteristics of the timber bridge structures to a measure of structural integrity, the bridges were also evaluated with the established method of static load–deflection field testing. Stiffness of bridge superstructure (*EI* product) could then be estimated from the field test results.

This vibration technique was intended to complement routine bridge inspections by providing an indication of the structural performance of the superstructure system. However, several factors have limited this technique for field inspection application. Focus has now shifted towards a fully automated system to measure vibration characteristics as part of a measurement and control system mounted onsite.

Figure 2. Measuring the response of a single-span timber bridge to forced vibration testing

DEVELOPMENT OF LONG-TERM PERFORMANCE MONITORING SYSTEM

Preliminary proof-of-concept laboratory studies have been completed towards the development of two sensors that measure the moisture content of wood beams. In addition, a field study to develop a fully automated, forced-vibration measurement and control system for timber bridges is underway. These new sensor systems are intended to be fully integrated with fully automated health monitoring systems mounted on a bridge.

Moisture Content of Bridge Members

Methods of acquiring the moisture content of wood beams have been explored. A freshly cut 8 in. birch log was cut into a 5 in. x 5 in. x 48 in. beam. A center line was drawn along the grain and six digital relative humidity/temperature sensors (SH-7) manufactured by Sensirion, were embedded in the birch beam and monitored over a 60-day period (Figure 3). Sensor #1 (E1) was embedded 4 in. from the edge at a depth of 2 3/4 in.. Sensor #2 (E2) was embedded 12 in. from the edge at a depth of 1 3/4 in. Sensor #3 (E3) was embedded 20 in. from the edge at a depth of 1 1/4 in. Sensor #4 (E4) was embedded 28 in. from the edge at a depth of 2 3/4 in. Sensor #5 (E5) was embedded 36 in. from the edge at a depth of 1 3/4 in. Sensor #6 (E6) was embedded 44 in. from the edge at a depth of 1 1/4 in.

To embed the sensors, a 3/8 in. diameter hole was drilled to 1/4 in. short of the sensor depth. A 3/8 in. diameter ribbed plastic sleeve with a light coating of silicon sealant was inserted into the hole. The sleeve was driven in the hole until it reached the 1/4 in. stop. Then a 5/32 in. diameter hole was drilled to the full sensor depth. The SH-7 sensors were then inserted into the sleeve until the head of the sensor passed the plastic sleeve and rested in the wood void 1/4 in. past the sleeve. A small amount of silicon sealant was placed over the top of the sleeve to keep the sensor in place and to restrict the introduction of ambient conditions into the sensor cavity.

In addition to the embedded sensors, a remote wireless resistance pin sensor, manufactured by Omnisense, was installed on the surface of the beam and monitored over the same 60-day period (Figure

4). The pin sensor measured the resistance between the pins that were threaded 1/4 in. into the beam along the center line, 24 in. from the edge. The wireless sensor is powered by a single 3/6v lithium battery. The sensor communicated with a receiver located approximately 100 ft. away in an adjacent office. The receiver is connected to the Internet, and the data transmitted from the sensor is sent to the Omnisense server where it is stored and prepared for display.

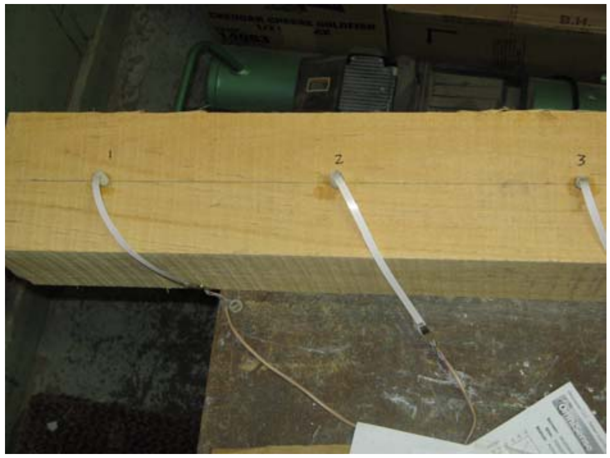

Figure 3. Embedded and surface-mounted sensors for measuring wood moisture

Figure 4. Omnisense wireless resistance pin sensor

The birch beam was placed in a controlled environment with a temperature of 70^{o} F and relative humidity of 12%. As a control reference the birch beam was placed on a digital scale, and the constant weight of the specimen was recorded for the 60-day period. The initial weight of the beam was measured and recorded, and upon completion of the test period, the beam was placed in a drying oven to obtain a dry weight.

Embedded and ambient temperature, relative humidity and dew point data was collected automatically at 30-minute intervals over the entire 60-day test period. The Omnisense surface mount wireless sensor also collected temperature, relative humidity, and dew point data, and it was collected continuously. Accessing the Sensor Activity Analysis display the data collected can be viewed over selected time periods. Daily averages obtained from the Omnisense website were used to graph the sensor's data. During testing, embedded Sensor #4 provided unreliable initial data and then failed completely. After 10 days Sensor #4 was disconnected, and no further data was collected from this point.

Analysis of the data collected indicates that the moisture content recorded by the sensors varied with the depth of the sensor. Readings from the sensors that were mounted at 2 3/4 in. and 1 3/4 in. tracked along the same trend line, and the sensors that were mounted at 1 1/4 in. also tracked together and paralleled the deeper sensors at a lower value. The values recorded by the wireless pin sensor were more linear but within the range of the values obtained from the embedded sensors.

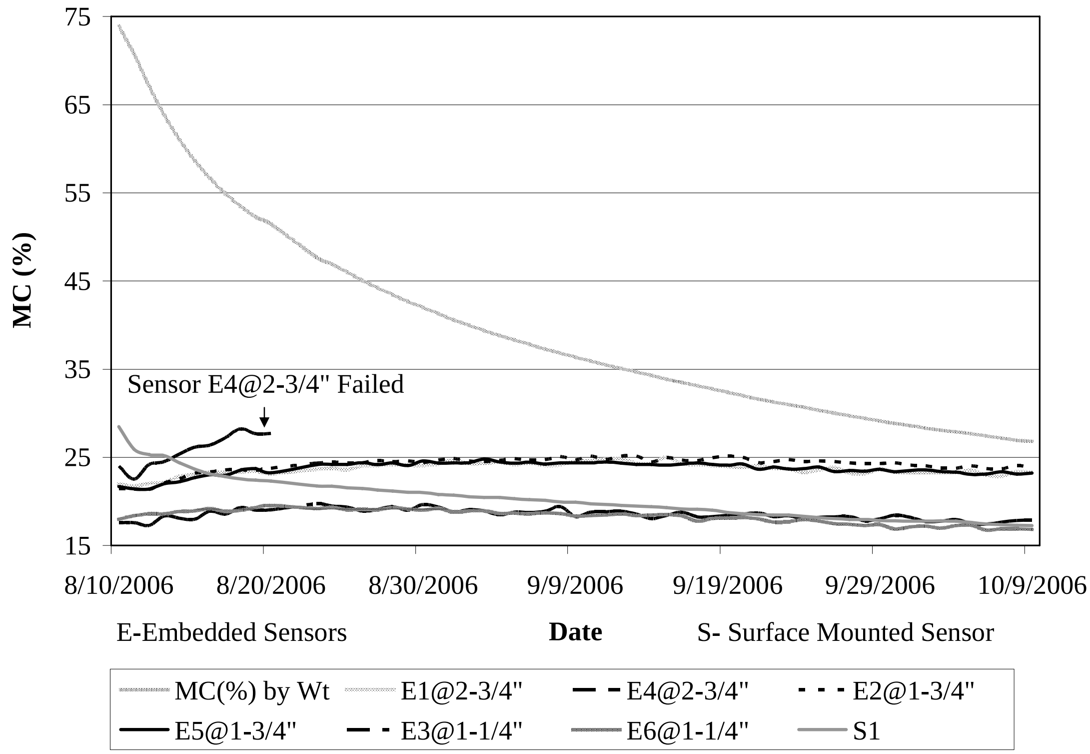

Figure 5. Average moisture content trends of embedded Sensirion SH-7 Temp/RH sensors and surface-mounted Omnisense Temp/RH sensor

Figure 5 displays the graphical representation of the MC trends for the 60-test period. The results showed that a definite trend exists based on the depth of the embedded sensors. Sensirion Sensors #1 and #4 were embedded at a depth of 2 3/4 in., Sensors #2 and #5 were embedded at a depth of 1 3/4 in., and Sensors #3 and #6 were embedded at a depth of 1 1/4 in. The laboratory results of this test indicate that depending upon the depth of the sensor there is a trend to the calculated %MC. However, sensor values are limited in the upper range by the fiber saturation point of the birch beam. The following equation from the *Wood Handbook* (FPL 1999) was utilized for calculating the equilibrium moisture content:

$$M = 1800/W\ [KH / (1-KH) + (K_1KH + 2K_1K_2K^2H^2) / (1 + K_1KH + K_1K_2K^2H^2)] \quad (1)$$

where,

M = moisture content (%)
T = temperature (^oF)
H = relative humidity (%)

and

$W = 330 + 0.452T + 0.00415T^2$
$K = 0.791 + 0.000463T - 0.000000844T^2$
$K_1 = 6.34 + 0.000775T - 0.0000935T^2$
$K_2 = 1.09 + 0.0284T - 0.0000904T^2$

Bridge Vibration Characteristics

A timber bridge in Minnesota has been instrumented with vibration motor (mounted underside) along with an onsite computer control system. The system is programmed to initiate vibration and collect pertinent data on a regular basis. The data collection is ongoing, and more details and results will be available soon.

SUMMARY

The key components of a long-term timber bridge performance monitoring program, including a description of the key performance indicators and a few data collection strategies, are presented.

Preliminary work has been completed with regard to development of wood moisture content and forced vibration measurement sensors and controls. Two electrical-resistance type sensors (one wired/embedded and one wireless/surface-mounted) were reliable and reasonably accurate in measuring the moisture content of a birch beam in the laboratory. Further study of the effectiveness of these moisture content sensors when exposed to the outdoors is ongoing. A reliable and accurate measure of the wood moisture content will predict when internal conditions of key bridge members are conducive to the development of decay deterioration.

Field studies are also currently underway to investigate the potential of an active bridge vibration measurement and control system. A viable system that measures the key vibration variables has been shown to be a good indicator of stiffness of the bridge superstructure. In addition, it may also provide key information for improving the reliability of estimating residual strength of the bridge system.

Further investigation into the use of fiber optic–based sensors is also underway. They have an advantage of minimal noise interference, which makes them a strong candidate for long-term performance monitoring applications.

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An Interdisciplinary Approach to Transportation Education

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ABSTRACT

Our current transportation system is a manifestation of the decisions made by transportation professionals in our somewhat recent past. Those decisions were influenced by the education that transportation professionals received and by their approaches to problem solving set forth by a culture imbedded in them throughout their professional lives.

We are now acutely aware of the impacts our current transportation system has, not only on our mobility and safety, but also upon the environment, disadvantaged populations, and numerous other aspects of our built and human environment.

This being said, it is important to explore new approaches to transportation education. Bringing together transportation students from various disciplines, such as engineering, planning, and public policy seems to enhance the learning experience and may potentially result in a more well-rounded transportation professional capable of influencing better transportation decision-making.

At the University of Wisconsin, Madison, and sponsored through the Gaylord Nelson Institute for Environmental Studies, is an interdisciplinary, graduate-level certificate program entitled, “Transportation Management and Policy.” Through this 17-credit program, transportation students of various backgrounds take courses together and interact with students of varying perspectives on the topic of transportation.

This paper serves as an exploration of this format of transportation education. Students from different educational backgrounds are afforded the opportunity to interact through settings that promote considerable discussion and even team efforts focused on transportation projects. Perhaps through this style of transportation education, the vocabulary and approaches to problem solving associated with the “cultures” of the various transportation disciplines might be meshed, thus resulting in a well-rounded transportation professional.

Key words: education—engineering—interdisciplinary—planning—transportation

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BACKGROUND

Our nation's transportation system is a complex one. It consists of many modes, owned by many parties, travels through many jurisdictions, and impacts many people, communities, businesses, and even ecosystems. Because our transportation system is such a complex one, the planning, design, construction, and operation of such a system requires a vast number of skilled professionals working together towards a common goal of an efficient transportation system that moves people and goods safely and effectively.

These professionals come from a number of different backgrounds. Planners determine need through collection of data, detailed analysis, and communication with the public and politicians to begin the project development process. Engineers design and oversee the construction and operation of the infrastructure. Environmentalists provide input during the NEPA process. Real estate specialists get involved during property appraisal and acquisition. Financial analysts prepare budgets and track expenditures throughout the process. Public affairs professionals coordinate political efforts and administer the funds. This is only a partial description and barely scratches the surface of the disciplines involved but makes the point that the players often come from very different professional and educational backgrounds.

Table 1 provides a description of how various disciplines contribute to each phase of the transportation project development process (Faucett 2003).

This description of disciplines and their responsibilities is an extensive one, though not entirely exhaustive (especially if you consider modes other than highway). Some of the other disciplines not mentioned but that are still integral to the overall successful operation of a multimodal transportation system include researchers, law enforcement officers, vehicle and system operators, and managers of port authorities, among others.

While it is absolutely critical for these parties to be involved to provide a transportation system of high quality, getting these parties to work together can be a very real challenge. This is a challenge not because they do not get along, but because they often do not truly understand one another. This difficulty in communication is rooted in the fact that each discipline has their own vocabulary and approaches to problem solving that are part of their discipline's culture, or as Hugh Petrie calls it, a discipline's "cognitive map," i.e., the cognitive and perceptual approach connected to a discipline. The longer a practitioner operates within a discipline, the more engrained this cognitive map becomes. It can get to the point where, "…quite literally, two opposing disciplinarians can look at the same thing and not see the same thing." (Petrie 1976; Hall and Weaver 2001)

For example, when looking at the problem of a congested segment of urban highway, an engineer may suggest expanding the highway to include another lane. The urban planner may attempt to employ demand management strategies such as car pooling, parking restrictions, and telecommuting to alleviate the peak hour volumes. The transit operator will likely suggest encouraging commuters to switch their trips to bus or commuter rail. The traffic control engineer may recommend the use of ramp metering and incident management control measures. Meanwhile the environmentalist is in favor of whatever strategy is least likely to adversely impact air and water quality and nearby sensitive ecosystems.

Table 1. Role of disciplines in each phase of the project development process

Phase	Description and Disciplines Involved
Early Planning (Long Range)	Mostly transportation planners and some engineers, traffic and environmental study personnel participate in preliminary assessment of transportation needs and the potential impacts of solutions that address those needs.
Project Programming and Budgeting (setting priorities)	Mostly engineers, planners and some financial staff with expertise in budgeting are involved in this process. Decisions are influenced by top ranking public officials, politicians, and special interest groups.
Preliminary Design and Engineering	Mostly engineers and surveyors are involved in consultation with the environmental study personnel, as well as architects, geologists, hydrologists and others when applicable.
Environmental Studies	Involves mostly personnel from the disciplines of engineering, planning, biology, archeology, and historic preservation, but may also involve personnel with expertise in real property acquisition (i.e. property appraisers) and relocation of families, businesses and institutions. Other disciplines potentially involved include architecture, landscape architecture, geologists, and lawyers.
Final Design	Mostly engineers who may consult as necessary with those involved in environmental studies (i.e. to address the compliance with environmental commitments or assess mitigation alternatives not previously considered) and real property acquisition.
Property Acquisition and Relocation Assistance	Mostly involves acquisition and relocation assistance personnel such as property appraisers and relocation assistants, but may involve personnel with expertise in civil rights and other areas.
Construction	Mostly involves engineers and engineering assistants, but also involves personnel with expertise in construction safety, accounting, and other disciplines mentioned above (i.e. those with expertise in disciplines associated with environmental studies particularly in cases where environmental commitments are in effect).
Operations and Maintenance	Mostly involves personnel with expertise in transportation facility maintenance, traffic counting, auditing, and other disciplines. For cases in which environmental or other commitments are in effect, personnel with expertise in those fields will be employed, as applicable.

Having different view points should not be considered a bad thing. It is true that it can be a struggle to find a solution when you have so many competing interests, but it is these competing interests that help to ensure all angles are examined, or at least considered, when attempting to find a solution to such a problem.

Often time there is no one right answer. It might be that the solution is a combination of the suggestions above. But if we did not have these groups working together at various points in the project development process, solutions considered best for all involved may have been missed. In order to find these solutions, the multitude of disciplines involved in the process must be able to communicate. In order to communicate effectively, we must have foundational understanding of where each other is coming from and why we each think and talk like we do.

So the question being begged here is, “How do we begin to understand one another?” “How do we learn to communicate more effectively?” Perhaps we need to begin this learning process before we become so immersed in our own subcultures. A natural place to start seems to be in college. It is during this time that transportation professionals begin to take on the vocabulary and approaches to problem solving that makes an engineer and engineer, a planner a planner, an environmentalist and environmentalist, and so on.

The idea for this paper came from a discussion with a former student in the Transportation Management and Policy (TMP) program at the University of Wisconsin, Madison. She was completing her master’s degree in civil engineering and working part time with an engineering consultant. Her supervisor put a blueprint in front of her with engineering specifications regarding the construction of a new roadway. Before becoming involved with the TMP program, she said she would have simply reviewed the engineering specifications for correctness and moved on. However, now that she had been exposed to the other disciplines in the transportation field and had attended class with transportation students from other disciplines such as planning and public policy, she now found herself taking a more holistic view of the project. She found herself asking questions such as, “How would this impact the community it’s running through?” “Does a facility of this kind and capacity fit the needs of the users?” “Who benefits from this project, and who is adversely impacted by its construction?” These are the questions transportation professionals, project managers, and decision-makers should be asking themselves.

BENEFITS OF INTERDISCIPLINARY EDUCATION

It is a difficult proposition to find quantifiable benefits of an interdisciplinary education. Not only would a method for assessing, or measuring, the benefits of an interdisciplinary education need to be developed, but there would need to be a way to track the student into the work force, and again, measure the benefits to the professional and the transportation system with which he or she interacts.

Qualitative benefits can be claimed. In a project report stemming from the Interdisciplinary Studies Project at Harvard Graduate School for Education, the author notes, “More than building a factual knowledge base, the emphasis of an interdisciplinary program seems to be on developing ‘bend of mind,’ or meta-thinking skills, that allow students to remain learners and seekers of information.” “[G]raduates of the program are prepared to become expert sifters, searchers, and synthesizers of information.” (Nikitina 2002) Students of interdisciplinary educational programs not only have the skills to remain focused on the technical details of their respective disciplines but gain the ability to see the big picture.

Connors and Seifer (2005) examine models of interdisciplinary service learning and reiterate this phenomenon: “university students report the development of key skills and attributes resulting from their participation in interdisciplinary service-learning, including gaining the ability to think beyond traditional academic disciplines and being more adept at integrating and applying what they are learning. …[I]ntroduced in the early years of study, [these skills] can help to foster the contribution of positive attitudes about working in communities and teams.”

David Plazak, Associate Director for Policy at Iowa State University’s Center for Transportation Research and Education, notes the benefits of an interdisciplinary education, “The real world is interdisciplinary and [is] becoming more so all the time. For students to function well in the real world, they need to understand what other professionals and technicians do and the type of value they add to projects. Students with interdisciplinary experience are more versatile and more flexible, and this is of benefit to their employer. I hear this all the time from firms and agencies that have employed our MS TRANS graduates."

Steven Polzin and Beverly Ward (2002) used a focus group consisting of 12 (transportation) professionals (at the level of vice president or director) from area firms and public agencies to assess industry support for an interdisciplinary transportation program. All were in agreement that an interdisciplinary transportation degree, though a non-engineering degree, would be valued by their organizations. There also seemed to be a consensus that the interdisciplinary approach would help satisfy a need for a workforce with a broader perspective needed in management-level positions. It should be noted that this group was almost entirely engineers.

APPROACHES TO INTERDISCIPLINARY EDUCATION

This paper is not an inventory of transportation degrees and certificates currently being offered. Nor is it an assessment or an evaluation. This section of the paper simply takes note of a few of the programs that currently exist around the country. These programs were located by starting with the U.S. Department of Transportation's listing of University Transportation Centers at http://utc.dot.gov/.

One could argue that a student gains significant benefit simply by sitting in the same classroom as students from other transportation-related disciplines. Students appear to gain insight into the perspectives of other disciplines through discussion with these peers and hearing the questions asked by these peers. In a transportation planning course at the University of Tennessee, a civil engineering student asked the question, "Why do we even need to involve the public in the transportation decision-making process?" This was a valid question and probably not one most engineers in the class had given too much thought to before. The professor turned to the planning students in the room and asked for them to provide an answer to the question, which they did, since public involvement is an essential part of the planning process that every planning student, even in his or her first semester of graduate school, is keenly aware of.

A similar situation occurred during a discussion in a course at the University of Wisconsin, Madison. The instructor posed the scenario of funding a bypass around an urban center. The planning and public policy students were quick to attack the idea for its many potential environmental and socially disruptive side effects. An engineering student pointed out advantages, such as safety and mobility, that this kind of facility might provide. All valid points. Still, these points may not have been heard in a classroom with students of the same discipline.

At the Massachusetts Institute of Technology (MIT), the Department of Civil and Environmental Engineering offers a Master of Science in Transportation (MST). The interdisciplinary degree, which typically takes students two years to complete, consists of two core courses – one focused on an introduction to transportation systems using a "softer" approach, while the other is a more technical analysis of the modal systems and tools used to operate and analyze these systems. Other course requirements include building a depth of understanding in a select area of interest, such as one of the transportation modes like air transport or ocean systems; that depth could also be attained through coursework in planning, policy, logistics, or management. Or, rather than focusing on depth, the student may choose to take coursework in numerous areas and build a broader understanding of transportation. Since research is at the heart of the program, a research-based thesis is also required.

Though this program is located within the School of Engineering, it brings in students with backgrounds in the physical and social sciences, urban planning, management, as well as engineering. All that is specifically required for admission are two courses in calculus and one each in economics and probability. The requirement for two courses in calculus appears to be unique to MIT and likely reflects MIT's reputation as a leader in technical education. Dual degrees are also an option with pairings available in MST/Master of Science in the Technology and Policy Program, MST/Master of Science in Operations Research, and MST/Master of City Planning.

Based on a preliminary review of the MIT program, it appears that the second option, where a student is able to build a broader understanding of transportation and the disciplines involved, is more likely to develop the skills and perspective that are desirable of a well-rounded transportation professional. This is the type of professional an interdisciplinary transportation education program seeks to develop.

The University of South Florida (USF) houses the Graduate Interdisciplinary Transportation Program (GITP) for graduate students of civil engineering, economics, and public administration. Here, graduate students enrolled in one of the three departments take a common set of core courses that emphasize urban transportation issues (as this is the theme of the National Center for Transit Research (NCTR), which has a close tie to the GITP), while pursuing either a Master of Science in civil engineering, Master in Civil Engineering (directed towards professional engineering practice), Master of Arts in economics, or a Master of Public Administration. Regardless in which of the three departments the student resides, he or she must complete a core of interdisciplinary courses in transportation engineering, transportation planning, urban economics, microeconomics, policy analysis, and urban planning.

While transportation courses must be taken as a part of the core courses, students pursuing a Master of Arts in economics offered within the College of Business Administration, and the Master of Public Administration offered within the College of Arts and Sciences, do not seem to be required to take any additional transportation-focused courses. However, students pursuing the Master of Science in civil engineering or the Master of Civil Engineering are required to take additional transportation-focused courses outside of the core courses. Again, this appears to be the case based on a preliminary review of the program. The Master of Science in civil engineering is a research degree and requires a six credit transportation thesis and seven credits in transportation engineering electives. The Master of Civil Engineering requires a three credit transportation project and seven credits in transportation engineering electives.

