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Measures for Highway Maintenance Quality Assurance

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ABSTRACT

This paper contains the findings of an investigation into the measures used in highway maintenance quality assurance. The study is an outgrowth of the Maintenance Quality Assurance (MQA) Peer Exchange held in October 2004 in Madison, WI. The peer exchange focused on highway maintenance and involved 74 participants representing 35 U.S. states and Canadian provinces. The conference's online document library, consisting of documents submitted by participating state DOTs, is the primary resource used to complete this study.

Highway agencies practicing MQA have become increasingly interested in what other agencies are doing; what measures are being used, and what works. The purpose of this study is to provide a resource for those agencies. The goals of this paper are to present a set of terms used in MQA, illustrate a process for identifying common measures for quantifying maintenance performance, and highlight some of the measures identified. The process is illustrated with the development of measures for maintenance features related to traffic management. A synthesis of the information gathered using results from previous workshops and surveys leads to conclusions about whether consensus exists about measures among agencies practicing MQA.

Key words: maintenance features—maintenance quality assurance—measures

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PROBLEM STATEMENT

Constrained budgets and reduced funding have caused states to re-evaluate spending and allocations for maintenance. Much attention is being placed on accounting for maintenance expenditures and justifying maintenance budgets. One approach is to relate highway maintenance to highway performance through maintenance quality assurance (MQA).

MQA is a process that uses quantitative or qualitative indicators to assess the performance of maintenance programs. These programs are outcome-based and provide statistically valid, reliable, and repeatable measures of asset conditions (Monroe, Bittner, and Lebwohl 2005). The idea of quality in maintenance was first considered in the 1960s as a part of a maintenance management system concept (Smith, Stivers, and Romine 1999). The notion of quality in highway maintenance has gained momentum in recent years as the national focus shifts from infrastructure design and construction to maintenance and rehabilitation (TEA-21 1998). Performance measures are now being used in transportation maintenance to ensure quality, as is being done in other transportation fields such as transportation planning (Cambridge Systematics 2000).

Most states establish MQA programs for a combination of reasons. The motivating causes include legislative mandates, increased accountability, and improved maintenance program management. Regardless of the desired goal, the output of maintenance quality assurance programs is the detection of insufficient maintenance efforts, poor material performance, and incorrect maintenance procedures (Smith, Stivers, and Romine 1999). In addition, MQA data are being used for condition assessment, maintenance policy analysis, efficiency measurement, and/or maintenance funds allocation (Adams 2005).

Performance measures are at the foundation of an effective MQA program (Hyman 2004). As the national focus shifts towards using measures to manage government, states are becoming increasingly interested in establishing MQA programs. Those states that already have a program are interested in communication with others about how programs are being used for accountability and budget justification. Additionally, all states interested in MQA want to know the maintenance categories and features to include in an MQA program and what measures to use.

RESEARCH OBJECTIVES

There are some guidelines available to highway agencies to assist in the development and application of MQA programs (Smith, Stivers, and Romine 1999), but for the most part each state creates its own program. This paper builds upon the results of previous studies to take the next step in developing common terms and identifying measures for highway maintenance quality assurance. Two critical barriers for establishing and maturing MQA programs are (1) the lack of a commonly understood set of terms for communication about MQA and (2) a lack of consensus on a set of commonly recognized maintenance features and their associated measures of maintenance performance. It is expected that common terms and measures will enable states to better evaluate their own programs and the performance of their highways and to improve communication on issues relevant to MQA.

The goals of this paper are to present a set of terms and to illustrate a process for identifying measures for quantifying maintenance performance. The process is illustrated with the development of measures for maintenance features related to traffic management.

MQA TERMINOLOGY

The terms used in the business of MQA are diverse and inconsistent. Each agency uses its own set of terms that are often poorly defined. To date, no standard set of terms has been proposed to describe the activities involved in an MQA program. In many cases, the same term is used to describe subtle but importantly different aspects. This lack of consensus on terms makes it difficult for maintenance officials to communicate with each other and with those outside the maintenance profession. A set of terms has been adopted for the purpose of this paper. The terms used build upon a glossary provided in the NCHRP report on Highway MQA (Smith, Stivers, and Romine 1999). MQA terms commonly used throughout this paper are defined below. Definitions for these MQA terms were compiled following an extensive review of documents and literature relevant to the MQA process.

1. *Maintenance category*: A maintenance category is a logical grouping of maintenance features based on their location or function along a highway. Examples include pavement, shoulders, and traffic management. Categories are made up of features whose condition is measured with respect to a particular characteristic.
2. *Maintenance feature*: A maintenance feature is a physical asset or activity whose condition is measured in the field. There is one or more maintenance feature in each category. Collectively, the maintenance features describe the maintenance quality of a maintenance category.
3. *Maintenance characteristic*: A maintenance characteristic is a specific quality/defect in a maintenance feature that is condition-evaluated (e.g., signs can be evaluated with respect to retroreflectivity, appearance, sign height, and other characteristics/deficiencies).
4. *Standards*: A standard is a criterion for determining whether a characteristic or feature requires maintenance attention. A standard can be thought of as a tolerance level that helps identify whether a feature is functioning as intended. Standards may also help identify the particular aspect of the feature that should be measured.
5. *Measures*: Measures describe ways to quantify the deficiency of a maintenance feature (e.g., linear feet, percentage area, or amount of deficiency).
6. *Thresholds*: Thresholds are predetermined system-wide maintenance levels for features and categories. Thresholds can be thought of as a grading scale or LOS indicator for MQA. Thresholds indicate how much or what percentage of the system is with or without deficiency. Thresholds also relate measures to customer satisfaction.
7. *Targets*: Targets relate thresholds to the maintenance budget. The target represents the expected threshold level that is attainable.

The relationship between a category, feature, characteristic, standard and measure can be described in the following way. A maintenance category is made up of one or more maintenance features. Each feature can be measured with respect to one or more characteristics. One or more standards can then be used to describe whether a characteristic requires maintenance attention, and for each standard there is a specific measure.

RESEARCH METHODOLOGY

The process for identifying measures for MQA involved several steps. This paper illustrates the steps for identifying measures for the traffic management maintenance category. The following narrative outlines these steps. The steps are then summarized in a flow chart (see Figure 1).

The first step required that programs measuring the traffic management category be identified. The MQA online documents and materials library was the main resource used to identify measures. A variety of documents relevant to MQA were submitted to this database; of primary concern were field guides, rating manuals, reports, and field checklists. These documents may be accessed via the Midwest Regional University Transportation Center (MRUTC) webpage (MRUTC 2004). For the purpose of this paper, only agencies submitting relevant documents to the online library were labeled as “practicing MQA.” A total of 33 transportation agencies, including 2 Canadian provinces, submitted documents to the library. Following a thorough review of these documents, 26 agencies were identified as “practicing MQA.”

The second step was to complete an inventory of the traffic management maintenance features. Maintenance inventory, as the term implies, is simply a checklist relating the maintenance features commonly included in MQA programs to the agencies that include each feature (see Table 1). The purpose of the inventory was twofold. First, the inventory identified the maintenance categories most frequently used to group maintenance features. Second, the inventory identified the maintenance features most frequently measured by agencies with MQA programs.

In the third step, maintenance features were harmonized. Action was taken to harmonize or bundle the inventory (pair similar categories and features), in an attempt to reduce the list. Definitions included by agencies in the MQA documents reviewed were used to complete this step. Eight maintenance categories and 122 maintenance features were identified after bundling.

In the final steps, measures and standards were identified for all possible maintenance features. The list of measures and standards provided in this paper includes all traffic management features for which measures could be identified.

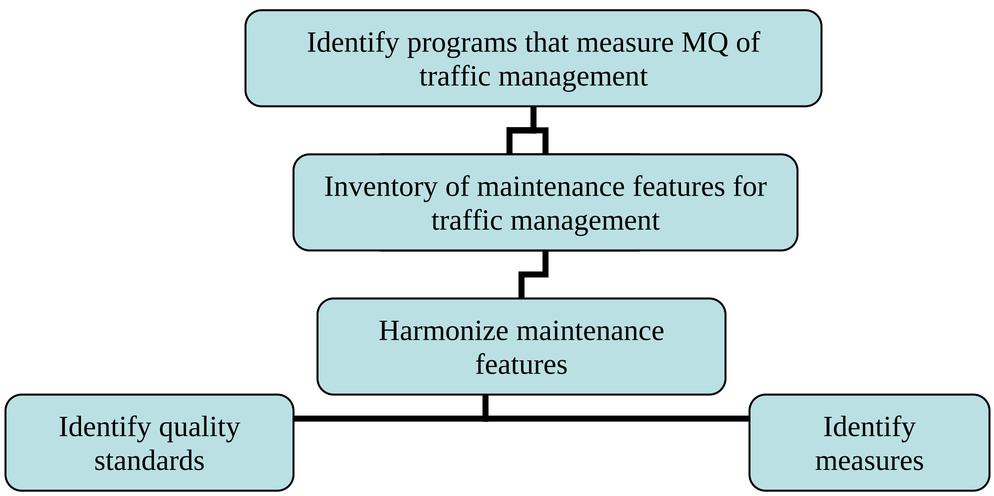

Figure 1. Flow chart of process for identifying common measures

The process highlighted above resulted in a maintenance inventory that provided information about the categories and features included in MQA programs nationally. This paper specifically highlights the results pertaining to the traffic management category. The MQA documents reviewed were used as a guide to develop categories appropriate for this research effort and assign elements to these categories.

MAINTENANCE OF TRAFFIC MANAGEMENT INVENTORY

The traffic management inventory communicates information about the traffic management category. The traffic management category contains features specific to maintaining safety along the travel way. Twenty-two out of twenty-six MQA programs reviewed included a traffic management category.

Table 1 shows the inventory of the maintenance features for each state's program. The table indicates the traffic management category in the MQA programs and the specific maintenance features that are included in the traffic management category for each state. Figure 2 shows the percentage of MQA programs measuring each feature.

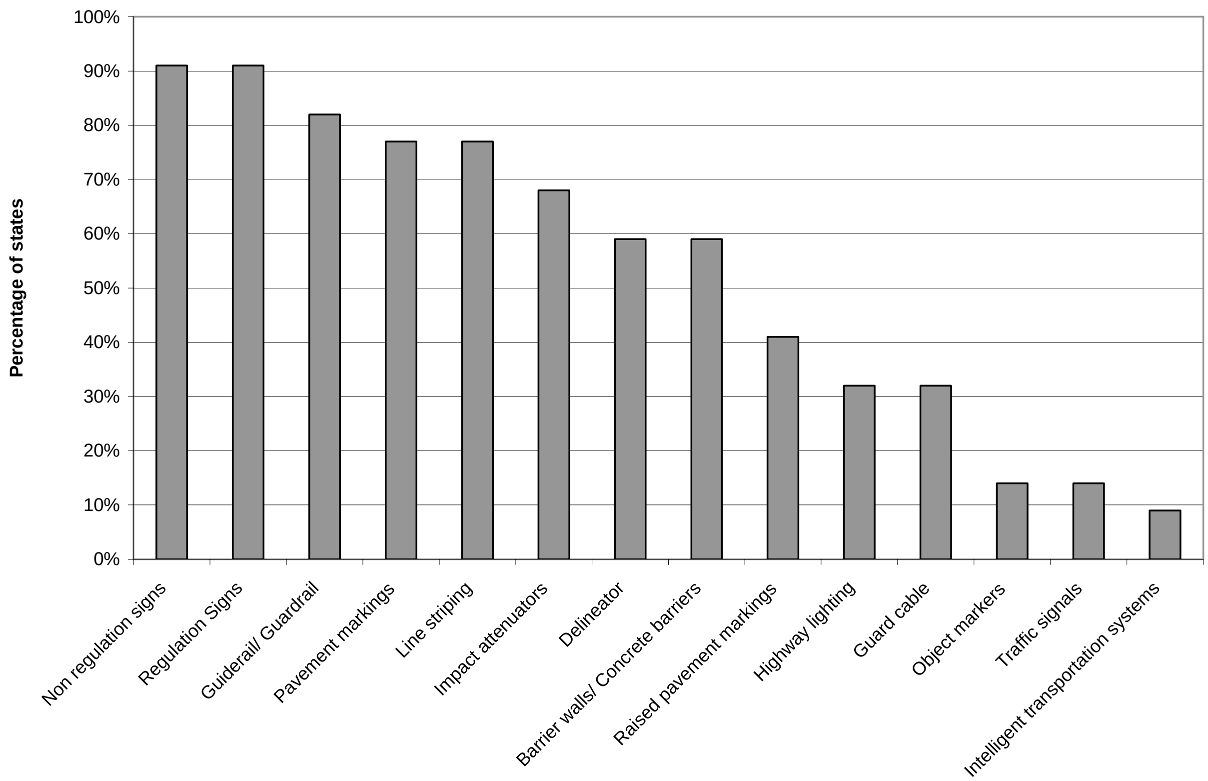

Figure 2. Traffic management features and the percentage of programs measuring each

The traffic management category included 14 features. Regulatory and non-regulatory signs were the most frequently measured features. Seventeen of twenty-two MQA programs measured these features.

Table 1. Maintenance features that evaluate traffic management at transportation agencies

Maintenance Features	CA	CO	DC	IA	IN	KS	KY	MD	MN	MO	MS	MT	NC	NY	OH	SC	TN	TX	UT	VA	WA	WI
Non-regulatory signs	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x	x	x		x
Regulatory signs	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x	x	x		x
Guiderail/guardrail	x	x	x	x	x	x	x	x		x	x		x	x	x	x	x	x	x	x	x	
Pavement markings	x	x	x		x	x		x		x	x	x	x	x	x	x	x		x	x	x	x
Line striping	x	x	x	x		x	x	x	x		x	x	x	x		x		x	x	x		x
Impact attenuators	x	x	x		x	x	x	x		x	x		x		x	x	x	x				x
Delineator		x	x	x	x	x		x		x	x	x	x					x	x			x
Barrier wall/concrete barriers	x		x	x	x	x	x	x		x	x	x					x			x		x
Raised pavement markings	x		x		x						x					x		x		x	x	x
Highway lighting		x	x	x						x	x	x								x		
Guard cable				x						x				x	x	x				x		x
Traffic signals		x										x					x					
Object markers				x		x				x												
Intelligent transportation systems		x										x										

STANDARDS AND MEASURES FOR TRAFFIC MANAGEMENT FEATURES

Table 2 compiles the standards and measures used in MQA programs for assessing the maintenance condition of traffic management features. The table is a representation of the many measures used in MQA programs nationally to quantify the deficiency of any given traffic management feature. In addition, the standards or tolerance levels used to identify whether the traffic management element was functioning as intended are included.