The GITP is now being offered as a six-course certificate program. The certificate was developed for early and mid-career transportation professionals in response to a need expressed by the profession for increased training in interdisciplinary approaches to transportation issues. The certificate may also eventually be offered statewide through USF's distance learning system.

Iowa State University too has a Master of Science degree in transportation. This 36 credit degree, with a required thesis or creative component related to transportation, is an interdisciplinary degree offered under a cooperative arrangement with three departments including Civil and Construction Engineering, Community and Regional Planning (in the College of Design), and Logistics, Operations and Management Information Systems (in the College of Business).

In addition to requiring the student to take courses in each of the three departments, he or she must also take core courses in statistics, urban transportation planning, economic analysis of transportation investments, and a seminar in transportation planning. This seminar is similar to the one offered as a part of the Transportation Management and Policy certificate program at the University of Wisconsin, Madison.

Admission requirements include an undergraduate degree in a transportation-related field such as business, planning, engineering, psychology, sociology, government, etc. It is not certain what the specific requirements are as some of these fields can only be considered transportation-related only if a focused set of coursework is taken.

As one can see, the programs do vary, though each affords students from various disciplines the opportunity to interact with one another in a classroom setting. These are only three such examples, and one can be certain that many similar interdisciplinary transportation programs exist with varying levels of connection between the disciplines both programmatically and in the classroom.

UNIVERSITY OF WISCONSIN, MADISON, APPROACH: THE TRANSPORTATION MANAGEMENT AND POLICY PROGRAM

History of TMP

Teresa Adams, program chair and champion of the TMP program, explains, “We looked at educational needs in this area and discovered that they go beyond just civil engineering. Technical issues are part of it, but the context in which we deliver transportation systems has changed dramatically, so now we’re concerned with the environment, social justice, political issues and the not-in-my-backyard syndrome.” (Adams 2003)

Teresa Adams, along with other faculty at the University of Wisconsin, Madison, and within the Midwest Regional University Transportation Center (MRUTC), realized the need for a program such as the TMP to train these well-rounded transportation professionals. Research into the existence of programs like the TMP note that graduate schools routinely offer transportation-related courses through departments of engineering, business, urban planning, public policy, and geography. Some graduate schools offer special tracks in transportation within one or more of these traditional disciplines; however, few of these programs provide a curriculum reflecting the essential interdisciplinary nature of transportation management and policy.

In order to address this need, an multidisciplinary Academic Advisory Board made up of faculty from the School of Business, La Follette School of Public Affairs, Department of Civil and Environmental Engineering, Department of Agriculture and Applied Economics, the Law School, the School of Human Ecology, Department of Urban and Regional Planning, and the Gaylord Nelson Institute for Environmental Studies oversaw the development of the TMP program with encouragement from the state and federal transportation agencies and the Wisconsin transportation industry. The MRUTC, at the University of Wisconsin, Madison, led the initiative. A series of meetings with the Academic Advisory Board of the MRUTC and the Land Resources Program Committee were held between May 2001 and May 2002 to develop and approve the TMP certificate program.

TMP Curriculum

In 2002, the Transportation Management and Policy Certificate program was born. It is a 17-credit graduate-level certificate program that welcomes applications from students in any graduate program at the University of Wisconsin, Madison. However, the program is especially well suited for students with academic backgrounds in business, economics, engineering, environmental studies, land management, public affairs, and urban planning.

Every TMP student must take a set of core courses, which consist of the Practicum in Transportation Management and Policy and the Colloquium in Transportation Management and Policy. The practicum is a three-credit course that must be taken once during the program, while the colloquium is worth one credit and must be taken at least twice. Both of these courses will be discussed in further detail later in this paper.

Beyond the core courses, each TMP student must take one course in each of the focus areas. These areas include technology/engineering, economics, policy/management, and environmental. Each focus area lists two to three recommended courses but the student can, with approval from the TMP Chair, take courses outside of the recommended ones. However, the substituted course must adequately satisfy the intent of the of the focus area in which it's being substituted. These recommended courses are offered by multiple departments and schools at the University of Wisconsin, Madison, including civil engineering, urban and regional planning, economics, La Follette School of Public Affairs, operations and technology management within the School of Business, and the Nelson Institute for Environmental Studies.

In addition to the course requirements, each TMP student must also complete a transportation-related internship equivalent to at least 120 hours. Many TMP students complete the requirements of the internship by working at the Midwest Regional University Transportation Center (MRUTC). While working in the center, students are often involved in activities such as marketing for the center where he or she might assist in putting together the annual report, newsletters, research summaries, or brochures. Students working for the MRUTC also have opportunities to assist principal investigators associated with the MRUTC with research, which may lead to thesis topics.

Practicum in Transportation Management and Policy

One of the two core courses required of TMP students is the Practicum in Transportation Management and Policy, which is offered in the spring of each academic year. This course puts students into interdisciplinary teams of three to six and tasks them with providing solutions to transportation issues solicited from various industry partners. Practitioners from these agencies serve as liaisons and are a consistent information resource for the student teams. At the end of the course, students prepare a white paper and present a PowerPoint to the senior managers within the participating agency(s).

This practicum course accomplishes several objectives. It provides students with real-world experience in the analysis of transportation issues. It gives them experience communicating the results of their work to industry managers. And, critical to this discussion, it provides the students with experience working with teams of people from other disciplines who have different talents, vocabularies, and approaches to problem solving.

The spring of 2007 marked the fifth spring in which this course was offered. The transportation topic areas that students have been assigned to work on thus far have included developing an alternative to the fuel tax (students focused on the vehicle miles traveled tax), a project examining what factors should be considered in programming capital maintenance, a project analyzing a range of options for keeping the aging population safer on the roadway (students looked at traffic signing and information to improved transit service to keep them mobile), and an evaluation of how Wisconsin's comprehensive planning law was achieving its goals.

Other projects have tasked practicum students to analyze the Wisconsin Information System for Local Roads (WISLR), including how the system was used and how it had impacted decision-making at the local government level; an economic analysis of the impact ferry systems have on their respective communities in Wisconsin; an analysis of the statewide travel demand planning model; and a documentation of best practices for freight planning at the Metropolitan Planning Organization level.

Through these many team-oriented transportation projects, students have been able to interact within and experience the interdisciplinary nature that is characteristic of transportation.

Colloquium in Transportation Management and Policy

The other required course is the Colloquium in Transportation Management and Policy. This is a weekly seminar course where invited speakers from transportation agencies come in and present their perspectives and case studies on transportation management and policy. The colloquium course is held in the fall and spring of every academic year, and each TMP student is required to take this course at least twice in order to receive the certificate.

Each semester's colloquium has a theme. This theme is reinforced through the careful selection of practitioners who are invited to present. The history of thematic areas for the colloquium, going back to the Fall of 2002, include transportation management, finance and economics, environmental impacts of transportation, transportation and land use, transportation safety, politics of transportation, public transportation, non-motorized transportation options with a focus on pedestrian and bicycle facilities, freight transportation, ethics and social responsibility in transportation, and transportation history and law.

Future thematic areas for the colloquium include local and regional transportation management to be offered in the fall of 2007, and a likelihood of revisiting the topic of environmental issues in transportation in the spring of 2008. The idea here is to take advantage of a connection with the Nelson Institute of Environmental Studies and their Community Environmental Forum. Through this cooperation, the TMP will be able to bring in some new, exciting speakers and many industry representatives to further engage the colloquium students.

As with the practicum, students of various educational (and sometimes professional) backgrounds, but with a similar interest in transportation, are brought together in the same classroom. Though these students are not asked to work together on a transportation-related project as they are in the practicum, they seem to benefit greatly simply by being exposed to, and interacting with, students from the other disciplines as is evidenced by the student who inspired this paper.

The colloquium is an informal setting where the practitioner presents his or her information, or data, and students are encouraged to engage the presenter and other students in dialogue. These informal discussions go a long way towards familiarizing the students with their peers and their differing perspectives, vocabularies, and approaches to problem solving. Hearing some of the questions their peers are asking of the presenter seems to stimulate interest in dialogue that students often times wouldn't experience with like-minded students from the same disciplines.

CONCLUDING NOTES

As transportation professionals, we often assume a certain level of knowledge of transportation history possessed by all of us. As noted above, the theme for the spring 2007 Colloquium in Transportation Management and Policy was Transportation History and Law: Setting a Context for Today's Transportation Challenges. The students were assigned to write a memo demonstrating their understanding of how the lessons learned from transportation history impact today's transportation system. One student working on his PhD in civil engineering states, "I always believed that somewhere in the past, planners took a decision to build the interstate system the way it is now and had it built. It turned out I was wrong. Planners were never involved in the decision making process. If I had not been sitting in a class like this with engineers and planners discussing issues and history on the same platform, I would never have realized my misconception. Hence the importance of multidisciplinary education in the field of transportation cannot be stressed more."

In that same memo, students were asked to pay specific attention to the interaction among the various disciplines in transportation and to provide personal observations on the perceived effectiveness of an interdisciplinary transportation education program. Though a few students failed to address this aspect of the memo, no students regarded the interdisciplinary aspect of the classroom setting with negative sentiment. All students suggested that success of the transportation system depends on the effective collaboration among disciplines and that beginning this collaboration during the educational process is a step in the right direction.

Another student, working on a dual degree on civil engineering and law, writes, "the sooner that engineers, planners and others work together [such as] in a classroom setting, the greater the understanding of each other's disciplines will be. Engineers can better anticipate planners concerns, and vice versa. Future projects will rarely, if ever, be successful without the interaction of many different groups and this interaction will only be successful when its leaders can speak each others' 'languages'."

A particularly eloquent urban and regional planning student writes, "It is important for decisions, which have such resounding effects upon the urban form to be made with due regard to all likely positive and negative results. An educational program that integrates planners, engineers, and social science students in a classroom setting allows for discussion and debate of real-world transportation topics is important so that all can realize the limitations of their own scope and the biases affecting their chosen field. Planners, who may have a strong aversion to highway construction, need to be aware of the strong incentive that an efficient highway system provides to transportation-reliant companies. Likewise, civil engineers, who may be committed to creating roadways which will move the highest number of vehicles most efficiently, benefit from seeing the way that high traffic roadways can negatively impact the lives of those who reside near them." "An educational environment that fosters this ideological cross-pollination between disciplines yields the dual benefits of generating transportation professionals with a well-rounded knowledge base as well as creating a culture of collaboration that will, hopefully, continue into the future so that past gaffes, which resulted from overly narrow perspectives, can be avoided in the future."

At the time the Transportation Management and Policy program was established at the University of Wisconsin, Madison, there was a lack of similar interdisciplinary transportation programs in the United States (Adams 2003). In fact, it wasn't until the 1950s and 1960s that transportation engineering programs began to surface at the collegiate level. (Transportation Education and Training Committee 2000) This no longer seems to be the case. Interdisciplinary transportation programs of varying approaches are emerging all over the United States. This can be discovered by performing a simple Google search.

Though no quantitative research results have been published to support the benefits of an interdisciplinary transportation education, the qualitative and anecdotal findings seem to suggest there is a great deal of benefit to be gained from the meshing of disciplines in a classroom or team-oriented project setting. Outside of the transportation sector, it appears that a majority of the research performed on the benefits of and obstacles to the success of an interdisciplinary environment occur within the medical field, specifically in oncology. Here too, the research suggests there are many positives outcomes from such an interdisciplinary environment but only if each of the disciplinarians are technically solid within their discipline and open to the idea of working in a team environment. (Petrie 1976; Hall and Weaver 2001)

As the noted American poet and critic Mark Van Dorn once stated, “The student who can begin early in life to think of things as interconnected, even if he revises his view every succeeding year, has begun the life of learning.” So is the process of interdisciplinary education. It is no longer sufficient to view transportation education as just a series of college courses. It is, and will continue to be, multidisciplinary and a life long endeavor. (Transportation Education and Training Committee 2000)

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Review of Demand Modeling Methodologies for Air-Related Transportation: An Institutional Challenge to Intermodalism

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ABSTRACT

Fundamental to intermodal development is consistent demand modeling across the intermodal system, which coordinates resource allocation. However, different planning organizations are responsible for the development of different components of the intermodal system. In the case of air transportation, air demand modeling is mainly conducted by Federal Aviation Administration and commercial airlines for air traffic control and air flight operations. Airport-related surface demand modeling is undertaken primarily by airport authorities and metropolitan planning organizations for airport design and urban planning. These agencies adopt different methodologies from different perspectives, and there is an institutional gap between the methodologies, which may undermine the well being of the intermodal transportation system. In this paper, methodologies for demand modeling in the airport-related intermodal transportation system are reviewed, and the relevant problems are identified. The authors highlight a great need for a consistent demand modeling in support of intermodalism.

Key words: air demand modeling—intermodal transportation—urban planning

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INTRODUCTION

Air related intermodal transportation has been attracting attention. A major component of this system, air transportation, has grown to be an indispensable part of both the global economy and social interaction among a large number of people around the world. Air transportation has been developing very quickly over the past decades. Further, the number of air travelers is projected to double in this decade (Nettey 2000). Much of the projected growth is expected to concentrate disproportionately at large commercial service airports in major metropolitan population centers, and this growth will generate additional service demands on airports, which are the point of transition between air and surface modes of transportation for both passengers and cargo. As such, Nygard (1999) notes that “airport administrators are faced with implementing a heavy program of improvement and construction to meet the rising demand for facilities and services in an efficient manner.”

Growth in air transportation imposes a challenge to urban transportation planning as well. An impact of this growth in air transportation is its contribution to traffic congestion in the areas surrounding the airport. Therefore, much literature has been devoted to how infrastructure is planned to improve access to airports. Khan (1996) envisions a change from conventional airports to intermodal-oriented airports in the future. Coogan (1996) highlights a deep interest in improving the intermodal connection between the aviation and ground transportation systems for passengers. He further points out that, in both Europe and the United States, the problem of improving the quality of ground access to airports is getting attention from policymakers at the highest levels. Improved intermodal connections for passengers in such advanced airports as Charles De Gaulle in Paris and San Francisco International in the United States demonstrate that the airport congestion problem is being tackled on both sides of the World. Roth (1995) discusses incremental high-speed rail issues to relieve airport-related traffic congestion. The author believes that rail has a strong role in the new era of intermodal transportation, prompted by the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). Clearly, there has been a worldwide effort towards improving the quality of intermodal connections at airports (Coogan 1997).

All the abovementioned efforts towards air-related intermodal transportation focus conceptually on infrastructure improvements only. However, fundamental to infrastructure planning is demand modeling. In our research, we have found that air-related demand modeling methodologies currently in use are not consistent between different planning organizations, which may create dislocated resource allocation activities between the different components of the larger intermodal system. Organizations with different responsibilities develop methods from different perspectives, and therefore a need exists for coordination to overcome this institutional gap. Surprisingly, studies on consistent demand modeling across the intermodal system have never been conducted, according to the published literature. To begin addressing the problem in this paper, we will present a brief review of the methodologies for air-related demand modeling from different organizations, which will highlight the models’ different perspectives and the discrepancies among them. Our focus is not to go into detail about each method and its scientific soundness. Instead, we will show the differences in their application and perspectives wherever possible to highlight the need for methods that can help coordinate the development of different components of the intermodal system.

The organization of this paper is as follows. We first cover the methodologies and concepts of air-related surface demand modeling. We then review the application of a four-step method to airport-related demand modeling using two major urban planning organizations in Texas. We also briefly mention the opinions of two metropolitan planning organizations (MPOs) that concurred with our observations. Finally, we will compare the methodologies before highlighting a need for an integrated demand modeling method.

REVIEW OF DEMAND MODLEING METHODOLOGIES AND PRACTICES

In this section, we will discuss airport-related demand modeling and urban transportation planning practices, and we will ultimately make a brief comparison of these practices before proposing a systems approach for computer simulation.

Airport-Related Demand Modeling

In the following, we will briefly review the methodologies that are directly or indirectly related to airport demand modeling.

Directly Related Methodologies

We have identified four main focuses of research, all of which deal directly with air-related travel demand from different perspectives.

Focus I. The first research focus is the problem of airport travel demand estimation inbound to and outbound from a particular metropolitan area. While this has not been directly covered in the literature, the need for air travel demand forecasting in urban areas has long been identified, and small-scale closely related problems have been studied. In Kaemmerle (1991), for example, the research estimated the demand for scheduled commercial passenger services in small communities in the United States. This research was motivated by the change in air travel demand patterns in light of the Airline Deregulation Act. Data were collected describing the social, economic, and geographic characteristics of 260 small communities in the contiguous 48 states, with airports enplaning at least 2,500 passengers in 1985. Small communities were defined as those with service area populations of 200,000 or less. Descriptive data included population, income, labor force characteristics, community economic base, geographic location, departures, airfare per mile, driving distance to an alternative airport, and attractiveness of driving to an alternative airport. A methodology for selecting the most probable alternative airport when choices are present was included in the study. Although this study focused on demand estimation for small communities, it provided insight into the nature of air travel demand generation.

In addition, this research (Kaemmerle 1991) also specified multiple regression models using the collected data to estimate enplanements. Good results were reported. The best model explained 80% of the variation in the data. The model was demonstrated by estimating the demand for air service at 52 small communities in the state of Texas. In addition, this method was designed to be applied easily in the field, and only readily available U.S. Census Bureau and Official Airline Guide data are required.

Along the same lines as Kaemmerle (1991), Flaming (1994) specifically dealt with the economic variables that contribute to regional air travel demand. A model was developed and estimated using data for seven states in the southeastern region of the United States over a period from 1975 to 1987. Interesting results were obtained that suggested that demand is relatively inelastic with respect to manufacturing shipments, tourism expenditures, and statewide flight departures. It is, however, more responsive to changes in the value of manufacturing shipments than to the changes in tourist expenditures. Other things being equal, states with major connecting hubs are likely to experience a significant increase in passenger enplanements.

Focus II. Quite different than the method used in Kaemmerle (1991) and Flaming (1994), another research focus is on capturing the temporal relationship between the upcoming demand and the historical traffic patterns, namely the time series method. This method is adopted in many practices for short-term demand forecasting. In the airline industry, this method is generally for the purposes of revenue management and re-fleeting for recovery from irregular operations. It is also adopted in short-term airport operations planning. The major advantage of this method remains its power to explain periodic effects, including seasonal and weekly phenomena. The general trends that follow economic development are also explainable by this method. As an example of the literature in this area, Sen (1985) uses monthly data on ridership and revenue, one of the most commonly available types of data. A technique was presented based mostly on such data for determining the effects of various factors on ridership and revenue and for forecasting demand under various pricing assumptions. The method in Sen (1985) consisted of first smoothing the data using running medians, and then examining the smoothed data for patterns, using data from Indian Airlines.

As a complement to Sen (1985), Karlaftis et al. (1996) presented a methodological framework for air-travel demand forecasting. In particular, an analytical framework for developing economic models was presented, and post-fact analysis was used to test the accuracy of the models. The models developed were applied to two international airports, Frankfurt and Miami International. Results show that simple models with few independent variables perform as well as more complicated and costly models. Results also show that external factors have a pronounced effect on air travel demand.

Focus III. To our knowledge, the only literature to date explicitly dealing with the fluctuation of demand for airport facility planning is by Odoni and Neufville (1996). The study demonstrates the application of stochastic programming techniques to demand modeling. Though the study had airport planning as its focus, researchers in urban planning can benefit from the general concept as well. As stated in Odoni and Neufville (1996), typical design procedures can be summarized as follows:

1. Forecasting traffic level for peak hours
2. Specification of level of service standards
3. Flow analysis and determination of server and space requirements
4. Configuration of servers and space

Although airport design has a different focus from that of urban transportation planning, both need an accurate forecast of air travel demand, and both need ways to deal with the forecasting errors. Such a forecasting procedure normally first estimates aggregate traffic for the target year for which a new plan or design is developed. This aggregate forecast, in turn, is converted into a further estimate of traffic for the design day, normally taken to be the 30th or 40th busiest day of the year, or something such as the average weekday of the peak month. This is done partly based on historical data.

Note that target year is arbitrary, generally a round number, and that the use of conversion factors assumes that the pattern of traffic over twenty years or so is predictable, contrary to the predominant experience. As indicated by Odoni and Neufville (1996), forecasts are, in any case, demonstrably inaccurate. This has been shown by retrospective analyses comparing forecasts to what actually occurred (U.S. Office of Technology Assessment 1982; Ascher 1978; De Newfville 1976). According to Odoni and Neufville (1996), the 6-year forecasts of the U.S. Federal Aviation Administration (FAA) over the years have been more than 15% to 20% different from reality about half the time. The 11-year forecast for 1981, for example, overestimated the number of flight services by about 119%, and the 7-year forecast miscalculated by 70%. The situation usually gets worse for the more detailed forecasts commonly generated in the standard design process for airport terminal building. In response to the inaccuracy of the forecasts, the authors concluded that it makes more sense to concentrate on professional effort in investigating the implications and effects of uncertainties. Thus, the design effort should create a set of scenarios, with plausible ranges both for the levels of traffic and for key parameters that affect the design. According to Odoni and Neufville (1996), a few airport planning studies do, in fact, already use scenarios with broad estimates of traffic; so far, however, these are exceptional.

Focus IV. Another important method is called unconstrained demand modeling. This is related to an understanding of the difference between demand and traffic. Traffic is a realization of demand subject to network capacity constraints and other constraints. People often take the forecasted demand based on the current travel behavior as the true travel demand. In fact, current travel behavior just reflects a constrained demand. There has been interesting research on untruncation of demand in the airline industry. The reason may be that untruncation of demand is more significant in the airline industry than in any of the other areas, based on the major airlines that are able to study the demand elasticity with respect to air fares across different markets. Demand untruncation is also important to airport planning. One such effort can be seen in King County, WA (2006). The following is the basic concept adopted by King County International Airport, one of the nation's busiest general aviation airports. Unconstrained demand is a projection of what would be predicted as a demand for airport services from a variety of aviation market segments if the airport did nothing to encourage or discourage future demand. After a conceptual plan is identified, the unconstrained demand is adapted to reflect the projections of actual use, based on the proposed conceptual development plan. This projection of actual use is the constrained demand forecast. The constraint is the limitation of the airport to actually serve the entire projected unconstrained demand. As obvious as it is, the task to untruncate the demand is not an easy one. The major challenge is how to infer the concealed preference based on the revealed behavior. Reliability is also questionable. Regarding this front, there has been little published literature to date.

We have summarized research in four representative areas, including airport-related demand forecasting based on social economic characteristics in small communities, demand forecasting by time series, demand forecasting with scenarios (a simple stochastic method), and unconstrained demand forecasting. Though each of the areas is full of theoretical challenges, there is obviously a need to study how inconsistencies in these methods adopted by various organizations adversely affect an effective intermodal transportation system planning.

Other Related Research

Some other research studies and methodologies are indirectly related to air-related demand forecasting. However, these researches may provide insight into the nature of air travel demand. Part of the review that follows is based on the information from Horowitz and Farmer (2000).

Recreational Travel. As early as 1963, recreational trips were considered an important enough travel purpose to warrant separate study (Crevo 1963). In fact, in the late 1960s and early 1970s, the NCHRP (Ungar 1967), Indiana (Matthias and Greco 1968), Kentucky (Deacon et al. 1973), and other states (Gyamfi 1972; Berg et al. 1976) all conducted studies of the special characteristics of recreational travel. Strangely, although Americans seem to have dedicated an increasing amount of time to pursuing recreational activities, the latest of these studies was published several decades ago. Since many state economies depend heavily upon recreational activities, it would seem that this trip type might be important enough to require a closer examination than it has received in the past decades. For example, a Wyoming study (Wilson and Wang 1995) has indicated that peak traffic demand in that state is closely associated with recreational travel. The importance of recreational travel brings into question the appropriateness of using a small set of urban trip purposes in statewide models. In most cities, if not all, airport travel demand is believed to be strongly related to recreational travel to or from an urban area. The special characteristics of recreational travel starting with or ending at the airport, to our knowledge, have yet to be explicitly considered.