Table 2. Standards and measures for maintenance quality of traffic management features

Feature	Characteristics	Standard	Measures
Non-regulatory signs	Retroreflectivity Appearance Sign height Deviation of post from perpendicular alignment Worn or missing characters Incorrect lateral clearance	Requires attention if insufficient reflectivity, worn or missing characters, incorrect sign height, incorrect lateral clearance, or deviation of post alignment from vertical evident	Number of deficient signs/ total number of signs Number of missing, damaged, illegible signs Number of signs with poor reflectivity Number of non-perpendicular signs
Regulatory signs	Same as above	Same as above	Same as above
Guiderail/ Guardrail	Structural integrity Functionality Deviation from horizontal design height	Guardrail is deficient if damaged to the point at which structural integrity is compromised, functionality is impaired, or deviation of guardrail from design height is observed	The longitudinal length of any guardrail that is not functioning as designed or has been damaged Percent damaged as a function of original design capacity
Pavement markings	Paint worn or missing Lateral movement from original location	Requires attention if extent to which marking is worn is greater than desired, percentage of marking still in tact is less than desired, or distance of line from original location is greater than desired	Number of deficient markings/ total number of markings Amount (length) of line damage Distance of pavement markings from original location

Feature	Characteristics	Standard	Measures
Line striping	Paint worn or missing Visibility of line from a given distance Lateral movement from original location	Requires attention when percentage of paint missing from line exceeds allowable amount, line not visible from a given distance, distance of line from original location greater than desired	Total linear feet of worn, missing, or damaged lines Distance of line striping from original location
Impact attenuator	Functionality of original design	Requires attention if functioning at less than allowed percentage of design capacity.	Number of attenuators needing repairs Length of deficient attenuators Percent of attenuators free of defects per segment
Delineator	Missing or worn reflectivity Variation in vertical height alignment Variation in perpendicularity	Requires attention if a given percentage of reflectivity is missing or worn or if vertical height alignment or perpendicularity varies by more than allowed length	Number of deficient delineators/total number of delineators that should be present Number of delineators missing or defective
Barrier wall/ concrete barrier	Deviation of horizontal alignment Functionality of original installation	Requires attention once deviation in horizontal alignment is observed or certain percentage of installation not functioning as intended	Number of crash barriers damaged, malfunctioning, or missing/total number of crash barriers Linear feet deficient protective barrier Percent free of defects per segment
Raised pavement markings	Functionality of original installation Visibility of markers at night Evidence of excessive gaps between markers	Requires attention once a given percent of original installation not functioning as intended, unnecessary gaps in markers are observed, or markers are not visible at night	Number of RPMs that are deficient/total number of RPMs that should be present in the segment Total length of deficient pavement markers/total length of pavement markers

Feature	Characteristics	Standard	Measures
Highway lighting	Functionality of original installation Integrity of poles Integrity of hardware and lamps	Requires attention once given percentage of installation is not functioning as intended, poles are broken, or hardware and lamps are missing	Number of highway lights deficient/total number of highway lights Percentage of lights along segment that are functional
Guard cable	Structural integrity Deviation of horizontal alignment	Requires attention if damaged to the point that the structural integrity is compromised or there is deviation of horizontal alignment from design height	Linear feet deficient/total linear feet of protective barrier All cables not functioning as intended
Object markers	Functionality of markers	Missing or non-functional markers require attention	Number of consecutive missing or non-functional markers
Traffic signals	Functionality of lamps Structural damage	Signals require attention if lamps are out or not functioning as intended	Number of signals with lamp outages, improper signal operation, or damage Percent of traffic lights with bulbs not working, structural damage, or improperly functioning loops
Intelligent transportation systems	Functionality of systems	Requires attention if the percentage of non-functioning systems is less than allowed	Percent of ITS systems not working

COMPARISON TO MQA WORKSHOP SURVEY

Progress to date as identified by this study can be analyzed in terms of the results of the MQA Peer Exchange pre-workshop survey (Adams 2005). Features included in the pre-workshop survey that are relevant to this study included regulatory signs, centerline pavement markings and edge line pavement markings (combined and labeled “line striping”), lighting, and guide rails. Participants of the survey were asked whether they “measure the condition of the element.” Comparison results are shown below (see Figure 3).

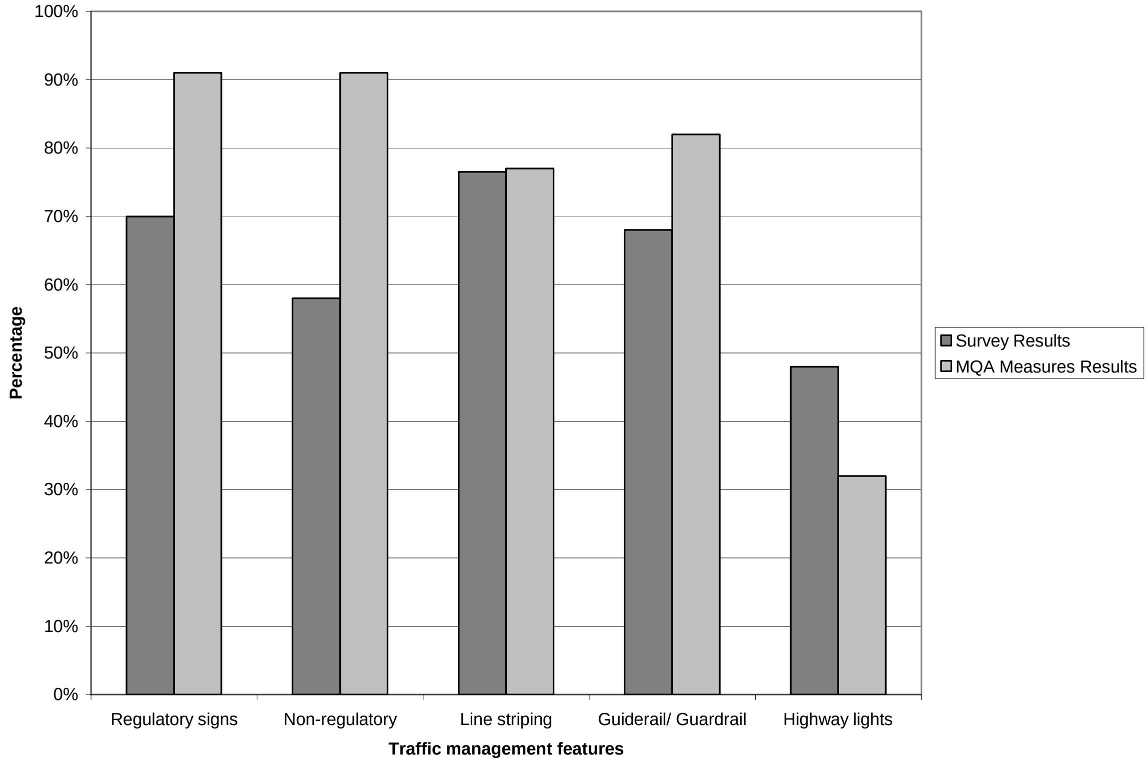

Figure 3. MQA Peer Exchange survey results compared to results of MQA measures study

A rough comparison of these results reveals that there of has been an increase in the percentage of states measuring signs, a decrease in the percentage of states measuring lighting, and little change in the percentage of states measuring pavement markings. It should, however, be noted that in each study there were differences in how features were defined and measured.

COMPARISON TO SCOTTSDALE WORKSHOP

The National Workshop on Commonly Recognized Measures for Maintenance was held in Scottsdale, Arizona from June 5-7, 2000 (Booz-Allen & Hamilton 2000). Representatives from 25 states and 18 organizations participated in the workshop, which was sponsored by the AASHTO Subcommittee on Maintenance with funding and support from the Federal Highway Administration (FHWA).

One of the primary objectives for the Scottsdale workshop was to reach consensus regarding an initial set of common measures that would reflect the outcomes from the delivery of maintenance services and products. The Scottsdale workshop represents the first major attempt to identify measures for maintenance quality assurance. As a result, the findings of this study with respect to terms and measures have been compared to those of the Scottsdale workshop.

TERMS

The proceedings of the Scottsdale workshop do not include a glossary of key terms and definitions used in MQA. In fact, the meanings of some terms are ambiguous. The Scottsdale report uses terms such as "maintenance area," "maintenance elements," and "measures." However, these terms are often used interchangeably, making it difficult to interpret the results and findings from Scottsdale.

Although terminology for MQA lacks consistency and consensus, it is apparent that progress has been made since the Scottsdale effort. There are now popular MQA terms that are used more frequently; however, there is still a need for consistency in the use of these terms and consensus on the definitions for these popular terms. There is a need to adopt a formal terminology for describing MQA. It is expected that the clear definitions provided in this research effort, coupled with the glossary provided earlier (Smith, Stivers, and Romine 1999), will provide a stepping stone towards the development of a formal terminology for MQA.

MEASURES FOR MQA

There is clear evidence that the use of measures in MQA has evolved significantly since the 2000 Scottsdale workshop. What was identified during the Scottsdale workshop as measures for maintenance are referred to as "features and characteristics" today. The findings most relevant to this study were those pertaining to signs, pavement markings, and safety features and appurtenances (see Table 3).

Table 3. Comparison of 2000 Scottsdale workshop results with MQA practice today

Maintenance category or feature	2000 Scottsdale workshop-recognized measure	Standards in 2005	Measures in 2005
Signs	Retro-reflectivity Physical appearance Customer satisfaction	Requires attention if insufficient reflectivity, worn or missing characters in message, incorrect sign height, incorrect lateral clearance, or deviation of post alignment from vertical are evident	Number of signs deficient/total number of signs Number of missing, damaged, illegible signs Number of signs with poor reflectivity Number of non-perpendicular signs
Pavement markings	Retro-reflectivity Physical appearance Customer satisfaction	Requires attention if extent to which marking is worn is greater than desired, percentage of marking still in tact is less than desired, or distance of line from original location is greater than desired	Number of deficient markings/total number of markings Amount (length) of line damage Distance of pavement markings from original location
Safety features and appurtenances	Attenuators Guardrail Guardrail end treatment	Considered features today	Considered features today

Table 3 compares measures identified in the 2000 Scottsdale workshop with those used today. The workshop-recognized measures highlighted in the table are both features assessed in the field (attenuators and guardrail) and characteristics used to assess condition (retroflectivity and physical appearance). Thus, progress has been made in the area of discerning maintenance features from their characteristics. Another sign of progress is the fact that measures identified at Scottsdale, though not adopted as measures, are now included in most MQA programs as features to be measured.

Since the Scottsdale workshop, states have taken steps to group features into broadly defined categories. Results from this study show that signs, pavement markings, and safety features and appurtenances are now evaluated collectively and are grouped together into a traffic management category. It is also apparent as MQA has evolved that customer satisfaction is no longer used as a direct measure. Instead, the measures used currently act as a surrogate for evaluating customer satisfaction.

CONCLUSIONS

Terminology for MQA business has evolved significantly, but no standard exists. Without commonly understood terms. agencies will not be able to efficiently communicate on the development of their programs. It is expected that the terminology developed and steps outlined in this paper will assist in the development of future MQA programs and the identification of measures for those features not commonly used by states to assess the performance of highways.

The inventory helped the standardization effort by identifying features that are measured. However, there was less success at identifying measures, as there were features identified as being measured for which little information on measures was available. Though a comprehensive list of measures was identified, there is still much opportunity for the expansion of this list.

The findings of this paper indicate that there has been some consensus reached on the identification of maintenance categories and maintenance features to be included in MQA programs. Consistency among MQA programs regarding features included in the traffic management category highlights this point. A preliminary comparison of results to previous findings suggests that a larger percentage of states have taken steps to include specific features in their MQA programs. In addition, states have adopted as features the measures highlighted at the Scottsdale workshop.

Overall, MQA programs have evolved since the 2000 Scottsdale workshop. Most states are beyond gathering information and now concentrate on using the information in decision making. Programs now include statistical analysis, and states are experimenting with alternate reporting formats to effectively communicate to legislatures and the public. As a consequence, a common understanding of terms like “thresholds” and “targets” will be essential to the future development of this field.

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Impacts of Weather on Urban Freeway Traffic Flow Characteristics and Facility Capacity

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ABSTRACT

Adverse weather degrades the capacities and operating speeds on roadways, resulting in congestion and productivity loss. Without a solid understanding of the mobility impacts of weather on traffic patterns, freeway operators do not have the estimates of speed and capacity reductions to predict and simulate the impacts of traffic management strategies. Nearly all traffic engineering guidance and methods used to estimate highway capacity assume clear weather. However, for many northern states, inclement weather conditions occur during a significant portion of the year.

This paper describes how the authors quantified the impact of rain, snow, and various pavement surface conditions on freeway traffic flow for the metro freeway region around the Twin Cities. The research database includes four years of detector occupancy information from roughly 4,000 detectors, weather data over the same period from 3 automated surface observing systems (ASOS) at nearby airports, and two years of pavement surface condition data from 5 road weather information systems (RWIS) sensors in close proximity to the freeway system. Our research classifies the rain and snow events by their intensity levels and identifies how changes in precipitation intensity impacts the speed, headways, and capacity of roadways.

Results indicate that severe rain and snow cause the most significant reductions in capacities and operating speeds. Heavy rains (more than 0.25 inch/hour) and heavy snow (more than 0.5 inch/hour) showed capacity reductions of 10%–17% and 19%–27% and speed reductions of 4%–7% and 11%–15%, respectively. Speed reductions due to heavy rain and snow were found to be significantly lower than those specified by the Highway Capacity Manual (2000).