Intercity Passenger Demand Models. Intercity travel demand literature is mentioned here because air travel is basically part of intercity travels. Statewide models, in general, have not made effective use of the considerable amount of literature (largely from academic sources) on intercity passenger demand. Intercity models can essentially be divided into four types on the basis of two categories: data and structure. The models can use either aggregate or disaggregate data, and can have a direct demand or sequential structure. Intercity travel demand models can be further classified by whether they encompass only a single mode (mode-specific) or multiple modes (total demand) and by which trip purposes they include. The earliest intercity models were of the direct demand type, which were developed in the 1960s as part of an examination of the Northeast Corridor (Koppelman et al. 1984). The most famous one of these was Quandt and Balmol's (1966) abstract mode model. Readers may also refer to the review by Koppelman et al. (1984) for a more complete historical perspective of significant intercity demand modeling efforts. Particularly interesting contributions to direct demand modeling have been provided by Yu (1970), who introduced regression coefficients that include a time series component; by Cohen et al. (1978), who attempted to eliminate unmeasured effects with a pivot point technique; by Peers and Bevilacqua (1976), who introduced a long list of policy-sensitive variables; and Kaplan et al. (1982), who developed the passenger-oriented intercity network travel simulation (POINTS) model, a multimodal model that explicitly considers accessibility to the transportation system. Disaggregate models typically use a logit formulation to provide a convenient way of including a number of mode-abstract, transportation accessibility, policy-related, and behaviorally based variables in the modeling process. These models were thought to be especially useful in the effort to estimate the shifts in mode share that were expected from deregulation of the air and intercity bus industries, and from the anticipated implementation of high-speed rail transportation (Brand et al. 1992; Buckeye 1992). Again, Koppelman et al. (1984) review many of the earlier disaggregate mode choice models. In addition, Miller (1992), Forinash (1992), and Forinash and Koppelman (1993) provide studies of the various structures (binomial, multinomial, and nested multinomial) available to more realistically represent the cross-elasticities between modes and to eliminate irrelevant alternatives in the logit mode-split formulation.

Intercity Air Travel Elasticity Models for Service Design. This type of research models consumer fare product choices and estimates the elasticity of international business and leisure passengers to fare product attributes. Airline elasticity can be viewed as a summary measure of individual purchasing decisions. By modeling individual choices, different airline elasticity estimates can be developed for groups that share similar characteristics. Through incorporating individual, trip, and fare product attributes, the model can assess an airline's elasticity by different market segments in response to a wide range of marketing strategies. Marketing research and advanced travel demand and forecasting methods are typically integrated. Proussaloglou and Koppelman (1995; 1999) are examples of this effort. A more recent study by You (2001) examined passenger upgrade from a low fare class to a higher fare class when the booking request is rejected, while previous research had assumed that the rejected demand is lost.

Intercity Air Travel Demand Forecasting by Flight. This research develops models of total intercity air travel volume and its allocation to flights based on departure and arrival schedules, routing, fares, market presence, and carrier reputation and performance. This type of model is used to predict future passengers and revenues for different service scenarios. Some of the research in this area can be seen in Morrison and Winston (1985), Koppelman and Hirsch (1991), and Koppelman (1989). Worthy of a note is that almost all major airline companies make significant efforts to improve their ability to forecast itinerary-based air travel demand for the purpose of fleet assignment and revenue management. Much work has been done in this area. A good understanding of the literature in this respect helps with the understanding of the characteristics of air travel demand.

Air transportation is one of the travel modes between city pairs, and it becomes more dominant with longer distances. With large cities such as Houston and Dallas, the amount of peak-hour traffic around the airport that is attributable to tourist activities remains a question. It remains a serious theoretical problem to combine the air travel demand forecasting methodologies in a consistent, coordinated, and reliable framework for airport-related demand modeling that covers urban, airport authority, and aviation planning and that promotes the healthy development of intermodalism.

Airport-Related Ground Transportation Demand Forecasting

Four-step Demand Modeling Method

A four-step method, which includes trip generation, trip distribution, mode choice, and traffic assignment, is primarily designed for long-term urban transportation planning and is generally adopted by metropolitan planning organizations (MPOs). There is no explicit explanation in the literature about how to specifically deal with crucial situations, such as large commercial airports that experience a higher-than-average growth in travel demand. Therefore, in practice, there is a large operational flexibility in dealing with airport-related demand.

As two examples of this method, we examined the practices of the Houston-Galveston Area Council (HGAC) and the North Central Texas Council of Governments (NCTCOG). In both cases, the airport is typically treated as a special trip generator, as seen in the pertinent Texas Transportation Institute report (TTI 1985) and the NCTCOG's website (2007). In the case of the NCTCOG, base-year data is used to adjust the *k*-factor to make the forecasted traffic match the observed patterns. This method virtually raises the trip rates of the local residents (home-based work and home-based nonwork trips) evenly across the zones to compensate for trips made by visitors from outside the region. There is no detailed information about how airport-related travel demand is modeled in the practice of the HGAC. As can be seen, airport-related demand modeling is too coarsely aggregated into the general urban planning process in some practices.

In addition to these passenger concerns, a serious concern in urban transportation planning is regarding airport-related freight traffic. Urban transportation planning typically does not have the capability to explicitly deal with air freight transportation. However, air cargo has been developing quickly in the past years. It is ignored in air travel demand modeling because freight is typically transported in the belly of passenger airplanes and does not increase air traffic. In ground transportation, however, the case is different. The significance of freight transportation to traffic and air pollution should be examined carefully.

Concurrence from Two MPOs

Questionnaires were made to both NCTCOG and HGAC for their opinions on the errors associated with airport-related demand modeling. The former considers the unusual weekday travels by airline pilots and stewardesses an important part of airport traffic while the latter points out that the trips made by people who are not residents or employees of the region are present but difficult to account for. This survey confirms our observation that (1) urban planners do not pay much attention to coordinated demand modeling with other organizations, such as the FAA and airlines, and (2) current urban demand modeling itself needs to be improved to account for the growth of air travel demand in support of the intermodal system.

Remarks

We have reviewed the methods for air and ground travel demand modeling, and inconsistencies have become obvious between the methods adopted by the planning groups. Air transportation planners, including airport authorities, the FAA, and commercial airlines, have as their focus historical data regarding air traffic between regions, while urban transportation planners rely on the social and economic characteristics of the local residents in the planning area. The former emphasizes the temporal relations between regions, while the latter carefully considers the travel preferences per household. Each of the methods shows advantages and disadvantages. Urban transportation planners fall short of the capability to explicitly incorporate the visits by outside travelers and the capability to incorporate air freight transportation; air transportation planners typically do not consider the change of land use and socioeconomic characteristics in the target area at all. Air transportation planners also implicitly assume a relatively stable land use policy and a stable but consistently changing local economy. On the other hand, research on intercity travel offers more opportunity to look at other alternative modes and the dynamics between the modes. In a word, this inconsistency in demand modeling between and within each of the components of the intermodal system is likely to bring about uncoordinated resource allocations and thus undermine the well being of air transportation–related intermodalism.

Proposed Systems Approach for Integrated Demand Modeling

Each component of the intermodal system has different types of demands. For example, the demand for travel to and from the airport, which is typically modeled by MPOs, includes demands by airport employees, airline pilots, and stewardesses, as well as the air passengers, while the aviation system only models the air passenger demand. The demand for air travel shared by each component should be consistent.

As has become evident, coordinating demand modeling across the intermodal system is a fairly complex task. This is justified by the fact that each component of the intermodal system has some extra demand specifically on its own facilities. Because there has been no literature devoted to the coordination of the demand modeling across a large intermodal system, ad hoc analysis could be an option to consider. For this purpose, we propose an integrated microsimulation system that spans the surrounding urban area, airport terminals, and airlines operations, as in Yu et al. (2002), which is beyond the scope of this paper. This system is currently under careful consideration for development. We hope that through this system some insights can be obtained in the near future.

CONCLUSION

With the fast growth of air transportation, air-related demand modeling becomes fundamental to coordinated resource allocation activities between air and ground and between different components of the ground system. In this paper, we have reviewed some methods for air travel demand modeling and airport-related urban travel demand modeling. We have covered disaggregate methods, based on social and economic characteristics designed for small communities; time series methods, based on historical data; stochastic methods, with simple scenarios; and demand untruncation methods. In addition, we have reviewed some indirectly related methods that might shed some light on the characteristics of air travel demand. We further used the NCTCOG and HGAC as examples to examine how the four-step demand forecasting method deals with airport-related urban travel demand in the context of general urban planning practices. Some of the limitations that we have observed in the urban planning methodology in practice have been confirmed by these two MPOs. It is obvious that current urban planning practices do not have the capability to explicitly deal with airport-related ground transportation demand. Finally, we have highlighted the deep institutional gaps between the methodologies.

The gap between the methodologies adopted by the different planning organizations can easily lead to poorly planned resource allocation within a large intermodal transportation system that includes air and ground components. Lack of coordination between the different components could be costly. Research that examines the cost caused by the poor distribution of resources across the intermodal system is needed. Based on the results of our investigation, we specifically propose an integrated demand modeling method based on computer simulation in an effort to develop ways to overcome this institutional gap.

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What the Public Sector Needs to Know about Logistics and Freight Management

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ABSTRACT

Transportation agencies have long considered traffic volumes, crash statistics, and safety concerns in the project development process. Economic development and the concept of return on investment has now begun to find consideration in project selection as well. In many cases, however, public sector transportation planners fail to understand the business practices of the customers—the shipper, the carrier, and most importantly the transportation supply chain manager or logistics provider—to develop the necessary projects to maintain our nation's competitive economic advantage.

Researchers at the University of Wisconsin, Madison, through the Mississippi Valley Freight Coalition are currently exploring a set of guiding principles and an initial training course for public sector officials to address this concern. The research team will also provide examples of how to successfully integrate these principles into the decision making process.

Implementing sound transportation policy requires a sophisticated understanding of the marketplace and especially the changing nature of globalization and logistics management. Traditional engineering and planning courses do not incorporate these business trends, and, as such, decision makers fail to account for the transportation infrastructure needs of particular facilities. The growth of managed supply chains and the use of third (and fourth) party logistics providers have minimized transportation costs for products. At the same time, this new means for doing business has often changed the demands for transportation infrastructure.

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The focus for this research remains on transportation agencies (both state departments of transportation and representatives from metropolitan planning organizations) in the upper Midwest region, although the components and strategies identified will be applicable outside of this upper Midwest region and readily transferable to other transportation agencies.

Key words: freight management—logistics—public sector

Improving Communication on Wisconsin Department of Transportation Construction Projects

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ABSTRACT

Concerns have been expressed by both the state highway agencies and the construction contractors that deliver transportation projects regarding working relationships, communication, and dispute resolution on construction projects. The increasingly frequent use of consultant engineers for project management has added to the complexity in both the chain of command and in decision making. It has been estimated that 50% to 75% of all construction project time extensions, cost overruns, and contractor claims might have been mitigated or eliminated with better communication and a stronger pre-award focus.

A study was undertaken by the Wisconsin Department of Transportation in collaboration with the Wisconsin Transportation Builders Association to develop tools and techniques that can be used jointly by agency engineers, consulting engineers, and contractors to enhance project-level communication, promote early identification of risk items, and facilitate timely decision making. The communication tools developed include pre-construction and progress meeting agenda templates, meeting note taking forms, responsibility matrices, and dispute escalation forms. A project-specific risk identification process has been developed for use by construction teams that accounts for both the likelihood of the risk and its consequence for the project and suggests communication/issue resolution strategies for the team based upon the risk analysis. A request for information process that fosters better communication and more timely decision making is also outlined and detailed. Instructional guidelines are provided for the tools presented in the paper.

Key words: construction—project communication—project management

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INTRODUCTION

Communication and collaboration are fundamental components of construction management for delivering successful transportation projects. However, the highway construction industry continues to experience poor communication, coordination, and decision making on projects, which leads to cost and schedule overruns, increased disputes, and protracted project close-out processes.

The Wisconsin highway construction community was seeing these issues with increasing frequency on construction projects, and in 2006 the Wisconsin Department of Transportation (WisDOT) and the Wisconsin Transportation Builders Association (WTBA) began jointly looking for solutions. The initial step was to hold fact finding workshops involving WisDOT engineers, consultants, and contractors to obtain feedback on problem areas and to solicit recommendations. The outcome of these workshops was to begin a new initiative that would focus on project communication efforts. The feeling was that improving communication on a project would lead to the more open sharing of information, resulting in more collaboration, improved decision making, a less adversarial approach to the project, and ultimately fewer disputes.

A joint WisDOT/WTBA task force was assembled in late 2006 to work with the Construction and Materials Support Center of the University of Wisconsin, Madison, to develop specific actions that could be taken by contractors, WisDOT personnel, and WisDOT's project management consultants to improve project communication. The task force called the endeavor the Project Communication Enhancement Effort (PCEE). The result of this work was the creation of a series of tools that could be used by construction project teams to enhance their communication efforts and processes.

Project Communication Enhancement Effort

The overall goal of the PCEE effort was to improve communication on highway construction projects before, during, and after the construction phase. The specific identified goals of the task force included the following:

- To avoid adding to the already heavy workloads of field personnel
- To provide tools to help the efficiency and effectiveness of communications already occurring
- To provide guidance and support for inexperienced field personnel

The communication tools were classified as being either (1) post-project award, (2) pre-construction, or (3) construction contract administration. It was felt that this division provided a systematic approach to both developing and implementing the new processes in the field.

Special attention was paid to the post-award (prior to contract execution, in the WisDOT process) and pre-construction communication phases, since the Task Force felt these areas would benefit most from stronger communication between the contractor and WisDOT project leaders. Concentrating on communications before construction begins was also consistent with the results of an informal survey taken at the AASHTO Subcommittee on Construction. That survey of state highway agency construction engineers estimated that 50% to 75% of all construction project time extensions, cost overruns, and contractor claims could have been mitigated or eliminated with better communication and a stronger pre-award (i.e., contract execution) focus.

The task force felt it was at the post-award, pre-construction phase where good communications set the stage for the open sharing of information, early identification of issues, commitment to collaborative problem solving, and the timely making of decisions to facilitate the successful completion of the project. Tools were also developed to enhance the communication that occurs during the construction phase of a project in an effort to identify and resolve issues before they impact the cost or schedule of the project and to improve the project close out process.

The following sections develop in more detail the three classifications of communication tools: post-award, pre-construction, and construction contract administration.

POST-AWARD COMMUNICATION

Post-award communication tools were developed to help the project team get organized and initiate project communications at the initial stages of the construction project. In the WisDOT contract process, the post-award period begins when the low-bid contractor is notified that they have been awarded the project, but no work can start until the construction contract has been executed by both parties. This process takes approximately four weeks, and it gives both the contractor and WisDOT project management staff time to begin planning for the work. The tools developed for improving communication at this stage were as follows:

- Line of Communication and Decision Time Form
- Pre-construction Issue Identification Form

Line of Communication and Decision Time Form

The Line of Communication and Decision Time Form, Attachment 1 at the end of this paper, is to be initiated by the WisDOT project leader as soon as the low-bid contractor has been confirmed. The form identifies who will be on the project team from each organization (prime contractor and WisDOT) and lists these individuals' contact information. This information is to be recorded prior to the project pre-construction conference and handed out at that conference to facilitate communication between all the stakeholders on the project.

This form also identifies nominal decision times for making decisions that affect the project. These nominal times are to be discussed and agreed upon by both organizations. This information should be presented at the project pre-construction meeting so that everyone on the project is familiar with the decision making process and understands the commitments from the project team to make decisions within these timeframes. If decisions are not made at the identified organizational level within the agreed upon timeframes, the issue should be elevated to the next level for a decision.

The form also provides the basis for dispute escalation should a disagreement arise on the project. It starts at the project leader/foreman level and escalates through the respective organizational hierarchy. The escalation times should be the same as the nominal decision times unless modifications are mutually agreed upon.

Pre-construction Issue Identification Form

Construction projects are complex endeavors, and every project has unforeseen issues that require decisions be made within short timeframes to prevent impacts to the project that may increase costs or extend the schedule. To the degree these potential issues can be identified and planned for prior to starting construction, listing these issues can greatly reduce the impact on the project and the construction delivery team. The Pre-construction Issue Identification Form, Attachment 2, was developed as a tool to start the process of identifying these potential issues for the project and assessing the impact they may have on the project. What is done with the identified issues will depend upon the type of issue and the nature of the project. Relatively low-impact issue items may simply be assigned to project team members to monitor, while other issues may require extensive analysis, planning, and mitigation efforts. The purpose of this tool is to help identify those issues early in the project and begin the communication process to minimize any effect they might have on the project.

The WisDOT project leader and the prime contractor's superintendent each fill out the Pre-construction Issue Identification Form independently. The two then jointly review the assessments, identify areas of agreement and disagreement, and determine what communication level they will utilize to begin addressing these issue items. A fairly small number of noncomplex, low-cost issues may be best addressed within the pre-construction meeting itself. A large number of complex, high-cost issues could require separate meetings of the project delivery team.

The form is filled out by reviewing each general issue item (other project-specific items can be added as needed) and determining the impact the issue would have on the project should it occur, i.e., a minimal impact or a significant impact. Next, an assessment is made as to the likelihood of the issue occurring on the project. A number from 0 to 3 is then assigned to that particular issue. For example, if an issue/situation has little or no likelihood of occurring, it is given a value of "0." If an issue has little likelihood of occurring and, were it to happen, it would have minimal impact on cost or schedule, it should be assigned a "1." An issue with a high likelihood of occurring and that would have a significant impact should be given a value of "3." The assigned value for each issue is circled, and then a total for each category of issues is calculated and entered on the form. At the bottom of the form, an overall total is calculated.

Depending upon the value calculated for the total of the individual issue identification assessments, various levels of communication strategies are suggested on the second page of the form to assist in further evaluating, planning for, and mitigating these issue items.

PRE-CONSTRUCTION COMMUNICATION

Pre-construction tools were developed to help the project delivery team continue their communications before actually starting construction. The tools developed for this stage include the following:

- Pre-construction Meeting Agenda
- Subcontractors Contact Information Form
- Responsibility Matrix

The first two tools listed above are not presented in this paper, but copies are available from the authors. The Pre-construction Meeting Agenda template was developed to enhance the guidance already provided in the WisDOT *Construction and Materials Manual* for pre-construction meetings in order to specifically focus discussions on issue identification and resolution. The Subcontractor Contact Information Form is similar to the Line of Communication form shown in Attachment 1.

Responsibility Matrix

The Responsibility Matrix, Attachment 3, was developed to provide background information that would give members of the project delivery team a better understanding of who is responsible for initiating and approving various items and how the communication of these items occurs. This information is to be discussed at the pre-construction meeting and to be posted within the project construction office.

CONSTRUCTION CONTRACT ADMINISTRATION COMMUNICATION

Communication between the contractor and the project management team is essential for a successful project, and most project delivery teams feel they do communicate. However, the communication is often not very well structured or documented. The communication tools for this phase of the project were developed to bring more structure to the process and to improve the effectiveness of the communication that had been occurring. The tools developed were as follows:

- Request for Information Submittal Form
- Request for Information Log
- Progress Meeting Agenda
- Progress Meeting Note Form

All projects are encouraged to hold weekly progress meetings. The Progress Meeting Agenda template was developed to provide structure to these meetings and to ensure topics such as work progress, schedule updates, delays, upcoming activities, etc., are discussed and issues resolved. The Progress Meeting Note Form was created to assist project staff in concisely summarizing project developments, major areas of concern, and needed action items. This form also increases accountability for resolving issues and following through on commitments. A meeting agenda template and sample note form are not included in this paper, but copies can be obtained from the authors.

While project progress meetings are fairly standard, use of a request for information (RFI) process is completely new to Wisconsin highway projects. This tool, which includes the first two items listed above, was created to provide more structure to the issue identification process, more accountability for providing answers or decisions to questions, and a more formal documentation process for the issues identified.

Request for Information Process

The purpose of the RFI process is to obtain clarification of the plans, specifications, special provisions, or other contract documents. It also provides for a systematic collection of the analysis and a resolution for questions that arise during the construction of the project. WisDOT had not historically used an RFI process, but a review of construction practices in private building construction and other state highway agencies indicated that an RFI process had many benefits, including enhancing communication on projects, improving the speed at which questions are answered, and providing documentation on the questions and issues that come up during the project. While the use of an RFI process was expected to initially add to the workload of the project team during the project, it was felt that there would be a net time savings over the life of the project and during the project close-out process. The RFI process includes two documents: (1) the RFI Submittal Form, Attachment 4, and (2) the RFI Log, Attachment 5.

The contractor typically initiates the development of an RFI. However, anyone can submit an RFI for clarification on an issue. The WisDOT project leader monitors, tracks, and expedites the response to an RFI. Responses are to be provided on a timely basis (usually within seven days of receipt) so as to not affect the construction schedule. The desired response time is indicated on the RFI Submittal Form to indicate the urgency of the question.

Request for Information Process Responsibilities

Successful implementation of an RFI process requires each member of the construction delivery team to take responsibility for various actions and steps in the process. The responsibilities detailed below are for the most typical situation, in which the contractor initiates the RFI:

- The contractor is responsible for notifying the WisDOT project leader of a request for information using the RFI Form.
- The contractor clearly and concisely sets forth the issue for which clarification or interpretation is sought and explains why a response is needed. Appropriate references to specifications, plans, or drawings should be provided to facilitate a timely response.
- The WisDOT project leader sequentially numbers the RFI and logs it in the RFI Log form.
- The WisDOT project leader processes the RFI and coordinates the response by consulting with others as needed (e.g., project manager, designer, topic experts, etc.). If a response time longer than seven days is needed, the requester is notified of the anticipated response time.
- The WisDOT project leader prepares the response, forwards one copy to the RFI requester, and files one copy on-site for reference.
- The WisDOT project leader maintains the RFI Log to track the status of an RFI and to maintain a catalog of all RFIs submitted during the project.
- The WisDOT project leader and contractor's superintendent discuss outstanding RFIs and potential RFIs as a standing agenda item at the project progress meetings.
- Disagreements regarding the response to an RFI immediately trigger the issue resolution process identified on the Line of Communication and Decision Time Form.
- The WisDOT project leader and contractor are responsible for working together to assure that all RFIs are appropriate and to control the number of RFIs. Contract documents are to be reviewed and researched before submittal of an RFI so as to not overburden the project management staff with large numbers of RFIs.

CONCLUSION

In response to growing concerns within the Wisconsin highway construction community over the increasing number of disputes, growing length of time to resolve issues, increasing amount of time to close out projects, and deteriorations in overall project working relationships, WisDOT and the WTBA collaborated with the Construction and Materials Support Center of the University of Wisconsin, Madison, to enhance communication on WisDOT projects. The result of that effort was the creation of a number of communication tools to improve upon the effectiveness of the communication already occurring on construction projects. Many of the tools specifically focus on the pre-construction communication between the construction delivery team to help identify project issues and promote resolution of the issues before the start of construction. Other tools assist in communication during construction in an effort to develop collaboration and information sharing among everyone on the project. The tools will initially add to the workload of the project team, but overall it is believed that workloads will be reduced with less time spent resolving disputes and dealing with claims, and fewer issues will come up during the project close-out process. The tools are being piloted on seven WisDOT construction projects in 2007, and follow-up studies will be conducted to evaluate the effectiveness of the tools and further improve communication on highway projects.