Key words: capacity—freeways—operating speeds—precipitation

INTRODUCTION

Adverse weather impacts on freeway traffic operations have become a growing concern for federal and state transportation agencies. Although it is obvious that inclement weather conditions reduce freeway capacities and slow traffic, little research has been conducted to quantify the impacts of rain and snow in the United States. In addition, the results obtained from studies outside the United States or rural freeway segments within the United States may not be applicable to urban freeway segments due to different roadway and driver characteristics. For example, the decrease in capacity for rural freeway segments of I-35 in Iowa during heavy snowfall may not be same as the urban freeway segments of I-35 in the Twin Cities.

The effect of inclement weather is important in northern metropolitan areas (e.g., Minneapolis/St. Paul, Denver, Salt Lake City, Detroit, and Buffalo), where appreciable snowfalls (more than 0.1 inch/hour) occur frequently (averaging 38, 35, 34, 39, and 62 days per year, respectively) and heavy snowfalls (more than 2 inches/hour) occur about 8, 7, 10, 7, and 12 times per year, respectively (United States Snow Climatology 2004). A precise estimate of capacity and speed reductions due to adverse weather can be useful in managing freeway systems using control, advisory, and road treatment strategies. Operational efficiency can therefore be maximized.

LITERATURE REVIEW

Chapter 22 in the Highway Capacity Manual (2000) provides information regarding speed and capacity reductions due to rain or snow of light and heavy intensities. The manual recommends between 0% and 15% reductions in capacities and 2%–14% and 5%–17% reductions in speeds due to light and heavy rains, respectively. Similarly, it recommends 5%–10% and 25%–30% percent reductions in capacities and 3%–10% and 20%–35% percent reductions in speeds because of light and heavy snow conditions. However, the manual does not consider these effects by precipitation intensities, which is important for freeway operators to optimize capacities and operating speeds due to anticipated precipitation (rain or snow) using intelligent transportation system (ITS) devices (e.g., dynamic message signs, ramp metering).

A study on I-35W (Ries 1981) estimated and compared capacities for rain and snow and concluded that the slightest amount of precipitation (also called a trace amount) either in the form of rain or snow reduces capacity by 8%. The study also found that snow caused 2.8% and rain caused 0.6% additional reduction in capacity for every 0.01 inch/hour increase in precipitation (measured in water equivalents) exceeding trace precipitation. Hall and Barrow (1988) examined the impacts of adverse weather conditions on the flow-occupancy relationship for Queen Elizabeth Way near Hamilton, Ontario. They found that the congested portion of the flow-occupancy curve was random due to lower headways, incidents, weaving sections, etc. Therefore, the authors used the uncongested portion of the flow-occupancy curve and concluded that adverse weather reduces the slope of the flow-occupancy linear relation, thereby resulting in reduced capacity. However, this research (Hall and Barrow 1988) did not classify rain and snow by intensity, and, further, obtained weather data from remote weather stations (greater than three miles). A study (Okamoto et al. 2004) on 19 sites of the Tokyo-Nagoya expressway in Japan categorized rainfalls intensity groups (0.0, 0.01–0.06, 0.07–0.12, 0.13–0.24, 0.25–0.48, and 0.49-0.96 cm/hour) instead of categorizing rains by light and heavy rainfall. This study concluded that freeway capacity was reduced by 0%, 5%, 11%, 14%, 25%, and 33%, respectively, for increasing precipitation intensities. Also, it emphasized that highway capacity can be better estimated using both rain intensity and design variables (curvature and grades).

Prior research (Brilon and Ponzlet 1996) concluded that wet roadway conditions cause a reduction of 9.5 km/h (6 mph) on four-lane highways, and 12 km/h (7.5 mph) on six-lane highways. As a result, Brilon and Ponzlet concluded that freeway capacities were reduced by 350 vehicles per hour (vph) and 500 vph, respectively. However, the study was conducted in Germany, where there are no maximum speed limits on freeways and driver behavior and expectancies may differ from U.S. counterparts. Previous research (Smith et al. 2004) also emphasized the importance of rainfall intensity values in capacity and average operating speeds. They classified rain into none, light, and heavy (less than 0.01, 0.01–0.25, and greater than 0.25 inch/hour, respectively) and used Scheffé's method to compare the statistical significance of differences in capacities and speeds for various intensity categories. A text by Neter et al. (2000) explains that Scheffé's method uses an equal variance assumption when comparing the means of data groups, which may not be appropriate to adverse weather conditions.

Prior research by Ibrahim and Hall (1994) used dummy variable multiple regression analysis for rain and snow. This study concluded that light rain and snow resulted in similar reductions in speeds (3%–5%), but heavy rain caused 14%–15% and heavy snow caused 30%–40% reductions in speeds. Although Ibrahim and Hall defined rain and snow in light and heavy categories, they did not specify intensity ranges within these categories. Duration of weather data was also quite limited (they used only six clear, two rainy, and two snowy days). Liang et al. (1998) explored a 75-km (45-mile) rural section of I-84 and found that mean speed was reduced by 8 km/h (5 mph) and 19.2 km/hr (12 mph) for fog and snow events, respectively. Similarly, a study by Kyte et al. (2000) found that light rain or snow resulted in 50% higher reductions in speed, but heavy snow caused 20% lower speed reductions than the values stated in the Highway Capacity Manual (HCM 2000). Interestingly, the Kyte study used the same freeway section of I-84 (with a larger quantity of weather and traffic data). Both studies were limited to rural freeway sections, and did not classify precipitation by intensity.

PROBLEM STATEMENT

Although previous research efforts provide substantial evidence that speeds and capacities can be quantified for snow and rain, the following issues regarding adverse weather impacts on freeway capacities and speeds remain unaddressed and require further study:

- Much of the research pertaining to weather impact is obtained from studies outside the United States. Also, few studies were conducted on urban freeway segments. Thus, research should be conducted to expand the limited guidance about the impacts of weather on traffic flow for urban freeways, while they operate at or near capacity.
- It is necessary to relate measures of weather intensity to traffic flow, as there has been limited research on this issue.
- Prior studies used short-term data, primarily on heavy rain or snowfall. Long-term data sets are needed to quantify weather's impacts on traffic flow and highway capacity.

RESEARCH OBJECTIVES

The main objective of this research was to quantify the relationship between weather and traffic flow variables, thereby providing an analytical basis for the future development of objective guidelines for practitioners. A second objective was to evaluate the adequacy of automated surface observing systems (ASOS) and road weather information systems (RWIS) data for such research. The authors set out to statistically assess reductions in operating speeds and capacities due to various intensities of rain and snow. Finally, this research compared findings with suggested values obtained from the Highway Capacity Manual (2000).

METHODOLOGY

The study area, a portion of the freeway road network of the Twin Cities that is managed by the Traffic Management Center, contains a number of roadside and in-pavement ITS field devices and includes several interstates and trunk highways with segments built to freeway design standards. Unlike prior research, this study used a larger dataset of four years (January 2000 to April 2004) of traffic and weather information, and categorized the impacts of precipitation (rain and snow) by intensities.

Data Collection

The study area has around 4,000 detector loops installed in freeway lanes that collect traffic data (volume and occupancy) for every 30-second time interval. These data were archived from the Traffic Management Center by the University of Minnesota, Duluth. Weather records were obtained from five RWIS sites operated by the Minnesota Department of Transportation (Mn/DOT) and three ASOS sites managed by the National Climatic Data Center.