ATTACHMENT 1. LINE OF COMMUNICATION AND DECISION TIME FORM

Foreman		**Superintendent**	
Name	______________________	Name	______________________
Phone	______________________	Phone	______________________
Mobile	______________________	Mobile	______________________
Email	______________________	Email	______________________
FAX	______________________	FAX	______________________

Project Leader		*__Nominal Decision Time__
Name	______________________	
Phone	______________________	**1 Day**
Mobile	______________________	(Days)
Email	______________________	
FAX	______________________	

Project Manager		
Name	______________________	
Phone	______________________	
Mobile	______________________	**2 Days**
Email	______________________	(Days)
FAX	______________________	

Project Development Supervisor		
Name	______________________	
Phone	______________________	
Mobile	______________________	**3 Days**
Email	______________________	(Days)
FAX	______________________	

Project Development Chief			**Contractors Main Office**	
Name	______________________		Name	______________________
Phone	______________________		Phone	______________________
Mobile	______________________	**4 Days**	Mobile	______________________
Email	______________________	(Days)	Email	______________________
FAX	______________________		FAX	______________________

*Designated times are generally assumed to be the maximum, however, these decision time frames may be adjusted to fit unique project circumstances by mutual agreement.

ATTACHMENT 2. PRE-CONSTRUCTION ISSUE IDENTIFICATION FORM

Pre-Construction Issue Identification Form
(Construction issues that can impact cost and schedule)

[M: Minimal Impact S: Significant Impact] Circle one number per issue; Sum for each issue group

Problem/Issue/Situation	Likelihood & Impact Assessment		
Project Personnel and Coordination	**M**	**S**	**Total**
Coordination and directing of subcontractors	0 1	2 3	
Conflicting construction operations	0 1	2 3	
Significant 3rd Party involvement	0 1	2 3	
On-site management by prime contractor or WisDOT	0 1	2 3	
Utility Disruption	**M**	**S**	**Total**
Unknown or unanticipated discovery of utilities	0 1	2 3	
Relocation of utilities in work zone	0 1	2 3	
Coordination of work activities with utilities	0 1	2 3	
Differing Site Conditions	**M**	**S**	**Total**
Unsuitable subgrade material	0 1	2 3	
Groundwater	0 1	2 3	
Hazardous materials	0 1	2 3	
Man-made buried objects	0 1	2 3	
Unstable slopes or excavations	0 1	2 3	
Archeology sites	0 1	2 3	
Site Conditions	**M**	**S**	**Total**
Inadequate staging areas	0 1	2 3	
Erosion and sediment control	0 1	2 3	
Disruption to local traffic and business operations	0 1	2 3	
Complex traffic control plan	0 1	2 3	
Noise, vibration, and dust impacts on adjacent properties	0 1	2 3	
Detours, Haul Roads & Repair	0 1	2 3	
Schedule and Operations	**M**	**S**	**Total**
Submitted schedule, updates, controlling items	0 1	2 3	
Interim completion dates	0 1	2 3	
Staging and sequencing	0 1	2 3	
Waste and/or borrow sites	0 1	2 3	
Shortages or delayed delivery of materials	0 1	2 3	
Expedited schedules or night/week-end work	0 1	2 3	
Extreme weather conditions or seasonal effects	0 1	2 3	
Design and Contractual Issues	**M**	**S**	**Total**
Constructability of plan	0 1	2 3	
Sensitive environmental features	0 1	2 3	
Unique special provisions	0 1	2 3	
Document submittal, review and approval	0 1	2 3	
Need for permits	0 1	2 3	
QMP Specifications & certifications	0 1	2 3	
DBE involvement	0 1	2 3	

ATTACHMENT 2. CONTINUED

OVERALL TOTAL FROM PREVIOUS PAGE: ____________

OVERALL SCORE	**RECOMMENDED COMMUNICATION LEVEL**
0 - 15	(1) Discuss identified issues at the Preconstruction Meeting
15 – 30	(2) Project Leader, Project Manager, Contractor's Foreman and Superintendent meet to discuss strategies for addressing the identified issues
30 – 45	(3) Half-day issues meeting involving WisDOT Project Team, Prime Contractor, Major Subcontractors and Utilities to address issues, impacts, and how to minimize the impacts of should they occur.
45 +	(4) Facilitated full-day issues workshop to further identify the issues, impacts, and resolution. Continuing follow-up issues meeting should be scheduled monthly for duration of the project.

ATTACHMENT 3. RESPONSIBILITY MATRIX

Responsibility Matrix

I- Initiates R- Receives A- Approves C- Receives copy PS- Participate in/supports PN- Prepares Notes D- Distributes	DOT Inspector	DOT-Project Leader	DOT Project Manager	PD Super-visor	PD Chief	BPD Oversight Engr	FHWA-oversight only	Foreman	Superin-tendent	Contractor Main Office	Sub-con
Precon Meeting		I/PN/D	PS	C/PS		C	PS	PS	PS/I	C	PS
Request to Sublet		A	R						PS	I	
Project Schedule		PS/D	R	C		C	C	PS	I	PS	PS
Notice to proceed		I/D	C	C/PS		C	C			C	
Source of Materials		A	C/R					C	PS	I/D	PS
Project Progress Meetings	PS	I/PN/D	C/PS			C		PS	I/PS	C	PS
CRI Proposals	PS	R	A/PS	PS	C	PS/C	C	PS	I/D	PS	PS
RFI by Contractor		R/PS						I	I		I
RFI by WisDOT		I	I					PS	P/R		
Payment estimates	PS	I	R/A							C	
Contract Modifications	PS	I/D	C/A	C/A	A	C	C/A	PS	I	C/PS	PS
Work Inspection Report	I	R/D						C	C		
Report of deficient materials	I	I	PS			C		PS	PS	PS	PS
Final project acceptance	PS	I/D/PS	A/PS	C		C	PS/C			C/I	
DBE Performance										I	
Certified Payrolls		PS							PS	I/C	I/PS
Shop Drawings		PS/C								I/D	PS
Environmental permits		R							PS	I/D	PS
Modifications to traffic control	PS	I/D	PS	PS	PS		PS	PS	PS	I/D	PS
Erosion Control Insp. Report	I/PN	A						PS	A		
Claim Initiation Form		I	C	C	C	R					

ATTACHMENT 4. REQUEST FOR INFORMATION FORM

REQUEST FOR INFORMATION (RFI)	
PROJECT ID	**PROJECT NAME**
DATE	**RFI NUMBER (Assigned By Project Leader)**
TO:	**FROM:**
METHOD SENT: ____ FAX ____ MAIL ____ E-MAIL ____ DELIVERED	
INITIATED BY:	
DESCRIPTION OF REQUEST:	
ADDITIONAL SUPPORT DOCUMENTS ______ ARE ______ ARE NOT ATTACHED	
DATE REQUIRED:	
DATE RESPONSE RECEIVED:	
RESPONSE FROM:	
RESPONSE:	

ATTACHMENT 5. REQUEST FOR INFORMATION LOG

Request for Information Log

PROJECT ID: ______________________________
PROJECT NAME: ______________________________
LOCATION: ______________________________
PROJECT LEADER: ______________________________

RFI NUMBER	DATE SUBMITTED	DESCRIPTION OF INFORMATION NEEDED	SUBMITTED TO	DATE OF RESPONSE	WHO INITIATED	STATUS (PENDING/CLOSED)

Laboratory and Field Testing of Precast Bridge Elements Used for Accelerated Construction

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ABSTRACT

Black Hawk County (BHC) has developed a precast modified beam-in-slab bridge (PMBISB) system for use with accelerated construction. Individual components of the system have been tested in the Iowa State University Structural Laboratory, and the overall system was tested in the field. Using the BHC system, the bridge superstructure can be assembled in two days and the bridge opened to traffic as soon as the cast-in-place concrete connection between the precast panels has reached the required strength, which is usually one week.

Precast components tested in the laboratory include two precast abutment caps, three different types of deck panel connections, and a precast abutment backwall. Load testing on the first abutment cap revealed that it had considerably more strength than was required. Therefore, the cap was modified to be more efficient. The original PMBISB panel connection was inefficient in the amount of time and concrete needed for construction. Thus, three other panel-to-panel connection designs were tested in the laboratory, one of which has already been used in the field. The final laboratory work was the service load and ultimate load testing of a precast abutment backwall panel.

Field testing of the first precast bridge (L = 40 ft., W = 32 ft.) in Black Hawk County was performed in June of 2007. The testing investigated service load stresses, lateral load distribution characteristics, and overall global behavior of the system. This paper presents the results of the laboratory and field testing previously described.

Key words: accelerated construction—bridge construction—precast concrete

INTRODUCTION AND PROBLEM STATEMENT

Precast concrete elements can be used to construct a bridge much more quickly and efficiently than a traditional cast-in-place concrete bridge. In addition, the precast elements are usually more uniform and durable than cast-in-place concrete elements because of more controllable casting conditions and stricter quality control. The modified beam-in-slab bridge (MBISB) investigated in Iowa Department of Transportation Project TR-467 was improved upon by using precast concrete elements. Testing of the precast modified beam-in-slab bridge (PMBISB) was presented in a report to the Black Hawk Counter (BHC) engineer (Wipf et al. 2004). The PMBISB is a series of panels consisting of steel girders with concrete placed around the girders. Eighteen in. PVC pipe is cut in half and placed between the girders to remove the concrete from the tension zone of the slab. In addition to the superstructure, BHC developed a precast abutment cap that utilizes a steel W-section that fits on top of the abutment H-piles. A precast abutment backwall that is positioned between the abutment H-piles was also developed by BHC. Construction is accelerated by having these elements cast during the winter, thus reducing the amount of man hours and overall time required to assemble the bridge in the field. To determine the strength and behavior of the precast elements, they were individually tested in the laboratory. To determine their interaction, a bridge constructed using these individual elements has been service load tested.

OBJECTIVES AND METHODOLOGY

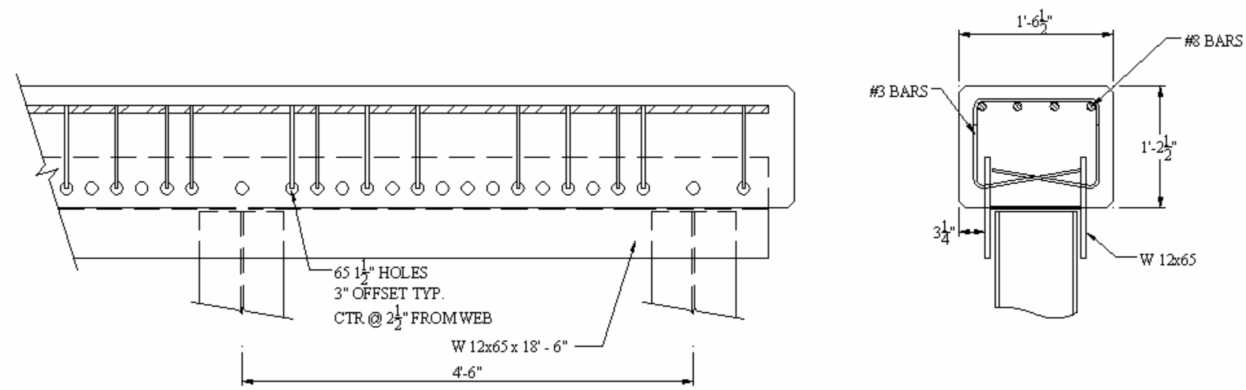
The purpose of the laboratory testing was to determine the strength, serviceability, and constructability benefits of the system. Specifically, it was desired to determine the strength and behavior of the precast abutment caps, the different cast-in-place connections between the deck panels, and the precast abutment wall. Field testing of the system focused on service load stresses, lateral load distribution characteristics, and overall global behavior of the system.

Precast Abutment Caps

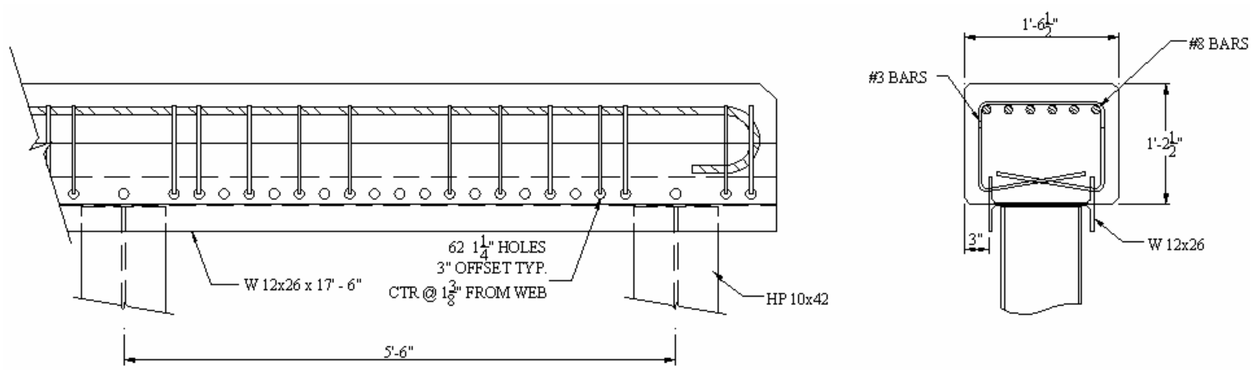
The precast abutment cap developed by BHC is made by casting concrete around the upper half of a steel W-section lying on its side, as seen in Figure 1. Holes are torched in the embedded flange to create a shear connection and composite action between the steel and concrete. When positioned on the abutment piles, the web of the W-section rests on top of the H-piles, with the flanges providing lateral restraint. Reinforcing steel is cast in the top of the specimen to provide increased negative moment capacity over the piles and to act as compression steel in the positive moment regions. Stirrups are also cast into the cap to provide increased shear capacity.

The first cap to be tested was constructed by Black Hawk County with a W12x65 section, shown in Figure 1a. The W12x65 was chosen to give the cap adequate flexural strength. During the service testing, supports were spaced at 4.5 ft., and the maximum stress in the steel was found to be 6.14 ksi when a service load of 40 kips was applied. Supports were spaced at 17.5 ft. for the positive moment strength test and spaced at 15 ft. for the negative moment strength test. The strength testing determined that the positive moment strength is greater than 765 kip-ft., and the negative moment strength is 363 kip-ft. In comparison, the factored design moment for the cap is 157 kip-ft.

A modification to the abutment cap was proposed as the strength of the cap was significantly larger than what was required. To increase efficiency of the abutment cap, Black Hawk County constructed a second cap using a W12x26 section, shown in Figure 1b, to reduce the weight, the cost, and strength of the cap. To accommodate new backwall sections, support spacing for the service level testing was changed to 5.5 ft., with the maximum stress in the steel only 6.46 ksi at the same service load of 40 kips, a 5.0% increase in stress. Only a positive moment strength test was performed on the smaller section, with support spacing at 15.5 ft. The cap failed at a moment of 465 kip-ft., significantly greater than the factored design moment of 156 kip-ft.



(a) W12x65 abutment cap



(b) W12x26 abutment cap

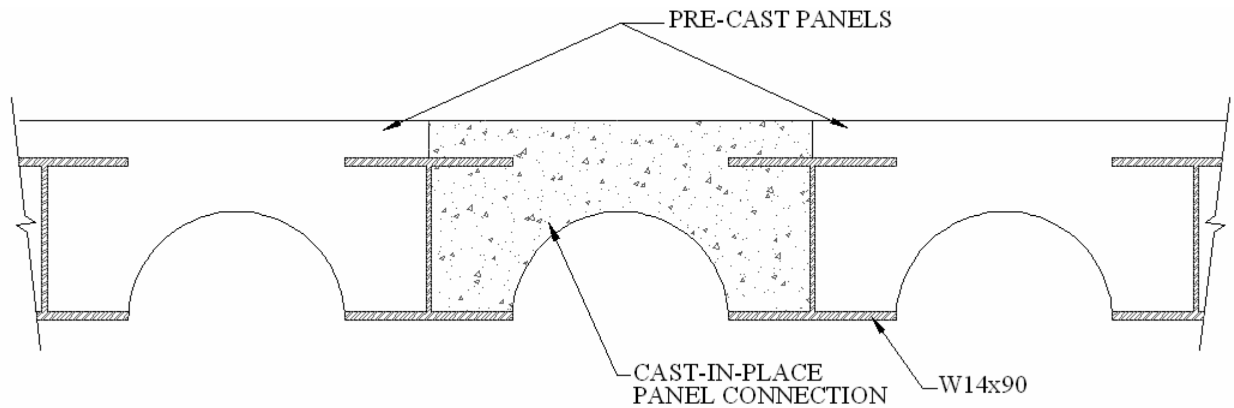
Figure 1. Precast abutment caps

Improving PMBISB Panel Connections

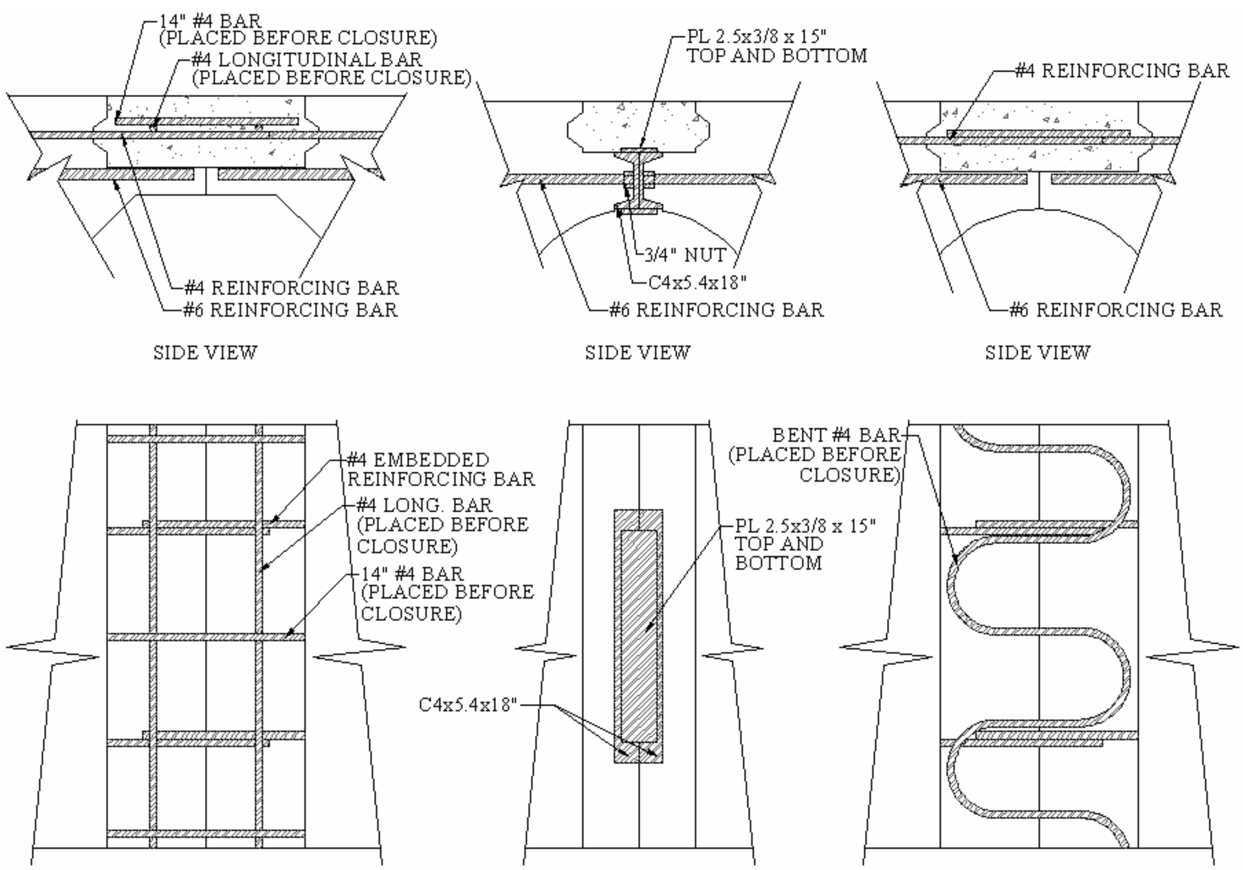
The original field connection, shown in Figure 2a, required a substantial amount of formwork to be constructed in the field and a significant amount of cast-in-place. Reducing the amount of time (for construction of the formwork) and concrete required in the construction was the main goal of redesigning the connection. The basic feature of the new designs is the use of a half-arch at the side of each panel. Above the half-arch is a notch that provides the formwork for the closure pour when two panels are placed next to each other. Three new designs were developed using the half-arch feature, and three specimens of each detail were cast and subjected to laboratory testing to determine their strength and behavior characteristics. For determining the strength of the connections symmetric point loads were applied on either side of the closure pour, creating a pure moment region.

Type 1 Connection

The first connection tested has #4 reinforcing bars on 15 in. centers that protrude out of each precast panel into the notched area on the adjacent panel (see Figure 2b). Before concrete is placed into the closure area, two longitudinal #4 reinforcing bars are placed along the entire length of the joint along with additional transverse #4 bars placed between the protruding reinforcing bars. This particular connection has already been used in the replacement bridge on Mt. Vernon Road in BHC.



(a) Original PMBISB field connection



(b) Type 1 Connection **(c) Type 2 Connection** **(d) Type 3 Connections**

Figure 2. Precast panel connection details

Type 2 Connection

The Type 2 Connection removes the reinforcing bars from the closure area, as shown in Figure 2c, tying the panels together with two PL2.5x3/8 plates welded to the top and bottom of two embedded C4x5.4 channels. The channels are attached to the panels by welding the channels to the #6 reinforcing bars at the top of the arch. Welding area between the channel and the reinforcing bar is increased by welding a 7/8 in. nut onto the end of the bar. One obvious difficulty with this detail is the need to perform overhead field welding.

Type 3 Connection

The Type 3 Connection is similar to the first in that reinforcing steel is placed in the field onto the protruding reinforcing bars before the closure pour. Instead of straight individual pieces of longitudinal and transverse reinforcing bar, the added reinforcing bar is bent into a series of S's, as shown in Figure 2d. The bent bar reduces the amount of steel tying required in the field.

Table 1 presents the ultimate moment capacities for each connection detail. The capacities were all normalized to the concrete strength of the Type 1 Connection.

Table 1. Connection detail ultimate moment capacities

	Normalized moment capacity (k-in./ft.)		
Specimen	**Type 1**	**Type 2**	**Type 3**
A	45.2	81.5	24.6
B	38.7	69.5	28.6
C	38.2	80.9	28.1
Average	40.7	77.3	27.1

While it is the weakest, the Type 3 Connection is the easiest to construct in the field, requiring less tying than the Type 1 detail. However, the Type 2 Connection has the greatest moment capacity, nearly double the capacity of the Type 1 Connection which is currently in use. However, constructability is an issue with the Type 2 Connection since an experienced welder would be required for the overhead welding in the field. The Type 1 Connection has been used successfully in the field and is relatively easy to fabricate.

Abutment Backwall

The precast backwall is a 14 ft. by 4.25 ft. reinforced slab of concrete designed to support the soil behind the abutment when supported by the flanges of the H-piles of the abutment. The variation in the transverse reinforcement accounts for the increased load with depth due to the lateral earth pressure of the soil: six #4 bars spaced on 12 in. centers for the first 5.5 ft. and 18 #4 bars on six in. centers for the remaining 8.5 ft. Longitudinal reinforcement is provided by four #5 bars that run the entire length of the slab, as seen in Figure 3.

Initially the backwall was service load tested without and with the supporting H-piles. The experimental cracking moment, determined from testing without the H-piles, was found to be 29.1 kip-ft., which is very close to the calculated cracking moment of 29.4 kip-ft. Three service level point loads were applied separately and simultaneously at linearly varying magnitudes to simulate lateral earth pressure. When the H-piles were added, the midspan deflection under the center load only was reduced from 0.091 in. to 0.049 in., a reduction of 46%. An ultimate load test was also performed by loading the slab at a single point approximately 112 in. from the top of the wall. The slab was damaged during the first attempt when the spliced H-pile failed prematurely. At the time of the H-pile weld failure, the system was carrying 80 kips. After repairing the H-pile, the test was performed again, and the slab was still able to resist a point load of 100 kips. The expected resultant load from both a truck over the abutment and the lateral earth pressure from clay under end-of-construction conditions is approximately 63 kips. The end-of-construction soil condition provides the largest lateral earth loads. Even in its weakened state, the backwall system provided a factor of safety of 1.6 against failure.

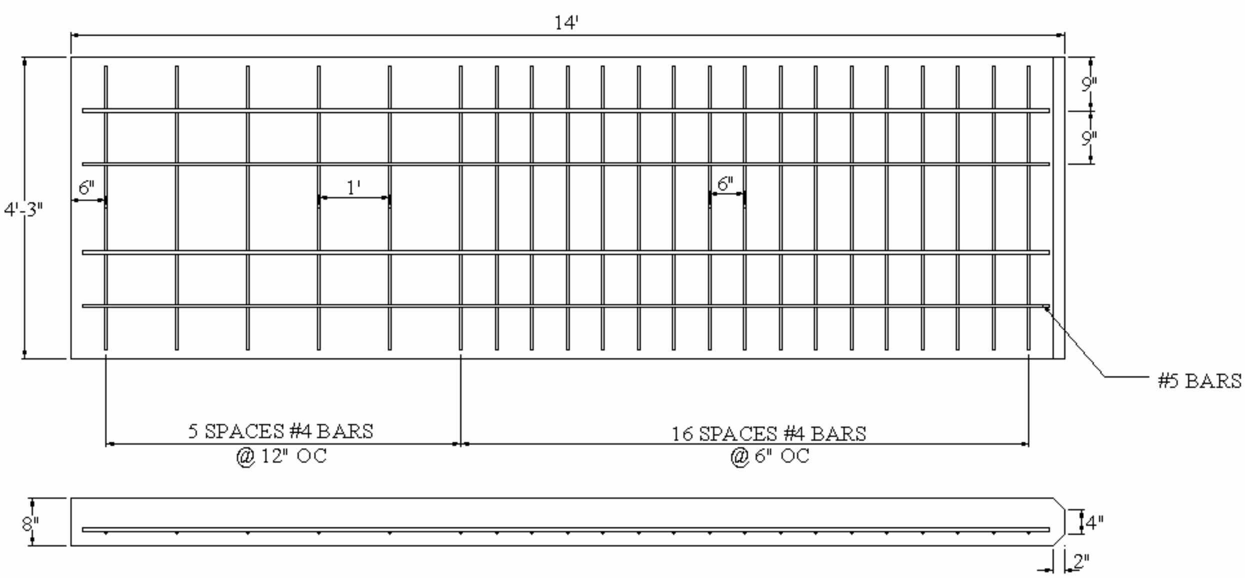
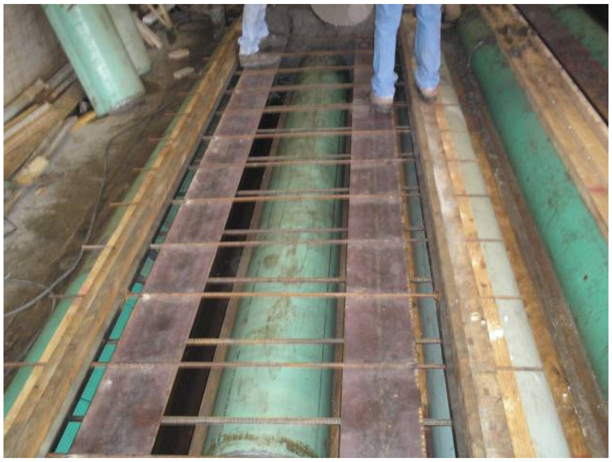


Figure 3. Precast abutment backwall

Mt. Vernon Road Bridge

The Mt. Vernon Road Bridge (L = 40 ft., W = 32 ft.) utilized two of the components investigated in the laboratory testing: the improved PMBISB connection detail, and the W12x65 precast abutment cap. The panels and abutment caps were precast at Black Hawk County facilities during the winter (Figure 4a). After the abutment walls were constructed, the abutment cap was placed on top of the H-piles (Figure 4b). At this point, the deck panels were transported to the site and placed on the abutment caps (Figure 4c), at which point the additional steel required for the joint detail was tied into the joint. Concrete for the closure joint was placed after all six panels were in their final positions and the joint reinforcement added. A standard C4 mix was used for the closure; workers were able to easily work the concrete using shovels and vibrators to fill the closure (Figure 4d). Total time to assemble the superstructure, including the closure pour (Figure 4e), was less than 40 hours. The bridge was opened to traffic after the concrete cured and a thrie beam guardrail system was installed (Figure 4f).

Field testing of the Mt. Vernon Road Bridge was conducted midway through June. A standard rolling test was executed over five different lanes on the bridge deck. Preliminary results show that the maximum tensile stress in the steel was 2.6 ksi, and the maximum compressive stress in the concrete deck was -0.25 ksi. The allowable stress for the steel used in the design of the panels was $.55f_y$ (27.5 ksi), while the allowable stress in the concrete was $.4f_c$' (2 ksi). Maximum deflection attained was only 0.18 in., well below the American Association of State Highway and Transportation Officials (AASHTO) live load deflection limit for vehicular loads of Span/1000 which is 0.48 in. (AASHTO 2004).



(a) Casting the deck panels

(b) Abutment cap on abutment piles

(c) Placing the deck panels

(d) Casting the panel connections

(e) Completed bridge without guardrails

(f) Completed bridge with guardrails

Figure 4. Construction of Mt. Vernon Road Bridge

SUMMARY AND CONCLUSIONS

Precasting the panels for the MBISB was an effective way to reduce the amount of time needed to construct the bridge. More reduction in construction time can be achieved through a redesign of the panel-to-panel connection. Precasting more of the bridge elements, such as the abutment cap and abutment backwall, further reduces construction time.

In the first design of the precast abutment cap, a W12x65 steel section was used. Due to its extra strength, it was decided to make the cap more efficient by using a W12x26 steel section. While expectedly having less strength than the original design, the new design still has more than enough strength to handle the loads expected to be placed on it from legal loads.

All three panel connection designs provide a more efficient and effective means of joining adjacent panels together in the field. The major drawbacks to the Type 2 Connection design are that it is very dependent on the skill of the welder and on favorable conditions under the bridge to facilitate overhead welding. The Type 3 Connection, while slightly easier to construct in the field, is markedly weaker than the Type 1 Connection. For these reasons, along with the simplicity of construction in the field, the Type 1 Connection is the preferred detail for joining the deck panels together.

A precast abutment backwall was proposed for the system to facilitate accelerated construction. Testing of the precast abutment wall, as expected, verified the system was much stronger when placed between the H-piles than it was when it was without the H-piles. Testing also determined that the strength of the abutment backwall system was more than adequate to resist the combined loads from the soil and a truck positioned above the abutment.

The construction of the Mt. Vernon Road Bridge was cost- and time-efficient. The PMBISB system allowed the superstructure to be built in less than 40 hours, while total time for the entire bridge construction was less than 22 days. Field testing determined very small stresses from the truck load, well below the recommended working level stresses. Measured deflections were also well within allowable values. Because of this, accelerated construction with the PMBISB system is a viable option for bridges on low-volume roads.

ACKNOWLEDGMENTS

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Building a National Freight Policy

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ABSTRACT

Freight is a growing issue in transportation. Freight tonnage and ton miles are growing more quickly than passenger mileage in many highway corridors. Many freeways are operating under increasingly congested conditions. Railroads are also often operating at or beyond theoretical track capacity. Congestion threatens to harm economic health, particularly in the Midwest, with its heavy reliance on manufacturing and agriculture. But the United States has been slow to respond to this growing challenge. In part the slowness is a factor of the traditional roles of the federal government, the states, and private companies in freight. In part, it may also be related to the fact that little real agreement exists on what a policy should contain. In January of 2007, a survey was sent to 250 observers or participants of an organization made up of the ten states of the American Association of State Highway and Transportation Officials' Mississippi Valley Conference, the Mississippi Valley Freight Coalition. This survey asked for reactions to a number of statements dealing with alternative federal freight transportation policy. The survey results were then used in a workshop made up of state, local, and private sector participants in the freight industry. This paper reviews the results of the survey and of the workshop. It outlines areas of apparent agreement and concerns and suggests future work.

Keywords: finance—freight—planning

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INTRODUCTION

Growing demands from both freight and personal travelers has in many corridors of the nation slowed the movement of people and goods. This has caused many people to ask how the demands for transportation can be met in the future. Many of those have asked where the national transportation policy stands. Many have also concluded that no national transportation policy exists. This paper reports on the efforts of a group, the Mississippi Valley Freight Coalition (MVFC), in the Midwest to develop a common perspective on future national transportation policy.

The coalition is made up of the ten states of the Mississippi Valley Conference of the American Association of State Highway and Transportation Officials: Illinois, Indiana, Iowa, Kansas, Kentucky, Michigan, Minnesota, Missouri, Ohio, and Wisconsin. The National Center of Freight and Infrastructure Research and Education at the University of Wisconsin and its partners at the University of Illinois, Chicago, and the University of Toledo provide staff support. The coalition was formed in 2006. One of its first efforts was to prepare regional testimony for the National Surface Transportation Commission, which held public hearings in the first half of 2007.

To gain agreement from the member states in the coalition, three steps were taken. A survey was administered; a workshop was held; and commission testimony was drafted, reviewed and revised. The entire process took more than four months.

SURVEY RESULTS

January of 2007, an electronic survey was distributed to about 250 observers or participants of the coalition. The survey asked for responses to a number of statements related to the role of the federal government in freight transportation. Eighty-one, or 34%, responded. Thirty-nine respondents identified themselves as government employees; forty-two as non-governmental representatives. The survey was intended to provide input for the workshop dealing primarily with that issue. Overwhelmingly, respondents favor a stronger federal role in freight transportation. They also favored a maximum degree of flexibility for the states, a federal focus (such as might be found through the designation of a federal freight system), a multimodal approach, and regional cooperation. These conclusions are summarized in Figure 1.

Figure 1. Survey conclusions

Somewhat less unanimity was found in dealing with defining performance objectives in return for financial assistance to rail and water modes, the use of regulation to ensure that needed services are provided by each mode, and the imposition of new taxes. The majority of respondents supported each of these propositions, but a large minority rejected them (see Figure 2).

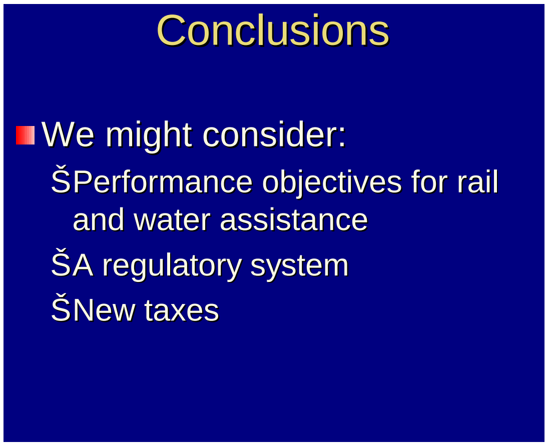

Figure 2. Tentative survey conclusions

Survey responses illustrate some of the key issues in arriving at a national policy. The first major issue is the conflicting roles of state and federal governments. The participants overwhelmingly said the federal role should be strengthened (see Figure 3),, but they also said funding should be available to the states in a flexible form (see Figure 4). This response of stronger federal direction with sustained or enhanced state flexibility is a difficult combination to produce.

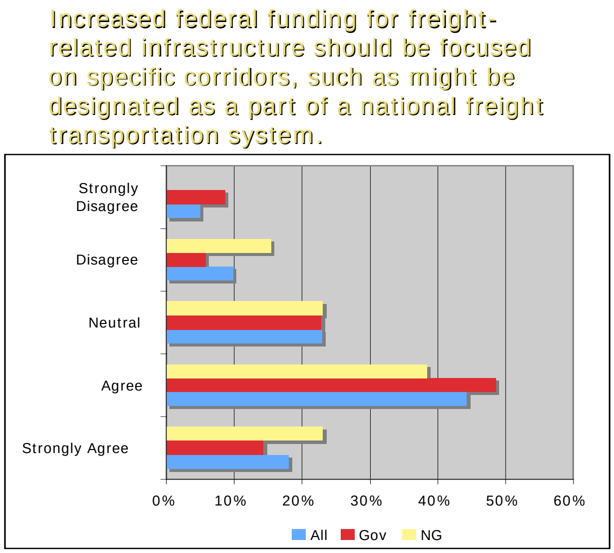

Figure 3. The federal role

Increased federal funding for freight-related highway infrastructure should be channeled through the existing core federal programs---interstate maintenance, NHS and STP---allowing the states the maximum flexibility in the use of funds.

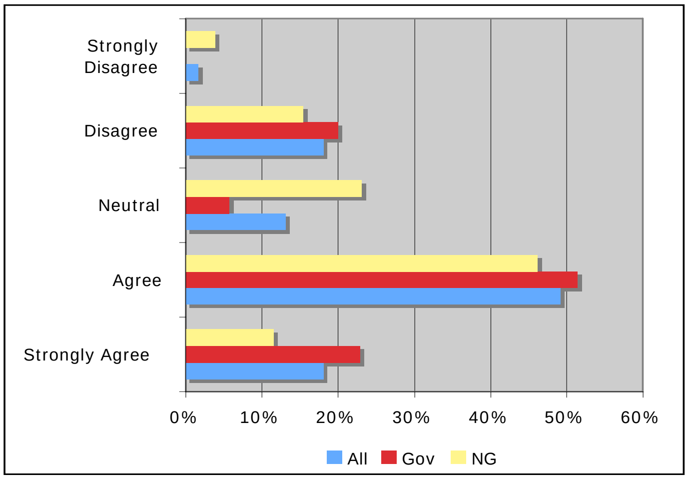

Figure 4. State flexibility

The second major issue deals with the government's role in the rail and maritime modes. Seventy percent of the respondents agreed or strongly agreed that federal transportation policy should encourage the use of nonhighway modes. Sixty-eight percent said that critical rail and water services should receive federal assistance. Sixty-two percent said critical water and rail services should be aided if the aid came from nonhighway sources. But only 46% said that a freight-related tax should be created to assist all modes. While the spread on this issue is not as great as the spread on state and federal roles, it does illustrate the fact that government participation in rail and maritime remains a difficult issue.

WORKSHOP

On February 26, 2007, 46 people braved the end of a Midwestern winter storm to attend a workshop on freight in Dearborn, Michigan. For many the trip was challenging. Closed airports and delayed flights proved to be a barrier for many who had planned to attend. Twenty-six state department of transportation representatives from nine states, eight private sector representatives, three local government officials, and nine university people were present.

The bulk of the effort within the workshop was devoted to finding points of agreement for testimony to the commission. The results of the survey were presented and discussed, as was an overview of information previously developed on freight in the region. The participants seemed to find few, if any, surprises in the survey results.

Armed with the survey and background information, participants broke into small groups to ponder the question: What do we want from the federal government?

What We Want From the Federal Government

The groups reported their findings. While they had a wide range of views, a great many common elements also existed, such as rational, flexible funding, improved data, better support for research, improved interoperability for technology, steps toward intermodalism, and modal connectivity.

These commonalities yielded a collective set of priorities for the federal government in freight policy that included the following: defining a sustainable national transportation and freight policy, intermodalism, interoperable technology standards and a support for technological innovation, leadership in developing and making available alternative and sustainable energy sources, a national effort to reduce bottlenecks, national support for improved freight information, measures to improve freight productivity, improved funding, better information to policymakers on freight, and a defined national multimodal freight system (see Figure 5).

Collective Priorities

National transportation/freight policy that is sustainable
Intermodalism
Technology: innovation, standards for interoperability, res
National energy policy for alternative and sustainable fuel
Bottleneck reduction to improve fuel efficiencies, reduce t
and increase reliability
Freight data for modelers and decision-makers
Improve freight productivity
Improve funding for freight
Improve awareness of freight by policy makers
Establishing a multimodal national freight network

Figure 5. Workshop priorities

How Should the Federal Government Carry Out Its Role?

Next, the participants divided into two groups to delve deeper into the ideas listed above. The basic question they addressed was: How do we want the federal government to carry out its role?

The first group dealt with funding, a sustainable transportation policy, and education and awareness. The second group dealt with data, technology, energy and emissions, and productivity.

Their reports succeeded in going deeper into each issue. With the funding issue, the group struggled with flexibility for the states and aligning federal funding to national strategies. They resolved this by arguing that federal programs should provide maximum flexibility to the states but use increased federal participation rates to encourage state actions that promote national strategies (see Figure 6).

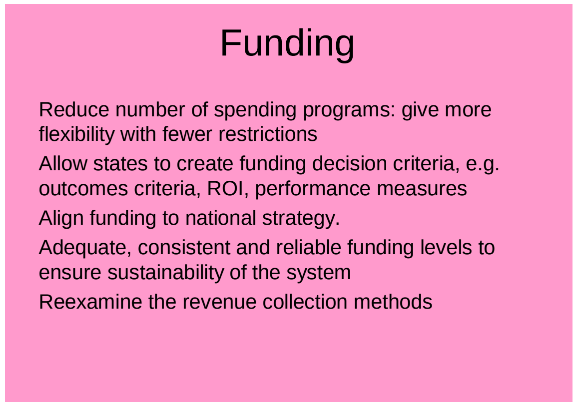

Figure 6. Funding recommendations

A national transportation policy should incorporate a number of factors in the decision making process: the economy, environmental concerns, safety, and equity. It should encourage the use of technology to maximize the benefit of the existing infrastructure. It should use the best information available to support rational decisions and management. It should create more choices for energy sources, aligning a transportation policy to an energy policy. Finally, it should be intermodal in its view and application (see Figure 7).

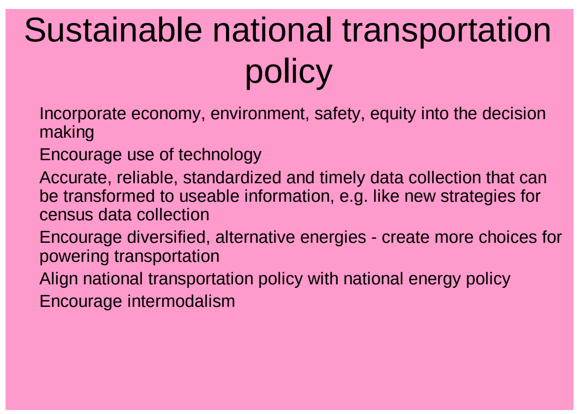

Figure 7. Sustainability recommendations

Education and awareness was a difficult issue. The group noted that we have modal awareness, largely because of the efforts of modal advocates, but we have no champion for freight. It suggested that the United States Department of Transportation (USDOT) should assume that role, working with the states and the various national associations. The message would be focused on the social and economic importance and impact of freight movements. The group sees that next federal bill as FREIGHT-LU, signaling the importance that freight will likely have in it. Now is, therefore, the time to begin spreading the message of the importance of freight (see Figure 8).

Education/awareness of the importance of freight

- Underlying issue: modal awareness (through lobbyists) but no champion for freight
- USDOT should be an advocate for freight by
  - Focusing on state-level decision makers and community leaders
  - Communicating the social and economic impacts of freight transportation
  - Coordinating their messages with freight-related national associations, AASHTO, ATA
  - Designating credible voices for freight at the federal level
  - Communicating the efficiency linkage between logistics network and transportation network
- Next authorization FREIGHT-LU
- Interconnectivity of state level freight action plans
- Define the components of a freight mobility plan

Figure 8. Education recommendations

Technology is moving to help solve the data issues, if we can solve the institutional questions associated with technology. Cell phones, in-truck communications, package and load tracking systems, roadway management systems, and others have the potential for providing much useful and needed information. But we must have a national approach, including funding and standards if this potential is to be realized. This will require partnerships between the federal government and the states and between the public and private sectors. Efforts must be made to meet the needs of each of these partners (see Figure 9).

Data

- Technology and data are linked
- National standards are needed for data
- National financial support is also needed
- A partnership with the states
- A partnership between the private and public sectors

Figure 9. Data recommendations

The discussion of technology focused largely on two issues: making greater use of inland ports and information technology systems (ITS), as shown in Figure 10. Inland ports have the potential for using a wider range of the available infrastructure, thus reducing the growth in congestion at some points and making the total transportation system operate more efficiently. To operate effectively, inland ports, such as the proposed Kansas City Smart Port, must have improved institutional arrangements. Essentially, many government agencies have to coordinate their activities. In the case of Kansas City, the major players are the customs agencies for Mexico and the United States. Inland ports also must have improved interoperability of information systems to improve the flow of date between agencies and transportation providers. And it must have improved cargo tracking capabilities, both for customs purposes and to facilitate intermodal transfers.

ITS has the capability of making real-time information available to both travelers and to the agencies that manage facilities, but to realize its potential, technologies must be operated across jurisdictional boundaries, which requires standards of interoperability and coordination.

Technology

Encourage the use of inland ports

Š Improved institutions
Š Improved data and modal interoperability

ITS

Š Real time information
Š Interoperable standards
Š Better use of WIM and other truck technology
Š Inter-jurisdictional coordination

Figure 10. Technology recommendations

The group suggested three basics approaches to the concerns of energy and emissions (see Figure 11). First of all, the federal government should lead the effort in research to refine and make available alternative energy sources.

Energy and Emissions

Funding for research
Incremental actions
Improved flows

Figure 11. Energy recommendations

Secondly, they suggested an incremental approach that recognizes that smaller steps can have a more immediate and greater impact. This follows the example of California that has mandated or encouraged greater fuel efficiency, alternative fuels and cleaner fuels from technology that is already available. Finally, recognizing that poor traffic flows waste energy and increase emissions, the group suggested a concerted national effort to identify and remove bottlenecks, thus improving traffic flow.

Productivity is often a euphemism for bigger loads and longer vehicles, but the group identified some steps that can improve productivity without venturing into those areas of controversy (see Figure 12). The first deals with a range of technologies, such as next generation CVISN, virtual weight stations, and similar technologies that could combine to make truck enforcement much more efficient and less intrusive.

Related technologies are parking information systems and traveler information systems that would provide drivers better information on where truck parking was available and on the reliability of the highway system, thus allowing trips to be better planned.

Productivity

Less intrusive enforcement
Parking information systems
Information on system reliability
Drowsy driver detection systems

Figure 12. Productivity recommendations

Other emerging technologies, such as drowsy driver detection, could improve productivity by allowing drivers to respond more quickly to their need for rest. It would also, of course, improve safety.

The issues reported by the groups and supported by most workshop participants became the substance of the testimony for the national commission. Once again, the pull between federal and state roles was not resolved: Flexible federal funding that was aligned to federal priorities was recommended. Again, the role of nonhighway modes was somewhat ambiguous, appearing only as "encourage intermodalism." Responses to this question also pointed to another issue in defining a national transportation, or freight transportation, policy, which is the role of the USDOT. A much stronger advocacy is suggested for the USDOT to make the decision makers aware of the challenges we face.

DRAFTING TESTIMONY

A draft of testimony for the national commission was prepared that included most of the points made at the workshop. It was sent to all participants in the workshop for their review and comment. Only state department of transportation and university people responded.

Most comments dealt with editorial issues. A few went to more substantive things. Those substantive comments usually had the result of weakening the message. A couple of states were not fully comfortable with flatly advocating new or higher federal taxes to deal with transportation. Some were not sure that they should or could advocate federal assistance to nonhighway modes. Some argued that the testimony should be more pointed, that is, fewer issues should be raised.

Comments generally indicated a reticence on the part of the states to embrace strong, clear positions. It still raised a number of issues, but the edge was somewhat blunted on several of the more key and controversial of those issues.