The locations of detectors in the vicinity of RWIS environmental sensors were identified using maps obtained from the freeway operations division of Mn/DOT. To obtain the detectors near ASOS sites, a buffer region with a radius of 2.5 miles around ASOS sites was created using ArcView GIS 3.2. The detectors along the freeways and state trunk highways in the buffer region were selected for further analysis. The researchers also ensured that the selected detectors were not on highway segments with transitional geometries (e.g., lane drops, weaving sections, significant grades) to avoid biases in results due to road geometries. Table 1 shows the list of detectors and the detectors' field lengths used to estimate flow, speed, and density.

Extraction of Traffic Data

As Mn/DOT does not maintain speed detectors on the freeway system, but instead estimates speeds using flow and occupancy data, the following equations were used to calculate speed, flow, and density for 10-minute time intervals using volume and occupancy data:

$$\text{Flow} = \text{Vehicles/Hour} = (\text{Vehicles/10 minutes}) \times 6 \tag{1}$$
$$\text{Density} = \text{Vehicles/Mile} = (5{,}280 \times \text{Occupancy}) / (\text{Field length} \times 100) \tag{2}$$
$$\text{Speed} = \text{Flow/Density} \tag{3}$$

Even though the detectors' field lengths may also change over time due to variation in construction, design, and age of detectors, this research used a constant field length of detectors to compute flow, speeds, and density (Mn/DOT did not maintain field length history for each detector). While this limitation may introduce a few errors, speed and flow measurements at the same detector were found to be consistent over time. Therefore, as long as comparisons were made for the same detectors with or without the effects of weather, relative capacities and speeds should be consistent for the analysis.

Table 1. List of selected detectors and their field lengths

RWIS site	Detector location	Detector ID	Field length (ft)
330.84	I-35 and Minnetonka Blvd	1874	23.2
		1875	27
330.85	I-35 and Minnesota River	257	24.5
		267	23.5
330.86	I-494 and I-94	2953	33.1
		2864	27.4
330.88	I-35 E and Cayuga St. Bridge	2462	24.2
		2391	23.8
330.89	I-494 and TH-110	2879	18.9
		2940	24
ASOS site	**Detector location**	**Detector ID**	**Field length (ft)**
Minneapolis St. Paul International Airport	Nicollet Ave (I-494)	890	21.18
		891	20.02
		893	18.75
	TH 13	3273	20
		3298	14.3
	TH 77 and Minnesota River	3281	20.5
		3279	23.1
		3292	21
Minneapolis Crystal Airport	TH 169 and 63rd Ave	3005	25
	TH 169 and Bass lake Road	3041	30.8
	I-94 and Brooklyn Blvd.	971	26.66
		972	26.97
		974	23.4
		977	25.2
		979	27.22
		960	29.65
St. Paul Downtown Airport	I-94 and TH-52	3191	31.4
	I-35E and Victoria Street	3240	21
		3431	28.7

Integration of Weather and Traffic Data

This step was accomplished by combining traffic and weather data with constraints of similar date, hour, and 10-minute intervals. Different datasets were prepared and analyzed for each pavement surface condition (dry, wet, and snow/icy) with traffic data at 10-minute intervals.

RWIS data (January 2002 to April 2004) were collected for 10-minute time intervals. Prior research (Hunt and Yousiff 1994; Smith and Ulmer 2003) concluded that time intervals between 5 and 15 minutes are appropriate to compute flow rates for an hour. This research assumed a 10-minute time interval to collect weather and traffic data for the similar time periods, because this time interval is sufficiently long to average out short-lived peaks in flow rates.

An analysis of the database containing pavement conditions and traffic data was conducted to investigate the flow-occupancy relationship for each pavement condition (dry, wet, and icy, as measured by the RWIS pavement sensors). The results obtained for each category were found to be similar to those shown in Figure 1. Figure 1 presents the flow-occupancy relationship during dry conditions. It is clearly evident that what is being observed is the flow-capacity relationship during two different sets of conditions. It is likely that the set of points in the cluster to the left were recorded during dry weather. The cluster to the right exhibits lower maximum flow rates and, therefore, there were probably inclement conditions when these data points were collected. In other words, it appears the RWIS road sensors are providing false readings. Therefore, due to the prevalence of false readings, the use of the RWIS weather data was rejected and the analysis proceeded through the uses of weather information from ASOS sites.

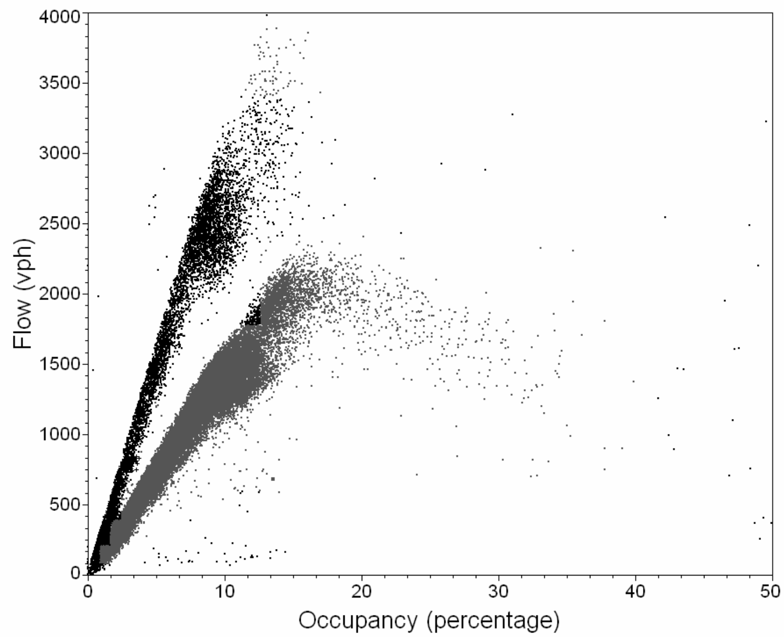


Figure 1. Flow vs. occupancy for dry pavement surface conditions

The weather data reported by ASOS sites are not organized by a specific time interval, but provide information on the amount of precipitation (inches/hour) and the start and end timings of precipitation. The weather data were integrated with the traffic data using a few rules. For example, if the weather data indicate that rain started at 7:23 a.m. and ended at 7:53 a.m., and the hourly precipitation was 0.2 inches. The time intervals for traffic data from 7:20 a.m. to 8:00 a.m. are assigned a precipitation intensity of 0.4 inches/hour (it actually rained 0.2 inches for 30 minutes, which equals and intensity of 0.4 inches/hour).

This research classified rain and snow by their intensities to understand their quantitative impacts on traffic flow variables, as shown in Table 2. This study used similar classifications of rain intensities recommended by Smith et al. (2004), except including the "Trace" category. Prior research did not classify snow events by their intensities and only categorized them by light or heavy snow. Thus, this research classified snow intensities based on the availability of an adequate dataset to assume appreciable speed-flow (parabolic) and flow-occupancy relationships (linear) for increasing snow intensities.

Table 2. Snow and rain intensity classifications

Snow category	Snow intensity (inch/hour)	Rain category	Rainfall intensity (inch/hour)
None	0	None	0
Trace	<= 0.05	Trace	< 0.01
Light	0.06-0.1	Light	0.01-0.25
Moderate	0.11-0.5	Heavy	> 0.25
Heavy	> 0.5		

Flow-occupancy and speed-flow relationships were examined for various weather conditions. An example is shown in Figure 2 for clear weather. The lightly shaded data points in the dashed circle are the top 5% of flow measured for the weather condition. Similar analyses were conducted for varying rain and snow intensities.

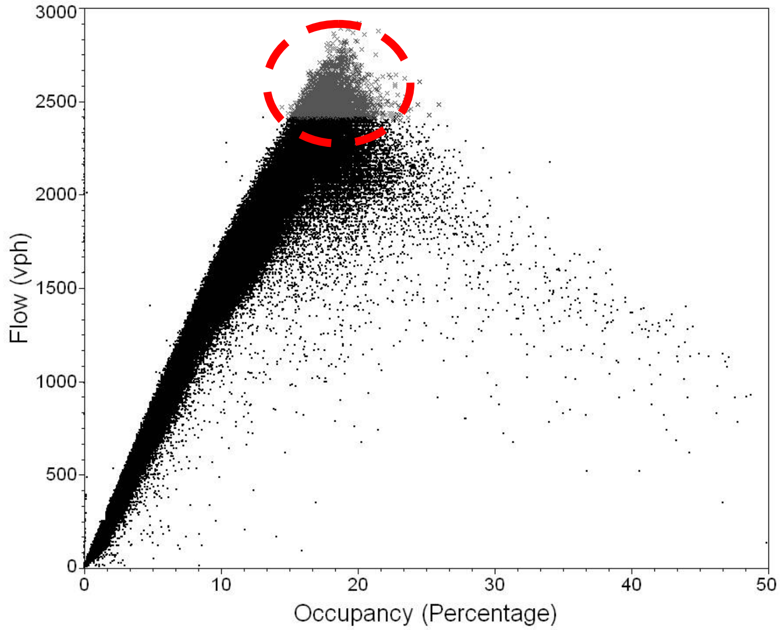
Estimation of Freeway Capacities

This research used the maximum observed throughput approach, as described by Smith et al. (2004). Prior research (Smith et al. 2004) found that the mean of the highest 5% of the observed flow rates by a detector would represent the effective freeway capacity. This method of estimating capacity is chosen because it ensures that a freeway segment will be able to clear the maximum number of vehicles at least 95% of the time. This method also requires prior examination of the flow-occupancy and the speed-flow relationships to ensure that system demand was sufficiently met to reach congestion, as shown in Figures 2(a) and 2(b). Additionally, this approach fits with the primary objective of this research to determine the percent changes in freeway capacities and operating speeds due to various categories of rain and snow. The collected data were further modified by removing records containing data where very low occupancies (less than 5%) or very high occupancies (greater than 50%) existed. Low occupancies are expected only when the freeway is operating well below capacity and very high occupancies are only like to occur after total breakdown of flow caused by a freeway incident. Finally, the freeway capacity for each detector for a selected weather category (e.g., snow intensity between 0.11 and 0.5 inches/hour) was obtained by calculating the average of the top 5% flow rates.

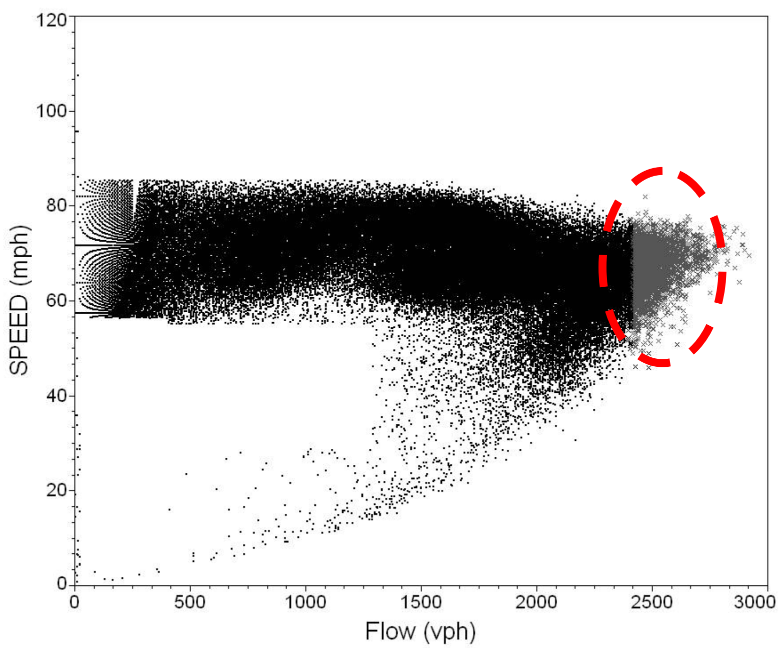
Estimation of Operating Speeds

Previous studies (Hall and Barrow 1988; Ibrahim and Hall 1994) showed that the uncongested portion of speed-flow relationships could be used to compare the adverse weather impacts on average operating speeds. The uncongested portion of the speed-flow relationship was examined for the expected relationship (parabolic), as shown in Figure 2 (b). Also, traffic theory noted that speeds are relatively insensitive to the increasing flow rates for the uncongested portion (speed greater than 45 mph) until congestion is started. Therefore, to compare the changes in speeds due to each weather type and category, a weighted average of speeds (between 45 and 80 mph) by flow rates was calculated to compute the average operating speed. The lower limit of 45 mph was used as a minimum uncongested speed and an upper limit of 80 mph was considered to exclude the errors in the data for 10-minute intervals. This approach is more meaningful for this research because many discrepancies can be avoided using weighted mean values. For example, speed values that are many times higher might be indicated for 10-minute intervals during periods of low-volume conditions (e.g., off peak hours) for a particular weather intensity range (e.g., rain intensity between 0.01 and 0.25 inch/hour), and results might be misleading by just calculating an average of speed values.

Finally, when capacities and operating speeds were calculated for different snow and rain categories, the next step was to compare the differences using the Bonferroni method (significance level of α equal to 0.05). The Bonferroni method was selected because it does not require an assumption of equal variances among compared datasets.



2(a)



2(b)

Figure 2. Flow-occupancy (a) and Speed-flow (b) for clear weather conditions

RESULTS

Once the data were collected and examined for appreciable flow-occupancy and speed-flow relationships, freeway capacities and operating speeds were computed as discussed above. The next step was to calculate an average of freeway capacities and speeds for all detectors near an ASOS site for every weather category. These average values were then compared to evaluate the percent reductions in freeway capacities and speeds due to varying rain and snow intensities.

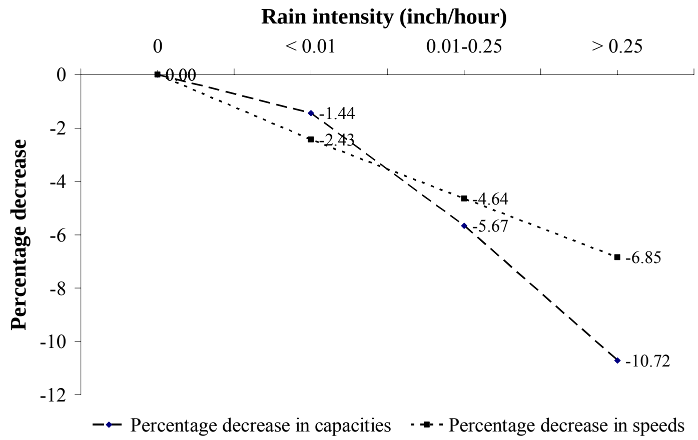
Rain

The rain data were divided into four categories (0, less than 0.01, 0.01–0.25, and greater than 0.25 inches/hour) for the analysis of the impacts of rain on freeway capacities and speeds. The database contained approximately 50,000, 1,400, 1,250, and 200 records for the above-defined rain categories by intensity values for each selected detector. Using these data, capacity and speeds were calculated, and statistical analyses were conducted as described below.