ANALYSIS

The survey, workshop, and drafting of the testimony pointed out several issues that will have to be raised, discussed, and resolved if we are to have a national freight transportation policy or any national transportation policy:

- The role of the states vs. that of the federal government
- The role of the public sector in rail
- The role of the USDOT
- The role of the private sector in making government policy

- The role of government as a financier of public works projects

Each of these points deserves explanation and elaboration.

The Role of the States

The states were reluctant to embrace strong national leadership in highways. Given our history, this is not surprising. The United States has never had a tradition of strong national direction in highway transportation. It has historically operated with what has been described as a system of federally aided, state-administered highway construction programs.

The 1956 Interstate Highway Act has often been cited as the last, some might argue the only, instance of articulated and implemented national highway policy. But even it fit well within the previous framework of federally-aided, state-administered. The Congress and the Eisenhower administration agreed that a national interstate system should be built. With the selling point of national defense, they imposed a tax and offered states 90 cents of every dollar spent to build the system. The system itself was defined by the states subject to very broad parameters outlined by the Federal Highway Administration: national connectivity, mileage guides, and design standards. Some states sought approval for every route that might qualify; others laid out more modest systems. Each state built at its own pace, within the allocated federal funding levels. Ultimately, many states withdrew approved interstate projects and used the funding for other purposes. At best the interstate was a federal vision interpreted and constructed through the eyes of 50 states. It achieved the desired outcome of a national freeway system, but the system was molded to fit the needs and desires of the states.

This molding has continued and grown over the years. Starting in 1982, a number of states noted that they were sending much more money to Washington than they were getting back in federal highway aides. These donor states asked for and got minimum guarantees of return on their contributions to the federal highway trust fund. Eighty cents of each dollar was the original guarantee. It is now 95 cents, and some argue it should be one dollar out for one dollar in. This donor argument has dominated national highway politics for the past 25 years. It is not likely to go away anytime soon. The question remains: How can you have a national transportation policy if the role of the federal government is to collect and return taxes to the states?

The donor arguments also raised a number of issues relative to state flexibility. The federal government provides money for specific programs—bridge replacement, interstate highway maintenance, the national highway system, safety, etc. This distribution of money may not fit the states' priorities. When this is the case, can the dollars be used in the programs that reflect the state priorities? Historically, the answer was rarely, but starting with the 1991 Surface Transportation Act, flexibility has been steadily increased. Transfers between federal categories are routine to the point of making those categories fairly meaningless.

On the whole, states' rights, return on contribution, and flexibility have defined national transportation policy debates for decades. The responses of the MVFC states generally reflected that history. When you have spent a career arguing for more authority, more return, and greater flexibility, it is hard to embrace the notion of a strong, federally directed effort.

The Role of the Public Sector in Rail

Few people disagree with the notion that more freight carried on rail is an objective that is in the public interest, but how to achieve that goal is not clear. It is not clear because of the history and traditions of the nation relative to public involvement in rail.

Most railroads in the United States were built by private companies, some with federal assistance in the form of land grants. Private companies operate most rail services. Since government deregulation in the 1980s, railroads have had a fairly free hand to define service areas and types and the rates for that service. They have aggressively tried to become more efficient. Generally, efficiency has meant bigger cars and locomotives, smaller train crews, longer trains, and fewer miles of track. It has also meant less service to smaller places, and often less frequent service. Less service to many locations has also tended to increase the length of truck hauls to the railhead, increasing highway congestion.

For the railroads, all of this means that they are profitable, which is important to a private company. It also means that rails are carrying record amounts of cargo, while their market share is falling.

As we look to a future with more freight tonnage than can be accommodated on many roadways, some have asked how to expand the role of rail. The status quo promises a continuation of the trends of the past 20 years. In fact, longer term projections of freight generally show growing absolute numbers for rail but a smaller market share. Few would argue for a return to the regulation that ensured service but drove most rail companies to the verge of bankruptcy. Short of that, what government policies are available to encourage the use of freight rail? The participants in the process of developing a position on freight policy were reluctant to embrace any solution. Direct public aid to private companies seems to hold a number of problems. Public ownership of rail facilities has even more problems. Aid with performance standards seems more logical, but it is not clear that it is workable.

Clearly much more public discussion is needed to come to grips with this issue.

The Role of the USDOT

Participants in this process suggested a stronger role for the USDOT in two areas. The first, defining standards for interoperability for technical applications, should not be controversial in most quarters. The second, becoming an advocate for freight transportation needs, might come into conflict with the realities of national politics. An advocacy role will almost certainly put the USDOT into a political discussion of the direction of national policy. While many current and former USDOT officials would argue that they have always played such a role, history does not support that view.

The USDOT is a cabinet agency. It tends to focus its activities on those issues, policies, and budget allocations that are supported by the sitting administration. For example, in the most recent reauthorization discussions, the Federal Highway Administration's own analysis clearly demonstrated the need of a much larger federal investment in both highway and transit transportation. Larger investments would have required some additional revenues, which the administration would not support. Not surprisingly, the spokesmen for the agency tried agilely to reconcile the need for more investment with a recommendation for less by noting the need for greater state and local support and the huge possibilities for innovative financing. As a result, members of Congress became the largest advocates for transportation inside the government.

The Role of the Private Sector in Making Government Policy

Those who depend on the transportation system for their livelihood, shippers and carriers, should be the most vocal and effective advocates for that system. Generally, they have not taken an active role in the process. Only eight private sector people took part in the workshop, and most of these were not directly shippers or carriers. None took the time to comment on the draft testimony. The private sector representative who was supposed to have taken part in the delivery of the testimony cancelled at the last minute.

Again this should not be surprising. Shippers tend to treat logistics as a cost to be managed. It is not a key business area. Presidents and CEOs do not come from logistics; they come from finance, engineering, or sales. Carriers, even the very largest, tend to see the world in terms of dealing with their competition and providing service to their customers. As long as everyone is dealing with the same congested transportation system, the field is level.

Competition also reinforces the need to maintain proprietary interests. Shippers and carriers are usually reluctant to take part in group discussions with their peers for fear of losing some competitive advantage. For the same reason, they are often reluctant to share company information with public agencies.

Finally, the slow, deliberative process of the public sector, with its 20-year planning horizons, tends to be nearly fathomless to most private sector people.

This reluctance to participate in public policy issues is beginning to change as our transportation system becomes more congested, but it remains the exception rather than the rule.

The Role of Government as a Financier of Public Works Projects

Historically, public agencies at the local, state, and federal level have been the primary financiers of highway and other public works projects. However, since the 1980s, public policy makers have grown more and more reluctant to impose the taxes that are needed to support public works. Instead, they have advocated greater efficiencies, the use of tolls, or privatization. As a result professional public transportation employees are usually reluctant to take any position on revenue issues before their elected leaders have taken a stand.

Unfortunately, this nation still operates within the framework of publicly financed public works. We do not have the traditions or the institutions to deal with widespread use of tolls or private investment. Without those alternative institutions, we tend to have rhetoric and occasional experimentation, a toll road leased or a concession given here or there. The result is a gradual increase in the fragmentation of the transportation system. Now we have many public agencies, most notably 50 state departments of transportation, controlling transportation system decisions, and a small number of private companies controlling relatively short but key segments of the highway network.

Public discussion and debate is needed to either reinforce the historic, publicly financed system, or to develop new institutions that will allow an alternative system to work.

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Laboratory Evaluation and Pavement Design for Warm Mix Asphalt

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ABSTRACT

For many years, the asphalt industry has been aware of the energy savings and environmental benefits inherent in cold mix or warm mix asphalt technologies. Environmental awareness and the need for improved energy efficiency has increased rapidly over the past few years and extensive measures such as the air pollution reduction targets set by the European Union under the Kyoto Protocol have encouraged efforts to reduce greenhouse gas emissions. Warm mix asphalt (WMA) is one technology that is gaining popularity in the industry in response to these effort. The objective of this study is to evaluate the performance of WMA made Aspha-min using the Mechanistic-Empirical Pavement Design Guide (MEPDG). An asphalt mixture with a nominal maximum size of 12.5mm (1/2") and PG64-28 binder was used. A control mixture, WMA with 0.3% of Aspha-min, and WMA with 0.5% of Aspha-min were tested and the test results were used in the MEPDG. The results from the MEPDG are also discussed.

Key words: Warm Mix Asphalt—Laboratory Evaluation— Aspha-min—Traditional Hot-Mix Asphalt—Mechanistic-Empirical Design Guide (M-EPDG)

INTRODUCTION

The asphalt industry has been concerned about energy savings and environmental benefits in cold or warm asphalt process for decades. In recent years, environmental awareness has been increasing rapidly and extensive measures like air pollution reduction targets set by the European Union under the Kyoto Protocol have encouraged efforts to reduce greenhouse gas emissions. Traditional hot mix asphalt (HMA) is produced in either batch or drum plants at a discharge temperature between 280°F (138°C) and 320°F (160°C). The amount of fuel consumed is relatively large due to the continuous heating of aggregate, thus increasing the energy costs and production of greenhouse gasses. Warm mix asphalt (WMA), a new paving technology that originated in Europe, appears to allow a reduction in the temperature at which asphalt mixed are produced and placed. To be practical, WMA production must use existing HMA plants, specifications, and standards. The current focus is on dense graded mixes for wearing courses. WMA allows the asphalt mixture to be compacted at a temperature range of 250°F (121°C) to 275°F (135°C). Figure 1 shows the compaction temperature for HMA, WMA, and cold mix asphalt. As shown in Figure 1, the WMA temperature is between HMA and cold asphalt mix. There are several proprietary technologies used to produce WMA (FHWA. 2001) which are:

1. Aspha-min®, a product from Eurovia Service GmbH, Bottrop, Germany. It is a synthetic zeolite and creates foaming effect in the binder.
2. WAM-Foam®, a product of a joint venture between Shell International Petroleum Company Ltd., London, UK and Kolo-Veidekke, Oslo, Norway. It is a two-component binder system that introduces a soft and hard foamed binder at different stages during plant production.
3. Sasobit®, a product of Saso Wax (formerly Schümann Sasol) from South Africa.
4. Asphaltan B®, a product of Romonta GmbH, Amsdorf, Germany. It is a low molecular weight esterified wax.
5. Evotherm®, a product developed by MeadWestvaco Asphalt Innovations, Charleston, South Carolina. It is a technology based on a chemistry package that includes additives to improve coating and workability, adhesion promoters, and emulsification agents.

All those technologies reduce the viscosity of the asphalt binder at a given temperature and allow the aggregate to be fully coated at lower mixing temperatures. The application of WMA can have a significant impact on pavement projects in and around non-attainment areas. It was reported that the manufacturers and materials suppliers achieved energy savings on the order of 30%, with a corresponding reduction in CO_2 emissions of 30%. The mixture production and placement temperature could bring several cost, environmental, and performance benefits (Jones 2004). The advantages of the WMA are briefly summarize as reduced fuel cost, reduced mixing and compaction temperature, early site opening, lower plant wear, lesser aging of binder, reduced fumes and emissions, improve workability, and extended paving window.

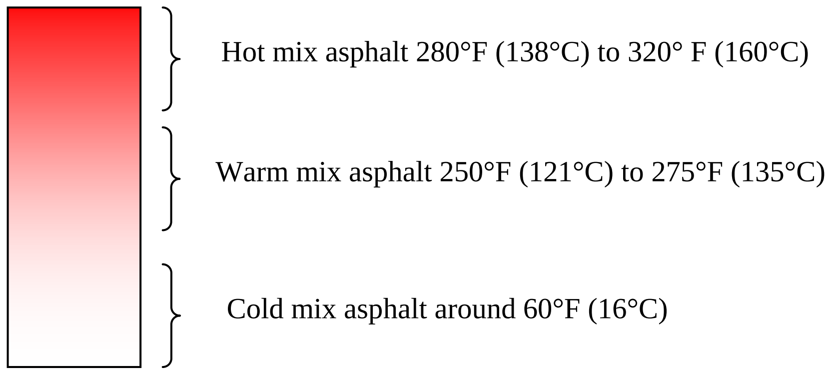

Figure 1. Typical mixing temperature range for asphalt mixtures.

LITERATURE REVIEW

Aspha-min is a product of Eurovia Services GmBH Bottrop, Germany (Von Devivere et al. 2003), often referred to as Eurovia. It is available as a very fine white powder in 25 or 50 kg bags or in bulk for storage in silos. It is a manufactured synthetic zeolite (Sodium Aluminum Silicate), which has been hydro thermally crystallized. Water is held internally by the Aspha-min at 21 percent by mass and is released in the temperature range of 185°F to 360°F (85°C to 182°C). The framework silicates (zeolites) in Aspha-min have large vacant spaces in their structure that allow space for large cations such as sodium, potassium, barium and calcium, and even relatively large molecules and cation groups such as water. In their most useful form, the spaces are interconnected and form several long, wide channels of varying sizes depending on the mineral. These channels allow the easy movement of the resident ions and molecules into and out of the zeolite structure. The most well-known use for zeolite is in water softeners. Zeolite is characterized by its ability to lose and absorb water without damage to its crystal structures. It can have the water in their structures driven out by heat and other solutions pushed through the structure. It can then act as delivery system for the new fluid (FHWA 2007; Kristjansdottir 2006).

By adding Aspha-min to the mix at the same time as the binder, a very fine water vapor is created. This release of water creates a volume expansion of the binder that results in the formation of asphalt foam, allowing increased workability and aggregate coating at lower temperatures (Harrison and Christodulaki 2000; McKeon 2006). Eurovia recommends adding Aspha-min at the rate of 0.3% of the mass of the total mix, which can result in a potential 54°F (30°C) reduction in typical HMA production temperatures. This reduction in temperature was reported to lead to a 30% reduction in fuel energy consumption. Eurovia stated that all commonly known asphalt and polymer-modified binders can be used with Aspha-min. Also, the addition of recycled asphalt is compatible with Aspha-min (Harrison and Christodulaki 2000; McKeon 2006).

A combination field and laboratory study was conducted using Sasobit and Aspha-min to evaluate the performance of WMA (Daniel 2006). The field section with and without Aspha-min additive were placed on the entrance road to Hookset Crushed Stone in November 2005. The samples and cores were tested using the third-scale Model Mobile Load Simulator (MMLS3) to evaluate performance with and without moisture. The laboratory tests using the TSR (Tensile Strength Ratio) showed WMA had higher moisture sensitivity than typical HMA. This project is currently ongoing and further study will be conducted to evaluate the performance of WMA.

A laboratory study was conducted by the National Center for Asphalt Technology (NCAT) to determine the applicability of Aspha-min to typical paving operations and environmental conditions (Hurley et al. 2006). Two aggregates, granite and limestone, were used. The Superpave gyratory compactor and a vibratory compactor were used to determine the mixture compactability over a range of temperatures. Mixes were compacted at 300°F (149°C), 264°F (129°C), 230°F (110°C), and 190°F (88°C), with the mixing temperature about 19°C (34°F) above the compaction temperature. The results obtained indicated that the addition of Aspha-min lowered the air void level in the gyratory compactor, increased the potential for moisture damage, and lowered the TSR (Tensile Strength Ratio) as compared to the control mixture. It was also found that the addition of Aspha-min did not affect the resilient modulus and rutting potential. However, it was indicated that the resilient modulus decreased as the compaction temperature decreased and air void level increased, and the rut depth increased as the temperature decreased for all the factors in combination.

A study on field performance of WMA was conducted at the NCAT test track (Prowell et al. 2007). The results indicated that both HMA and WMA field sections showed excellent rutting performance after the application of 515,333 ESALs over a 43 day period. One of the WMA sections was also evaluated for early opening to traffic and showed good performance.

Researchers have studied the rutting potential and the rheological properties of binders with the addition of Aspha-min and Sasobit (Wasiuddin et al. 2007). The results show that Aspha-min did not give any beneficial effect in temperature reduction in PG 64-22 based on the rotational viscometer results. The rutting potential decreases with a decrease in mixing and compaction temperature for both Sasobit and Aspha-min mixture and no significant direct decrease in production temperature with realized with Aspha-min. In addition, a field demonstration project in Florida indicated that the addition of Aspha-min in the mix has improved the workability compare to the control mix and was also equally resistant to moisture damage as the control mix (Hurley et al. 2006).

There are several Aspha-min comparison tests done by Eurovia. Results of the field test indicated that no significant changes were observed in surface characteristics after three years. The Aspha-min section was comparable to the traditional HMA comparison section (Von Devivere et al. 2003). A field demonstration test was conducted by Hubbard Group in Orlando, Florida in February 2004 (McKeon 2006). The objective for the field demonstration was to compare the conventional HMA with a mix containing the Aspha-min additive at the reduced temperature in a typical paving setting and to compare the workability, compactability, elevator drag strain, and mix volumetric properties. The compaction temperatures used during the test were 310°F (154°C) for the control section and 270°F (132°C) for the Aspha-min section. Aspha-min was added at the rate of 0.3% of the total weight of mixture during this test. The main results obtained from Hubbard Group were:

1. There were no changes in maximum specific gravity or bulk specific gravity for the Marshall and Superpave gyratory compacted specimens when Aspha-min was added.
2. There was a significant increase in air voids before and after the aging process when Aspha-min was added.
3. There was a slight decrease in stability when running the Marshall Stability test whenAspha-min was added.

The conclusions and recommendations drawn from Hubbard Group for the use of Aspha-min are:

1. Comparison of all laboratory tests is favorable with almost no change in volumetric properties and Marshall Stability.
2. The amperage meter dropped from 34 amps to 32 amps on the mix elevator, possibly indicating better workability in the warm mix asphalt.
3. The nuclear density was 2.8 pcf (44.85 kg/m^3) higher after initial compaction in the warm mix.
4. The lower temperature did not change the workability and the material texture was the same.

The combined results of past studies on WMA indicate significant promise in cost saving and reduction of emissions. Although a number of studies have been conducted on WMA, an evaluation of how the use of WMA impacts pavement design using the new MEPDG has not been done. Therefore, the objective of this paper is to use the test results obtained from laboratory tests to perform an evaluation of WMA using the MEPDG software 1.0.

MATERIALS TESTED AND EXPERIMENT DESIGN

In this study, a PG 64-22 asphalt binder was used. The control mixture (HMA) was sampled from the job site and the aggregates were sampled to produce the WMA. The Aspha-min was added to the control mix at the rate of 0.3% and 0.5% based on the total weight of the mixture. The control mixture was compacted at 142°C (288°F) and both mixes with 0.3% and 0.5% Aspha-min were compacted at 100°C (212°F) and 120°C (248°F). The dynamic modulus (E*) test was conducted and the results from the E* test were input in MEPDG. Figure 2 shows the general flow for the experiment design in this study.

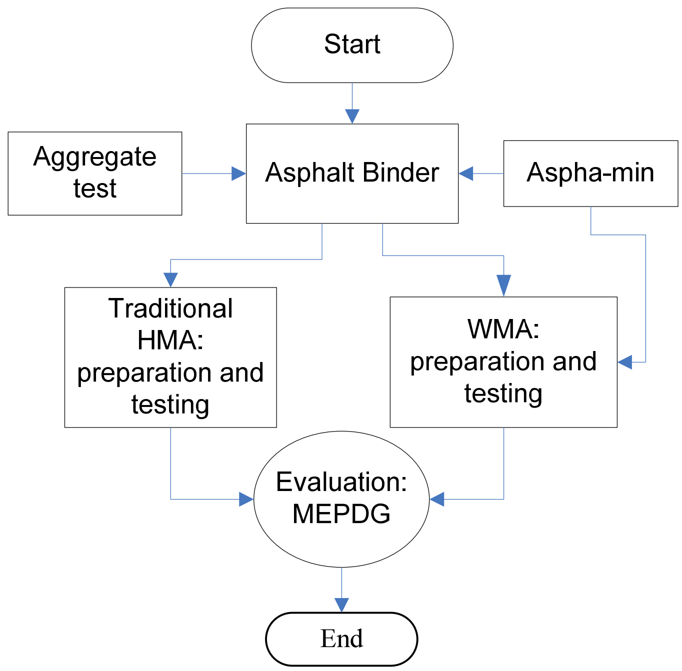

Figure 2. Flow chart illustrating testing and analysis sequence for asphalt mixtures.

DYNAMIC MODULUS

The dynamic modulus (E*) is the ratio of stress to strain under haversine (or sinusoidal) loading conditions and is used as one of the material characterization inputs in the MEPDG to model pavement performance. In the study described in this paper, E* testing was performed according to AASHTO TP62-03. The temperatures used for the measured E* were -5°C, 4°C, and 21.1°C. The frequencies used in this test were 0.1Hz, 0.5Hz, 1Hz, 5Hz, 10Hz, and 25Hz. Five types of mixture were used: a control mixture, 0.3% Aspha-min mixture compacted at 100°C and 120°C, and 0.5% Aspha-min mixture compacted at 100°C and 120°C. The descriptors used to identify these mixtures in this paper are shown in Table 1.

The E* test results are presented in Figure 3. Observations from the graph indicated that most of the mixtures to with Aspha-min added did not significantly affect the E*. In addition, mixture with additional 0.5% Aspha-min and compacted at 120°C have a higher E* based on statistical analysis, pair t-test. This raises question of how much this will impact the development of distress in the pavement. To answer this question, the results E* results were used in the MEPDG to evaluate the predicted pavement performance.

Table 1. Descriptors for each mixture used in the graph and tables.

Descriptor	Description
Control	Control mixture, compacted at 142°C
0.3% AM_100C	Asphalt mixture with 0.3% Aspha-min and compacted at 100°C
0.3%AM_120C	Asphalt mixture with 0.3% Aspha-min compacted at 120°C
0.5% AM_100C	Asphalt mixture with 0.5% Aspha-min compacted at 100°C
0.5% AM_120C	Asphalt mixture with 0.5% Aspha-min compacted at 120°C

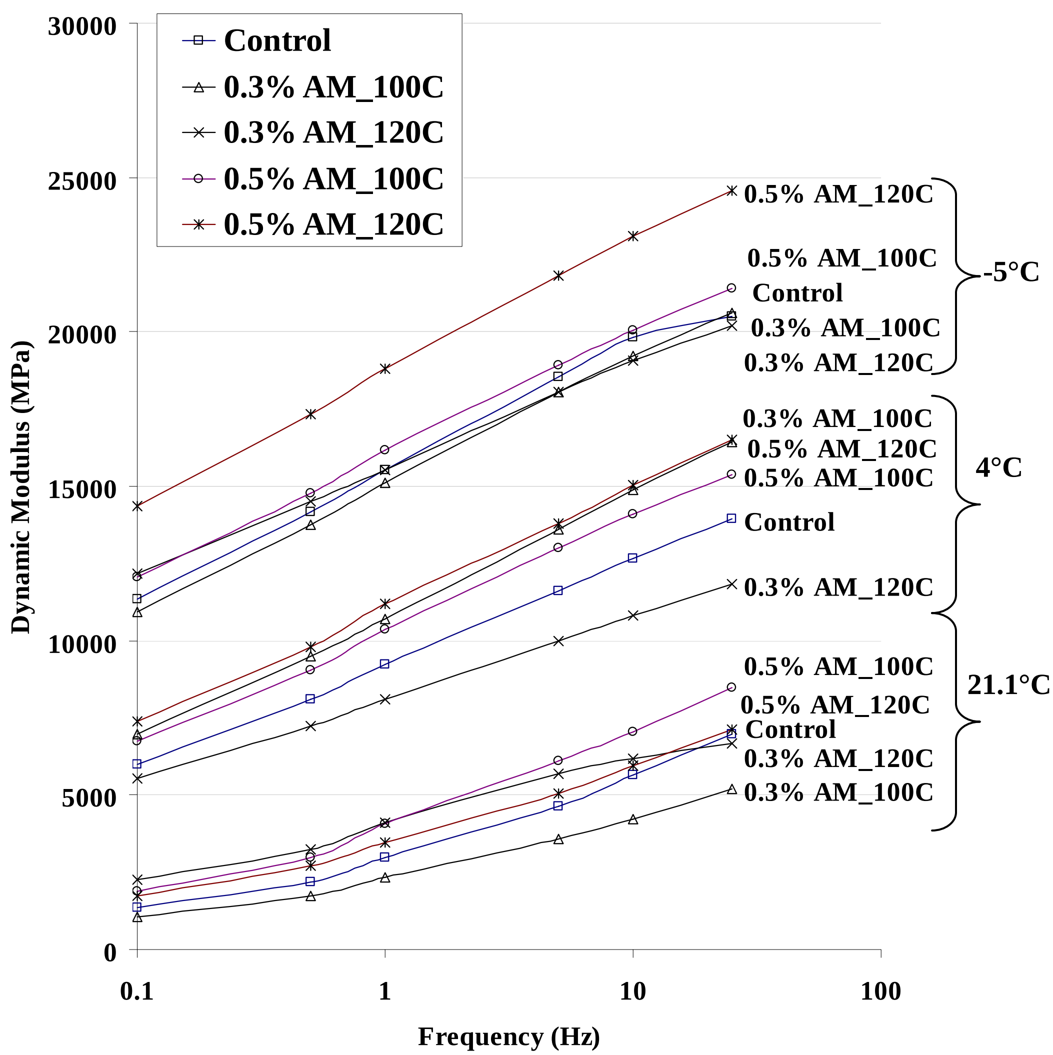

Figure 3. Dynamic modulus test result for the WMA and control mixtures.

MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE (MEPDG)

The Mechanistic-Empirical Design Guide (MEPDG) is being developed under the National Cooperative Highway Research Program (NCHRP) Project 1-37A and is designed to be adopted by the American Association of State Highway and Transportation Officials (AASHTO) for use as the future pavement design guide for the public and private sectors. The development of the MEPDG is based on the collective experience of pavement experts, data from road tests, calculation of pavement response, and mechanistic and empirical pavement performance models (Mulandi et al. 2006; Priest et al. 2005). The MEPDG software is able to predict the development and propagation of various kinds of pavement distress, including rutting and fatigue cracking, using input data on asphalt mixture characteristics obtained from laboratory testing. There are three hierarchical levels in the MEPDG, Level 1, Level 2, and Level 3, with the accuracy of prediction increasing from Level 3 to Level 1. Level 3 is used for the design where there are minimal consequences of early failure and the inputs would be typical average value for the region; Level 2 design is used when resources or testing equipment are not available for the tests required for level 1 and its inputs normally would be user selected possibly from an agency database, could be derive from a limited testing program or could be estimated through correlations, and; Level 1 design is typically used for obtaining inputs for designing heavily trafficked pavements or wherever there are dire safety or economic consequences of early failure. In addition, level 1 input require laboratory or field testing, such as dynamic modulus testing of HMA (Mulandi et al. 2006). In this study, a Level 1 design was used with the measured dynamic modulus as shown in the previous discussions. The assumed values for creep compliance were used for all the WMA and control mix. The creep compliance will most dramatically impact the prediction of thermal cracking. This study focuses exclusively on the development and propagation of rutting. The design pavement life was set at 10 years.

One of the features in the MEPDG is that it allows the user to input very specific climatic data and traffic information. In this study, the climate data for Lansing, Michigan was used. Table 2 presents the temperature data for the surface layer and Table 3 for the base layer. For traffic information, Table 4 presents the traffic parameters assumed for use in this study and Table 5 the vehicle distribution for different classes used in this study. It is noted that Level 3 accuracy for traffic inputs was assumed in the MEPDG software for this evaluation.

The rutting predicted using the MEPDG was used as the pavement distress for comparison in this study. Figure 4 and Table 6 show the results of the predicted rutting depth over 10 years using MEPDG software version 1.0 for each of the mixtures studied. Table 7 shows the percent difference in predicted rutting for each mixture compared to the control mixture.

Table 2. Average monthly quintile temperatures for surface layer in Lansing, Michigan.

Month	1st Quintile (ºF)	2nd Quintile (ºF)	3rd Quintile (ºF)	4th Quintile (ºF)	5th Quintile (ºF)	Mean Temp. (ºF)	Std. Dev. (ºF)
January	13	21	25.7	30.7	38.9	25.9	9.3
February	18.1	26.8	31.9	36.6	44.2	31.5	9.4
March	22.9	31.6	37.4	44.4	57	38.7	12.3
April	35	44.3	51.9	61.2	76.5	53.8	14.9
May	46.3	55.5	62.7	70.8	83.6	63.8	13.4
June	57.6	67.8	75.3	84.9	98.5	76.8	14.6
July	61.7	71.1	79	88.7	100.3	80.2	13.9
August	59	67.9	74.7	84.7	96.9	76.7	13.6
September	49.4	59.3	66.6	75.4	91.1	68.4	14.9
October	37.7	46.5	52.9	60.2	73.3	54.1	12.8
November	28.8	36.1	41.1	46.5	55.2	41.5	9.4
December	20.3	26.8	31.2	35.6	42.9	31.4	8.2

Table 3. Average monthly quintile temperatures for base layer in Lansing, Michigan.

Month	1st Quintile (ºF)	2nd Quintile (ºF)	3rd Quintile (ºF)	4th Quintile (ºF)	5th Quintile (ºF)	Mean Temp. (ºF)	Std. Dev. (ºF)
January	13.4	21.2	25.7	30.6	38.5	25.9	9
February	18.5	27	31.9	36.3	43.6	31.5	9
March	23.3	31.7	37.4	44.1	56.1	38.5	11.8
April	35.7	44.7	52	60.9	75.6	53.8	14.3
May	46.9	55.9	62.8	70.5	82.7	63.8	12.8
June	58.2	68.2	75.3	84.5	97.5	76.8	14
July	62.5	71.6	79	88.3	99.3	80.2	13.2
August	59.8	68.5	74.9	84.2	95.9	76.7	13
September	50.1	59.8	66.8	75.1	90.1	68.4	14.3
October	38.3	46.8	53	60.1	72.6	54.2	12.3
November	29.3	36.3	41.2	46.4	54.7	41.6	9.1
December	20.7	27	31.2	35.6	42.6	31.4	7.9

Table 4. General traffic inputs for MEPDG.

Description	Value
Initial two-way AADTT:	1000
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	95
Operational speed (mph):	60
Mean wheel location (inches from the lane marking)	189
Traffic wander standard deviation (in):	10
Design lane width (ft):	12
Growth Rate	4%
Growth Function	Compound

Table 5. Assumed AADTT distribution by vehicle class.

Classification	Percent Distribution (100% Total)
Class 4	1.8%
Class 5	24.6%
Class 6	7.6%
Class 7	0.5%
Class 8	5.0%
Class 9	31.3%
Class 10	9.8%
Class 11	0.8%
Class 12	3.3%
Class 13	15.3%

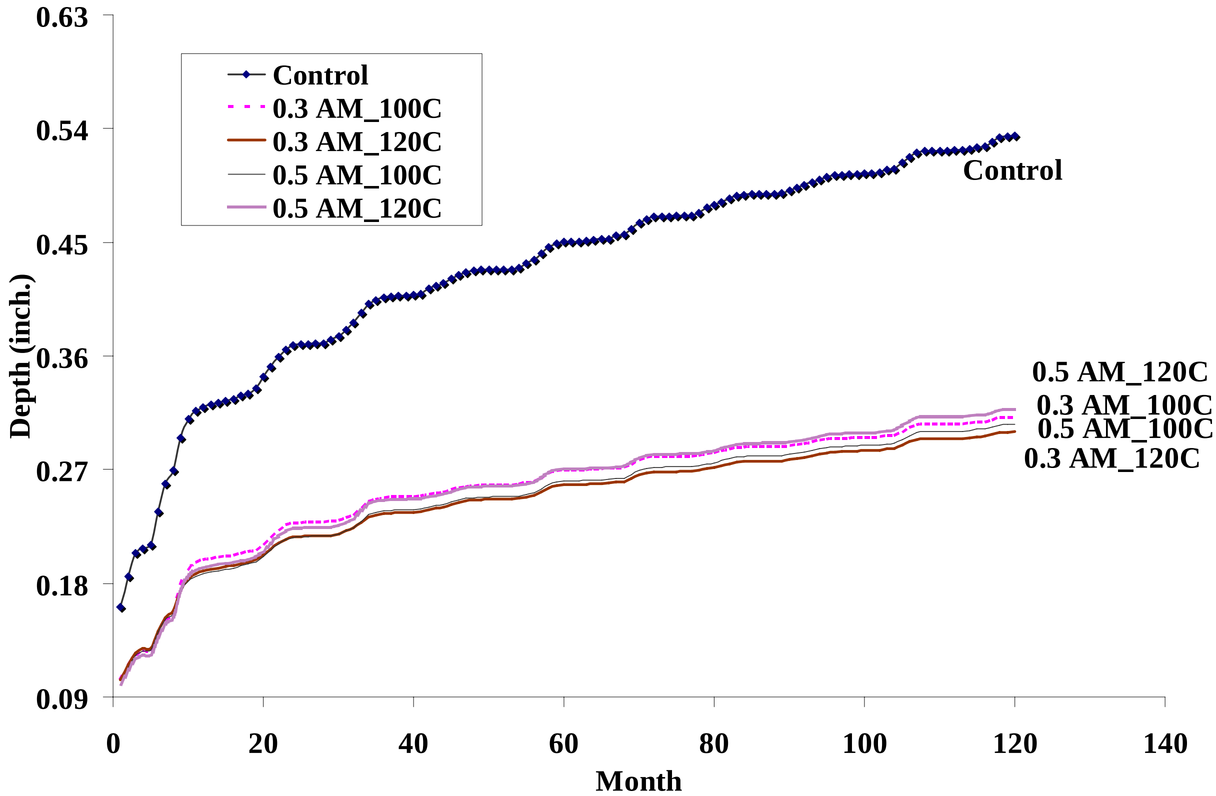

Figure 4. Prediction of rutting depth over 10 years using MEPDG Software Version 1.0.

Table 6. Prediction of rutting depth using MEPDG Software Version 1.0.

	Mixture				
Year	**Control**	**0.3AM_100C**	**0.3AM_120C**	**0.5AM_100C**	**0.5AM_120C**
1	0.3197	0.1984	0.1899	0.188	0.1932
2	0.3684	0.2268	0.2165	0.2159	0.2233
5	0.4502	0.2694	0.2579	0.2603	0.2701
10	0.5345	0.3112	0.3	0.3059	0.3179

Table 7. Percent difference in rut depth for each mixture compared to the control mixture.

	Mixture			
Year	**0.3AM_100C**	**0.3AM_120C**	**0.5AM_100C**	**0.5AM_120C**
1	38%	41%	41%	40%
2	38%	41%	41%	39%
5	40%	43%	42%	40%
10	42%	44%	43%	41%

From Figure 4, it is observed that the depth of rutting increases rapidly during the first 20 months with a decreasing rutting rate thereafter. It is also observed that the predicted rutting depths for all WMA mixtures are higher than for the control mixture. Table 6 shows the result of predicted rut depth at 1 year, 2 years, 5 years, and 10 years for asphalt pavement using the MEPDG. It was found that the additional Aspha-min improves the rutting resistance significantly. Table 7 reveals that in the percent difference for WMA compared to the control mixture. The greatest different for WMA and control is approximately

44% (compare with WMA made with 0.3% Aspha-min compacted at 120℃).These results are based on many assumptions and should be considered as preliminary results. Further study is ongoing to verify the mixture properties, mixture design, and pavement field performance.

CONCLUSIONS

WMA had shown significant promise in lowering the required mixing and compaction temperatures while decreasing emissions for asphalt pavement construction. The literature review indicated that WMA with Aspha-min has improved workability, decrease cooling time after construction, and in general allowed for a reduction in mixing and compaction temperature (although some studies indicated no reduction). In this study, it was found that the addition of Aspha-min does not affect the value of E* for all mixtures examined. In addition, WMA decrease the predicted depth of rutting based on the Level 1 analysis using the MEPDG and the greatest different for WMA and control was found to be 44%. This might give a potential in improving the rutting resistance for the future pavement design. Future research to gain greater understanding of both the short-term and long-term aging of WMA as well as how well this is modeled in the MEPDG needs to be conducted. To accomplish this, a thorough analysis of distress data collected from field projects needs to be undertaken

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Real-Time Traffic Operations Data Using Vehicle Probe Technology

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ABSTRACT

The vehicle probe industry is emerging as a viable means to monitor traffic flow, delivering both speed and travel-time information for the purposes of advanced traffic management systems and advanced traveler information services applications, as well as supporting a myriad of other transportation agency requirements, including monitoring the impacts of construction activities, planning, and engineering. Meanwhile, the high cost of installing and maintaining fixed-point loop detectors is driving transportation authorities to consider both outsourcing traffic monitoring and developing new methods of detection. Vehicle probe technology, as discussed in this paper, encompasses two primary methods: GPS data obtained from fleet management services and geo-location schemes that leverage cellular phone infrastructure. The proliferation of GPS and mobile data services is fueling these industries and strengthening the demand from travelers for accurate real-time traffic information. One of several ongoing procurements for such data is sponsored by the I-95 Corridor Coalition. As technical advisor to the Coalition, the University of Maryland assisted in developing an appropriate procurement strategy for acquiring vehicle probe data services. This paper reports on various aspects of vehicle probe technology, including technology differentiation, lessons learned from previous demonstrations, risk assessment, intellectual property issues, and institutional barriers for adopting and leveraging new technology.

Key words: cellular—GPS—traffic data—vehicle probe data

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INTRODUCTION

Vehicle Probe Technology

Vehicle probe technology is emerging as a means of monitoring traffic without the need for deploying and maintaining equipment in the right-of-way. In contrast to speed sensors, vehicle probes directly measure travel time using data from a portion of the vehicle stream. Commercial vehicle probe data services primarily include the use of cell phones and automated vehicle location (AVL) data. Early demonstrations of such systems relied heavily on a single method or technology. However, services are emerging that combine information from multiple probe sources and technologies, as well as data from existing fixed-sensor networks, into a comprehensive traffic information service.

Adoption of such technologies is being driven by the high cost of deploying and maintaining fixed-sensor networks, including loop- or radar-based detection. Concurrently, demand for comprehensive traffic monitoring is growing, both from travelers, who need accurate, real-time data, and transportation agencies, which need to assess the performance of the system as a whole. The cost for gathering traffic data, either from probe-based or traditional speed sensors, is declining due to both the proliferation of technology and the emergence of businesses dedicated to traffic data collection and dissemination. Traffic data collection within a transportation agency has traditionally been application-specific and geographically constrained, such as with the need to actuate a traffic signal or collect speed and count data for planning purposes. This “stovepipe” method is being replaced by comprehensive traffic monitoring across the entire roadway system. Such an approach feeds not only legacy applications, but also supports the growing demand for advanced traveler information services (ATIS) data, such as travel time and congestion reports, and performance measurement data that assesses and improves the efficiency of existing highway operations.

Vehicle Probe Technology as Used by the I-95 Corridor Coalition

The I-95 Corridor Coalition is a partnership of state departments of transportation (DOTs), regional and local transportation agencies, toll authorities, and related organizations, including law enforcement, transit, port, and rail organizations. The partnership area ranges from Maine to Florida (including the District of Columbia), with affiliate members in Canada. I-95 Corridor Coalition members work together to reduce congestion, increase safety/security, and ensure that the entire transportation network supports economic vitality throughout the region. In order to achieve this mission, the Coalition initiated a regional traffic monitoring system in 2006 that will act as a continuous source of real-time transportation system status information along a major portion of the corridor. Construction and maintenance of the system was outsourced. Rather than specifying a particular technology in the request for proposals (RFP), the technical requirements for the system were based on the need to support a broad range of ATIS, advanced traffic management systems (ATMS), engineering, and planning applications for the Coalition and its members without deploying additional infrastructure in the right-of-way. However, with outsourcing comes the burden of defining data ownership and usage and dissemination rights and restrictions. By specifying the minimum data rights needed to support all intended applications, the RFP allowed vendors to individually craft approaches that satisfied the needs of the Coalition while protecting the commercial viability of the traffic data for the vendor. These aspects, as well as an evaluation methodology that emphasizes risk management and requires demonstrated ability to meet technical specrications, promise to deliver an effective traffic monitoring resource for the East Coast of the United States.

TECHNOLOGY DIFFERENTIATION

As a whole, probe vehicle technology differs in concept from spot speed sensors in several respects, as summarized in this section.

Probe Vehicle Technology

Key aspects of cell phone and AVL probe technology include the following:

- Vehicle travel time is measured directly.
- Only a sample of all vehicles is monitored.
- Speed is inferred from travel time.
- Volume is inferred from sample size.
- Speed estimates are space-mean speed (as opposed to point speed or time-mean speed)
- Roadside infrastructure is minimized or eliminated.
- Quality of data is based on the percent of vehicles monitored.

Each probe vehicle technology possesses unique characteristics, as summarized below.

Cell Phone Probes

Cell phone probes cover any method used to infer the location of vehicles by use of cell phones and their associated tower infrastructure. Methods vary between vendors and fall within two broad categories. Signaling information, such as tower hand-off timing, is the most prevalent category, while the other category uses embedded assisted GPS technology within user phones. Whichever method is used, any cell phone approach requires a partnership with a major cell phone carrier within the region. This reliance has proven to be a critical risk factor in more than one demonstration project. Demonstrations in the United States and deployed systems abroad have proven the technology's ability to monitor traffic flow on freeways (Haghani, Yang, Hamedi 2007). Smith and Fontaine (2006) provide a summary of the results of various demonstrations in North America, for the reader's reference.

However, results on lower class roadways have shown less success. Cell probes have difficulty differentiating the traffic between closely spaced facilities, such as between frontage roads and the adjoining freeway. Moreover, no known cell phone demonstration to date has been able to consistently and successfully assess traffic on signalized arterials.

Automated Vehicle Location Services

AVL system information is gathered by established commercial businesses. Such systems rely on GPS receivers to track individual vehicles in a fleet, and locations are periodically reported via satellites, radios, or cellular data services. The proliferation of low-cost wireless data services, combined with the reduction in price of GPS receivers (driven by the consumer market), has increased not only the number of fleets utilizing AVL, but has also increased the reporting frequency of vehicles. Initial AVL systems adopted by long-haul trucking fleets relied on satellite communications and reported location once every 30 minutes, on average. Current systems report location more frequently, such as once every 5 to 10 minutes. With the reduction in costs, AVL systems are being used in regional fleets, such as taxis, buses, and short-haul truck delivery. As a result, the growth rate of GPS data available from AVL services is estimated to be 60% to 100% per year. The quality of traffic information derived from AVL data depends on the quantity and distribution of vehicles reporting through AVL systems. These distributions tend not to be uniform. For example, long-haul trucking tends to avoid peak hours in metropolitan areas. As a percentage of traffic, long-haul trucking tends to be low during peak traffic demand and high during off-peak and nighttime hours. Other fleets reporting AVL data exhibit their own patterns and peculiarities depending on the nature of the fleet business.

Toll-Tag Technology

A third class of probe vehicle technology exists, based on automated toll-tag systems. This technology shares the same attributes as the cell phone and AVL probes, but requires additional toll-tag readers to be deployed in the right-of-way. Unlike cell phone probes or AVL data, toll-tag systems are owned and maintained by road authorities or organizations closely aligned with public transportation management.

Probe-based Technology Market

The market trend is currently not leaning toward any single technology. Vendors often merge or blend data from multiple sources, including data obtained from fixed-sensor networks owned by public entities, with data collected using their own proprietary collection methodologies and possibly with data from partners who are able to augment the data collection network for specific geographic regions or specific types of roadways. Costs of data from probe-based services cannot be assessed at this time. All of the deployments to date in North America have been demonstration projects, although Wisconsin is currently deploying an operational system that is not yet active. Project costs have ranged from $200 to $5,000 per mile per year for collected data.

Fixed-point Speed Sensors

In contrast, fixed-point speed sensors (loop detectors, in particular) generally have the following attributes:

- Traffic volume and occupancy is measured directly.
- Traffic speed is inferred from occupancy based on an average vehicle length.
- Travel time is inferred from a network of sensors.
- Quality of travel time data is dependent on the density of the sensor network.
- Equipment in the right-of-way is required.
- Cost of deployment has historically been high.
- The technology has been historically maintenance-intensive.

Several new fixed-point sensor technologies are emerging based on radar, acoustics, and other sensing concepts. Such systems are less intrusive to deploy and maintain because such devices are mounted away from the roadway on adjacent structures, such as a light pole or overhead sign truss. This minimizes installation costs as well as traffic disruption during installation and maintenance, and it avoids pavement penetration. In some instances, wireless data communications and solar and wind power technology have been integrated to further reduce the cost of the supporting infrastructure. Vendors have introduced new business models to allow for outsourcing so that the road authority purchases only a data subscription (similar to that of probe vehicles) while the vendor is responsible for installation and maintenance. Although technology has enhanced the cost competitiveness of point detection, these sensors are still subject to the same fundamental constraints as loop detectors in that the quality of traffic data is proportional to the density of sensors in the study area, and equipment is required in or immediately adjacent to the right-of-way.

PRESSURES TO ADOPT PROBE-BASED TECHNOLOGY

Pressures are converging on state DOTs and road authorities to consider probe-based technologies.

The first pressure, as discussed above, is the historic cost of owning and maintaining a network of fixed-point sensors, particularly magnetic loop detectors. The large expense of deploying fixed-sensor networks and the fiscal and manpower burden of maintaining such networks are driving state DOTs and regional and municipal road authorities to consider probe-based services. New speed sensor technology (such as acoustics and radar) is remedying some of the concerns of spot-speed sensing based on loops. However, spot-speed sensing is inherently limited to the spatial deployment of sensing stations and cannot scale geographically as easily as probe-based solutions do at a lower cost.

Second, the primary responsibility of transportation agencies has been, and continues to be, the construction and preservation of the travel way. Under fiscal and manpower constraints, outsourcing typically occurs in skill areas that are not among the core competencies of the vested employees. At present, the sensing and data gathering functions needed to support ATIS and ATMS fall into this category.

Third, the proliferation of low-cost wireless data communications is also fueling the appetite of consumers for real-time travel data and timely reporting of slowdowns and road closures across the entire highway system. Probe methods offer a viable means of acquiring a systemwide view without the investment of a massive fixed-sensor network. Customer satisfaction was once based primarily on the quality of ride and the extent of the highway network. However, customer satisfaction, particularly in congested metropolitan areas, is now based on efficient management of limited highway capacity and communication of such data to customers to allow them to avoid slowdowns related to incidents and congestion.

In summary, traffic data collection within a transportation agency has traditionally been application-specific and geographically constrained, such as the need to actuate a traffic signal or collect speed and count data for planning purposes. This “stovepipe” approach is being replaced by comprehensive traffic monitoring across the entire roadway system that serves many applications. Such an approach feeds not only legacy applications, but also supports the growing demand for ATIS data, such as travel times on variable message signs, 511 and web 511 information, and performance measurement data that assesses the efficiency of existing highway operations.

I-95 TRAFFIC MONITORING PROJECT

The I-95 Corridor Coalition initiated a vehicle probe project in 2006 to provide comprehensive, multistate traffic flow monitoring along the corridor. The objective is the acquisition of traffic flow information based primarily on probe technology for both freeways and signalized arterials. The information produced by this project will be used to support a number of Coalition activities, such as corridorwide traveler information, incident management, and performance measurement. The wide-area coverage provided by this project is designed to support the unique planning, engineering, and operational needs of a heavily traveled corridor.

Member agencies will benefit from the vehicle probe project by receiving traffic flow information relevant to their respective jurisdictions. It is anticipated that they will use the information to support the operation of 511, display travel times on variable message signs, and manage traffic during incidents. Coalition members will also be able to utilize the contract developed for this project to expand coverage within their jurisdictions, develop information websites, and interface with existing traffic management systems.

By pooling the resources of several states, this project attempts to bridge jurisdictional boundaries in order to provide long-distance travelers with information relevant to inter-jurisdictional highway travel. Additionally, this project will provide the information needed to support implementation of long-distance diversions that are characteristic of major incidents that have a multistate impact.

The Coalition will contract with a probe data provider that will be selected based on a review of proposals. The RFP was released on April 27, 2007. It is anticipated that a contract will be awarded by the end of 2007, with traffic data available by summer 2008. The contract will be based on the purchase of data and does not include procurement of any hardware or software, except for the ancillary services that may be requested by member agencies.

Critical points of the RFP are summarized as follows:

- No particular probe technology is specified. The approach is limited only to methods that do not require additional physical equipment to be located in the right-of-way. Vendors can take advantage of data from existing systems that rely on field assets, such as loops, radar, or toll-tag systems.
- Specifications regarding the quality of the data were determined based on the intended uses of the data. The specifications limit the error in reported speed (and associated travel time) under varying roadway conditions.
- Data service will be validated by an independent agent on behalf of the Coalition.
- The vendor must supply a risk assessment for both the vendor and the Coalition. If service is dependent on third party contracts, evidence of the sustainability of such contracts is required.
- The vendor retains full ownership of data for resale in the commercial market. Minimum data rights are defined to support the intended applications within the Coalition. Vendors may propose additional restrictions (or fuller rights to the data) in the proposals. Any additional data rights (or restrictions) will be assessed as part of the RPF evaluation process.
- The vendor may provide data using any one of a number of common formats, technologies, and data standards. However, the vender must be able to transform or translate that format into whatever format is needed for integration into Coalition members' data systems as part of ancillary consulting services. The ability to transform the data format into ITS standard protocols is required.
- The base contract (and associated funding) is planned for the first three years, with options to renew for an additional seven years. Supplemental funding to extend the contract beyond the initial three years is to be provided by Coalition members. Supplemental funding is not guaranteed, but based wholly on the success of the project and its critical role in corridor operations.
- Coverage will include I-95, beltways, parallel freeways, parallel signalized arterials, cross-linking freeways, and cross-linking arterials. The Coalition prefers full coverage on all road classes for a limited geographical area, rather than coverage of only higher class facilities along the whole corridor.
- Evaluation and award of the contract will be based on the best value for the Coalition.

CONCLUSIONS

Traffic monitoring through the use of probe vehicle technology is emerging as a viable means of developing comprehensive traffic monitoring systems without a large investment in physical assets deployed in the right-of-way. Although new methods for detecting speed and volume are lowering installation costs and minimizing maintenance, probe-based methods of measuring travel time can easily scale across large networks without additional infrastructure in the right-of-way and its associated costs and maintenance burden. Probe vehicle technology is fundamentally different than fixed-point detectors, in that probe technology provides a direct measure of travel time, while any method of fixed-point detection infers travel time from a network of speed sensors.

Demonstrations of probe technology have been successful for freeway applications, but the technology remains unproven for signalized arterials. The I-95 Corridor Coalition is moving forward with an aggressive program to procure travel time and speed data through an outsourcing program that utilizes technologies that do not require road-side equipment. Vendors are not restricted from using existing fixed-sensor data. The specifications for quality of data, ownership, and dissemination rights were determined based on the intended applications of the data within the Coalition, allowing the vendor to propose innovative solutions with minimal constraints. If successful, the program will provide a means for the Coalition and its members to procure the quality traffic data to meet the expectations of its customers, support legacy applications, and assess performance of existing infrastructure to enable planning and engineering.

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Kansas Department of Transportation’s Experience with Procuring Wi-Fi at Rest Areas

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ABSTRACT

In 2005 the Kansas Department of Transportation embarked on a public-private partnership to acquire wireless Internet (Wi-Fi) services at Kansas rest areas. The effort is now reaching fulfillment, in that initial installations are in place and functional. A long-term contract with a Wi-Fi provider has been established, a web portal has been developed and implemented, and assessment of the operation is about to commence. This paper reviews several aspects of the project, including stakeholder interests, objectives, procurement method, impact on rest area operations, lessons learned, and projected benefits, such as an expanded communication infrastructure and outlets for local and regional travel information.

Key words: rest areas—Wi-Fi—wireless Internet

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INTRODUCTION

This paper documents the efforts within the Kansas Department of Transportation (KDOT) to secure wireless Internet (Wi-Fi) capability at state rest areas through a public-private partnership. The effort began in the spring of 2005 and is now coming to fulfillment in 2007, as service at four rest area locations is being activated in a pilot test. This service, referred to as Wi-Fi at rest stops, is projected to provide several benefits for KDOT, other state agencies, the traveling public, and the private sector partner. This paper documents the timeline, stakeholder interests, objectives, and lessons learned in the process of securing this service for the state of Kansas.

DEVELOPMENT OF THE IDEA

The initiative began in March 2005 at the Intelligent Transportation Systems (ITS) Heartland conference hosted in Topeka, Kansas, where a presentation by Mark Wheeler from I-Spot Networks, LLC, outlined a program by which Iowa was able to provide a similar service for Iowa rest areas. I-Spot Networks proposed a plan to install wireless Internet service at all Iowa rest areas at no charge to the state. The public would gain free access to Internet content, and the private firm would recoup and profit through advertising revenue. Although this business model would ultimately prove unsustainable, the system was initiated and deployed statewide in Iowa and, by 2005, was fully functional. The presentation at the ITS Heartland conference stirred interest among KDOT attendees.

On April 26, 2005, KDOT invited representatives from the Iowa Department of Transportation and I-Spot Networks to speak at KDOT headquarters. Representatives from various divisions and bureaus within KDOT were invited to attend, as well as representatives from the Kansas Highway Patrol, the Kansas Department of Commerce, and the Federal Highway Administration. About 20 to 30 people attended the initial meeting. Unlike Iowa, administration of the state’s rest areas is not consolidated to a single office. Responsibility for Kansas rest areas falls primarily under KDOT’s Division of Operations. While contracts for cleaning, vending, and staffing are centrally administered, mowing and other concerns are addressed by the corresponding KDOT districts.

At the April 26, 2005 meeting, Steve McMenamin, Iowa Rest Area Coordinator, presented the Iowa program from the state’s perspective, in terms of benefits and risks to the state. The private vendor, I-Spot Networks, presented the business model through which the services were procured. At the same time as the meeting, I-Spot Networks presented a proposal to KDOT to extend the basic service to Kansas rest areas under similar terms as that of Iowa, though no action was taken on the proposal.

A follow-up meeting scheduled for May 17, 2005, brought together the various Kansas stakeholders in the project. This group of representatives eventually constituted the core of the project steering committee and consisted of representatives from the following stakeholder agencies:

- Kansas Department of Commerce, Division of Travel and Tourism
- Kansas Highway Patrol
- Federal Highway Administration
- Kansas Turnpike Authority
- KDOT, Division of Operations, Bureau of Construction and Maintenance, Bureau of Materials and Research, Information Technology Support
- KDOT, Division of Design, Environmental Section
- KDOT, Division of Administration, Bureau of Fiscal Services, Bureau of Computer Services
- KDOT, Division of Planning, Bureau of Transportation Planning, ITS Unit

- KDOT, Office of Chief Counsel
- KDOT, Public Information, Advanced Traveler Information Systems

OBJECTIVES, BENEFITS, AND RISKS OF WI-FI SERVICE AT KANSAS REST AREAS

This meeting on May 17, 2005, served two primary purposes. It helped focus the objectives and expected benefits of providing Wi-Fi at rest areas, and it identified risks and potential negative consequences that should be guarded against. These objectives, benefits, risks, and consequences are outlined below.

Objectives and Benefits

- Wi-Fi provides or improves access to traveler information to aid travelers in making informed decisions to improve safety, make travel more efficient, and increase customer satisfaction. This may include free access to public information, such as Kansas 511 (road condition and road closure information), Amber Alerts, weather conditions and forecasts, and state tourism information. It also includes access to private traveler services, such as hotel reservations, dining, and other local attractions.
- Wi-Fi provides Internet access by subscription to the traveling public for general purpose viewing. This includes access to email and other websites that are not necessarily travel-related. This provides a valuable service and encourages fatigued motorists to take needed breaks.
- Provide KDOT personnel access to Internet connectivity for KDOT business purposes. This, in essence, extends to the field some of KDOT's enterprise applications, such as the Construction Management System, Comprehensive Project Management System, and KDOT email. KDOT performs several field data collection activities, and Internet access would allow for convenient and more frequent uploads of data to central systems, as well as access to department data while in the field.
- Wi-Fi provides Internet connectivity to other safety/emergency personnel. This is primarily seen as a benefit to the Kansas Highway Patrol. Although Kansas has a statewide radio voice system, mobile data is extremely limited, except in urban areas. Wi-Fi at rest areas would provide the state patrol with "islands" of information connectivity distributed throughout the state. Not only would this benefit the highway patrol, but it may also increase the frequency of patrol presence at rest areas and thus enhance rest area security.
- The Kansas Department of Commerce operates the Kansas State Welcome Centers. The western-most center is co-located with a Kansas rest area. The commerce department views Wi-Fi not only as a valuable attribute to have at the welcome center, but also as an effective method for promoting Kansas goods and services through use of the free portal provided by the Wi-Fi project.
- Information kiosks at Kansas rest areas are a longer term objective. Previous demonstrations used satellite feeds to deliver weather information to rest areas. Data communications capability via Wi-Fi, particularly in rural areas where data connectivity options are scarce, is a major step towards enabling information kiosks.
- Similar to information kiosks, data connectivity at rest areas opens the possibility for several other potential services and applications. Although none are specifically planned, Wi-Fi can enable such services as emergency telephones, remote surveillance, changeable message signs, and traffic sensing at rest areas.

Risks and Possible Negative Consequences

- Kansas rest areas, particularly rural rest areas on interstate routes, have a history of deviant activity during nighttime hours. This includes both criminal activity, such as prostitution and drug trafficking, as well as noncriminal but otherwise undesirable behavior, such as lewd and lascivious activity and damage and destruction to the facilities. By providing additional data connectivity, the DOT risks further enabling such activity.
- Truck parking facilities at Kansas rest areas are insufficient to handle demand. Trucks can be seen parked on shoulders of on-ramps and off-ramps at many Kansas rest areas, particularly during nighttime hours. Such practices create a safety hazard to the traveling public. The risk is that Wi-Fi will attract even more commercial truck traffic, exacerbating the problem.
- Publicly available Internet access may create a financial liability. A bystander may inadvertently view a website on another traveler's computer screen that he or she considers inappropriate and offensive. If Wi-Fi connectivity is viewed as a public service provided by KDOT, access to the objectionable content would be seen as state endorsed. Such a scenario could possibly create both a financial and political liability.
- Advertising as a method of revenue generation for the vendor creates additional risks for KDOT. If the service is viewed as a public service, the product or service advertised could be viewed as state endorsed. The nature of the advertised product and services, particularly if they border on adult content, have the potential of creating a public relations risk for KDOT.
- The business model of the vendor may become unsustainable, as exhibited in Iowa.
- Nonstate telecom equipment would need to be mounted and integrated into the KDOT rest area infrastructure. Improper or careless installation could cause harm to the structures. (This was not initially brought up as a project risk, but was identified later during the installation phase.)

Other Considerations

In addition to the objectives and risks mentioned above, the project also provoked a debate concerning the most appropriate method for obtaining Internet connectivity at state rest areas. The KDOT information technology department suggested that the state may be best served by supplying the data link using state resources rather than through a private company. Although there were many considerations in this debate, the consensus was ultimately reached to view Wi-Fi in much the same way as the vending services that are supplied at rest areas, i.e., through a consolidated service contract. This not only limits the manpower resources needed to maintain the system, but also provides a layer of liability protection in light of the risks previously expressed. Also, any attempt to install the Wi-Fi services using state resources could not take advantage of any potential revenue stream in order to pay for equipment and upkeep. Although the Iowa model relied wholly on advertising revenue, other business models, such as that employed in Texas, have allowed the private vendor to sell subscriptions for unrestricted Internet access as a way of recouping costs.

The steering committee met again on June 23, 2005. The risks of the project were mitigated through various methods, as discussed below.

RISK MITIGATION STRATEGIES

- The primary concern of the KDOT Division of Operations was possible adverse effects on rest area operations. This included both the truck parking problem and issues related to deviant behavior. Anecdotal observations from Iowa's program indicated no undue effects. However, differences in rest area management (Iowa rest areas are typically staffed 24/7) and rest area design between the two states provided a reason for concern. As a result, the Kansas Wi-Fi program was structured as a one-year pilot test at a limited number of rest areas, with the option of renewing the contract for up to an additional four years with extended service to the entire state. The one-year pilot program allowed KDOT to assess the impact on rest areas. If the impact was negative, KDOT could choose not to renew the contract and thus effectively terminate the program. Also, as part of the technical requirements of the request for proposals (RFP), a mandatory time restriction for individual Internet access was to be enforced at the rest areas.

- Concerns over liability from objectionable content were discussed, not only with stakeholders, but also with KDOT executive management personnel. Ultimately, KDOT chose to go with a business model that required travelers to purchase a subscription to access general Internet content. Free access was provided to a traveler information portal that contained all the traveler safety, service, and tourism information appropriate to KDOT's role as a traveler information provider. Subscription not only offered a method to recoup costs, but also demarked the line between KDOT-provided and endorsed content and content obtained through the purchase of access rights from a private company. Several issues factored into the decision, but liability concerns did influence the ultimate decision in favor of a subscription-based model.
- The sustainability of the business model was considered a risk of the vendor and did not influence the development of the RFP or the evaluation of the proposals received in response to the RFP.
- Installation of the equipment into state-owned facilities grew in importance as the project progressed. The selected vendor used a subcontractor for installation of equipment. Coordination between the vendor, the subcontractor, and various KDOT offices was a major challenge during installation. See the comments in Lessons Learned below for further details.

CONCURRENT INITIATIVES

At the time the KDOT effort began, two other states were known to have deployed systems similar in concept and scope as the system envisioned for Kansas. Those states included Iowa and Texas. However, several other states were in various stages of procurement, similar to KDOT.

As mentioned above, Iowa contracted with I-Spot Networks to provide Wi-Fi connectivity at all Iowa rest stops. The system allowed travelers to view a traveler information portal and access the Internet for free after providing registration information. Revenue from advertising was projected to sustain and provide profit. However, advertising revenue alone was incapable of sustaining the program. In 2006, Iowa had to revisit the program and make alterations in order to continue service.

Since the first business model proved unsustainable, to mitigate future risk the Iowa DOT decided to purchase all of the equipment and to contract directly with Internet service providers for wireless access. Iowa has contracted with another provider, Zoom, for technical support, software development, and kiosk installation.

The Texas Department of Transportation contracted with a vendor called Coach Connect, which had previously established a successful business practice delivering Wi-Fi service to recreational vehicle parks across the country. The business model for Coach Connect was to provide the traveler information portal free of charge, but to require a reasonable access fee to obtain unrestricted Internet access. Advertising was also part of the business plan.

DEVELOPMENT OF THE WI-FI SERVICE NETWORK IN KANSAS

KDOT Procurement Methodology

The RFP was developed and released in the winter of 2005/2006. The RFP was developed with input from the steering committee (as described above), KDOT executive staff, and the project team, which consisted of four individuals. The project team proved vital in keeping the project moving forward and on task. The individuals and their affiliations were as follows:

- Stan Young, Bureau of Materials and Research
- Jaci Vogel, Division of Operations
- Mark Clements, Fiscal Services
- Barb Blue, Advanced Traveler Information Systems Coordinator

The basic tenets of the RFP issued in the winter of 2005/2006 included the following:

- Outfit a minimum of four rest areas in the initial year as a pilot study. At each rest area, the public would be able to log into the Wi-Fi service to access traveler information such as weather, road closure, and construction/detour information free of charge. The public also had the option of accessing the Internet for other services, such as web browsing and email. However, the provider could charge a subscription fee (though this was not required). Advertising would also be an acceptable method for revenue generation.
- If KDOT deemed the one-year pilot program successful, Wi-Fi service could be extended to all KDOT rest areas. The contract could be renewed for an additional four years. Vendors were also asked to provide Internet connectivity at rest areas for state business purposes (KDOT, Kansas Highway Patrol, and the Kansas Department of Commerce, Division of Travel and Tourism), preferably at no charge.
- The vendors were asked to submit proposals with and without advertising revenue and with and without a profit sharing scheme.
- Executive staff agreed with the approach and added that any methodology that obtained the services at no cost while simultaneously limiting risk and liability to KDOT would be preferred.
- Awarding of the contract and initiation of the pilot study was targeted for January 2006. The steering committee would continue to monitor the project through the pilot phase and into full deployment.

The RFP was issued in the fall 2005, with a closing date of October 31, 2005. The evaluation and negotiation process lasted over the winter months, and a final contract was signed on February 15, 2006. The contract was awarded to Coach Connect, the same company that was under contract with the Texas Department of Transportation.

Coach Connect proposed an approach in which the basic Wi-Fi system would be deployed and operated at no cost to the state. A traveler information portal would be offered free of charge, and subscriptions could be purchased for a nominal fee for full access, within the constraints set forth within the RFP for duration of service at any single rest stop. The Wi-Fi could also be used free of charge for state purposes.

Development of a Web Portal for Traveler Information

KDOT began working on the traveler information web portal in April 2006. The first step was to research what other states with a similar project model (primarily Iowa and Texas) had done. From that research, a proposed layout for the portal and its content was developed.

KDOT representatives met with the Kansas Department of Commerce, Travel and Tourism Division, on April 24, 2006, to discuss goals and needs, desired and available content for the portal, and promotional ideas for the project. Since the portal would provide travel information not only for Kansas, but for all adjacent states as well, these states were contacted to inform them about the Kansas Wi-Fi project and to invite them to provide information to be included in the portal. All states agreed to share information.

Steering committee members and/or their representatives assisted with the development and approval of the portal. An individual from the KDOT Scenic Byways Program was also added to the committee to provide input regarding scenic byways. Additional KDOT personnel assisted by providing the maps and photos required for the portal.

The initial meeting to discuss the portal with the expanded steering committee was convened on May 25, 2006. A summary that included the research regarding portal content and layout in other states, the meeting with tourism representatives, and the results of discussions with adjacent states was presented. A basic concept for the portal, including the desired content, was discussed, and the results were submitted to Coach Connect for development of the first version of the portal.

When Coach Connect developed the initial draft of the portal, the steering committee met to review and critique it, and many changes were proposed. This was the beginning of a series of changes in not only content, but also map presentation, menu bar options, links and their functions, and general layout. After several versions and review meetings, the portal was approved by KDOT in March 2007.

Pilot Test Deployment

The initial-year pilot test was targeted for the following four rest areas:

1. One of the following off-interstate rest areas:
   - Greenwood County rest stop on US-400 (selected by the vendor)
   - Yates Center rest stop on US-75
   - Sabetha rest stop on US-36

2. One of the following high-traffic rest areas:
   - McPherson rest stop off of I-135
   - Williamsburg rest stop on I-35 (selected by the vendor)

3. Kansas Visitor Information Center: Goodland rest area on I-70 near Colorado
4. Paxico rest area off I-70 west of Topeka – nearest rest area to KDOT headquarters

Pilot phase deployment included installing equipment at each of the rest areas noted above. Coach Connect subcontracted the equipment installation. At the time of the RFP development and the discussion of critical issues, installation of equipment was not considered a significant risk and did not provoke significant discussion. However, due to the distributed nature of the rest area administration within the KDOT, a wide variety of structures at rest areas, a separate installation contractor, and the unfamiliarity between all parties, deploying the equipment proved more difficult and took much longer than expected.

The initial concerns of KDOT were installation procedures to ensure proper routing of cables, installation and sealing of roof and sidewall intrusions, and general quality issues. Before the equipment was installed, KDOT requested installation plans. After receiving written guidelines for sample installations and photographs of previous installations, KDOT field personnel had concerns about the installation procedure and its relationship to KDOT structures. After these concerns were resolved, the contractor, in order to minimize cost, planned to install equipment at all four locations on successive days in a single week. The installation crew was traveling in from out of state, and such a scenario would minimize installation costs. A KDOT representative was needed at all locations at the proper time to approve the installation plan and provide access to utility closets. Ultimately, equipment was installed in all locations, but it was delayed by about three months. Although it took longer than expected, the installations were for the most part successful. The only unresolved issue was at the Kansas Visitor Center in Goodland. Equipment was installed in only one facility rather than at both the eastbound and westbound facilities.

System Startup

Once the portal was complete, the equipment was installed, inspected, and tested by the final week of April 2007. Wi-Fi service at the pilot locations was made available to the public on May 1, 2007. The service was deployed smoothly, and no problems have occurred up to the present time. Both KDOT and Coach Connect agreed the service would be started with a "soft" deployment approach, in that KDOT did not publicize the service at the time it was released, in order to allow time to ensure that the service was working satisfactorily before promotion efforts began. Ultimately, users will have to pay a subscription for unlimited Internet access; however, the service has initially been made available without charge. Subscriptions are planned to begin sometime in June 2007. While notification signs for the pilot locations were installed by May 25, 2007, additional promotion will begin in June 2007, around the time subscriptions begin.

LESSONS LEARNED

This public-private partnership is not the first such venture for KDOT, but such relationships are not the norm. In retrospect, KDOT's approach proved workable, but several lessons were learned along the way. The positive aspects of KDOT's approach that should be considered for similar initiatives include the following:

- Build internal consensus of the objective, scope, and proposed benefits.
- Listen carefully to all stakeholders concerns, particularly regarding risks and potential pitfalls.
- Develop strategies to mitigate project risks.
- Keep representation broad on the procurement team.
- Provide open and frequent communication to the steering committee, stakeholders, and agency executive staff.
- Evaluate the business plan of the vendor, as well as the vendor's proposal.
- A project champion and cohesive project team are required.

Some of the oversights and lessons learned include the following:

- Installation can be more complicated than expected. Investigate and plan well in advance. Set expectations both for agency personnel and contractor personnel. Consider including guidelines in the RFP.
- With many stakeholders, portal development will take time. Allow adequate time for communication with not only the stakeholders, but also the contractor and the subcontractors to execute developmental changes. (A minimum of six months for portal development and final approval is recommended.) Develop a plan, timetable, and methods for continuously updating the portal.
- The challenges in web portal development were as follows:

- Map views and resolutions: The greatest challenge was selecting maps that provided enough detail to be helpful and not so much detail that they were difficult to read, especially for the resolution necessary for the portal space. After much trial and error, the committee determined that the best solution was to give users the ability to select a larger map view.
    - Menu bar: Determining the highest priorities for the menu bar was challenging. The goal was to keep the portal as user friendly as possible, without burying information under several "drill-down" layers.
    - Negotiating differing views and priorities with the various stakeholders is crucial.
    - Accommodate information, such as the Amber Alert system, that is not active all the time. The goal was to draw attention to the information when active. Amber Alert information is provided as crawl information at the bottom of the portal screen.

- Develop a deployment plan so that portal development and hardware deployment can occur simultaneously and be completed about the same time. At the suggestion of the contractor, Kansas waited for the portal to be completed before proceeding with hardware deployment, which was not necessary.
- Without financial obligations, partnerships must rely heavily on trust and communication. The incentive for the private partner must be sufficient to keep the partner's interest throughout the entire project.
- When disruptions in project leadership occur, communicate the change in responsibilities clearly to stakeholders. A project team member left KDOT in November 2006. This disruption caused additional delay during pilot phase installation. Another team member stepped up to lead the project and keep it moving forward and on target until the system became operational in 2007.

Although the Kansas project remains on track, the overall success of the project is still in doubt. The pilot system was activated in May 2007, and the system is currently being monitored to assess whether all of the potential risks have been successfully mitigated.