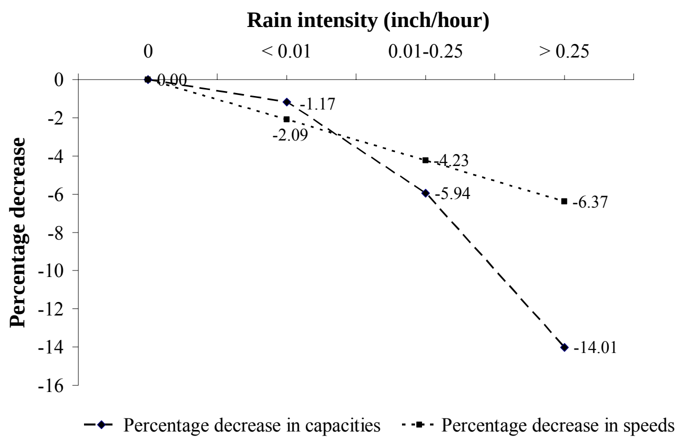
The freeway detector sites near the airports (Minneapolis-St. Paul International [MSP], Minneapolis Crystal [MIC], and St. Paul Downtown [STP]) were selected for this research. These sites showed average capacity reductions of 1%–3%, 5%–10%, and 10%–17%, for trace, light, and heavy rain conditions, respectively, as shown in Figure 3.

Statistical testing indicated that reductions in capacities were statistically significant when compared with no rain conditions, except trace precipitation conditions. Prior research (Ries 1981; Smith et al. 2004) indicated similar results for reductions in freeway capacities due to light rain conditions. However, this study showed lesser reductions in capacity (10%–17%) than the reductions of 25%–30% obtained by the study (Smith et al. 2004).

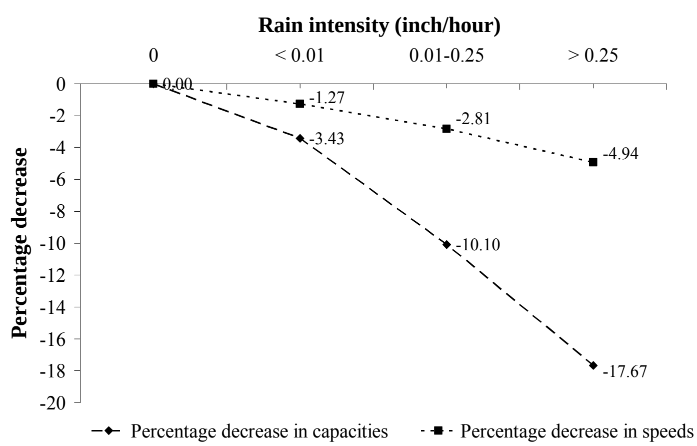
Similarly, speed reductions of 1%–2%, 2%–4%, and 4%–7% were found for freeway sites near the airports (MSP, MIC, and STP) for trace, light, and heavy rain, respectively. Statistical analysis showed that differences in speeds for light and heavy rain (0.01–0.25 and more than 0.25 inches/hour) were not statistically significant. Previous research (Smith et al. 2004) also found similar speed reductions (3%–5%) for both light and heavy rain (0.01–0.25 and more than 0.25 inches/hour) were statistically significant compared with clear weather conditions. In contrast, the differences in operating speeds during light and heavy rain (0.01–0.25 and more than 0.25 inches/hour) were not statistically significant. Thus, they concluded that heavy rain effects on operating speeds are similar to those of light rain.



3(a)



3(b)



3(c)

Figure 3. Effects of varying rain intensities on capacities and speeds for freeway sites near the MSP (a), MIC (b), and STP (c) airports

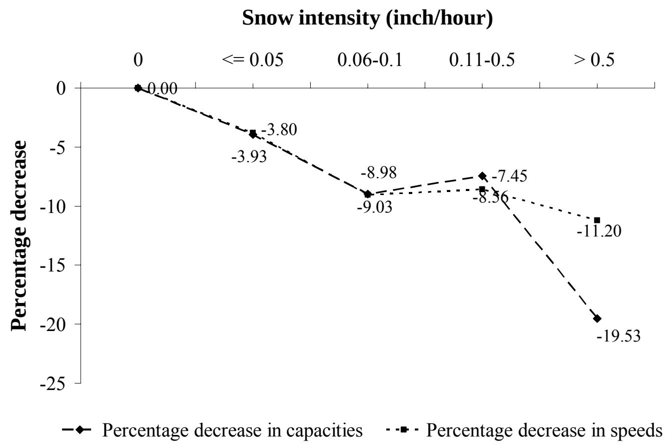
Snow

Datasets on snowfall events were categorized into five categories of none, trace, light, moderate, and heavy (0, less than/equal to 0.05, 0.06–0.1, 0.11–0.5, and greater than 0.5 inches/hour, respectively). The database contained approximately 50,000, 900, 550, 300, and 125 records for these snow categories for each selected detector. Reductions in capacities and speeds were found, as shown in Figure 4.

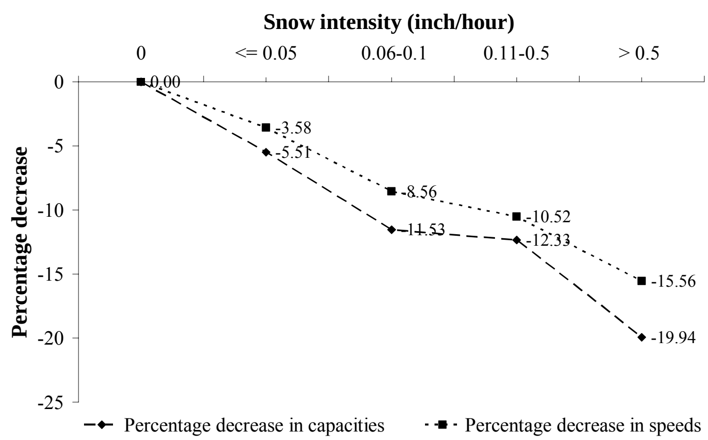
Statistical analysis using the Bonferroni method indicated that capacity and speed reductions were statistically significant for each weather category when compared with no precipitation conditions. However, differences in capacities and speeds for light and moderate snow conditions were not statistically significant for many detectors.

The Highway Capacity Manual (2000) shows that light snow causes 5%–10% reductions in capacity, and this study shows reductions of 3%–5%, 6%–11%, and 7%–13% for trace, light, and moderate snow. Also, capacity reductions of 19%–27% for heavy snow (more than 0.5 inches/hour) compare with the 25%–30% reductions because of heavy snow, as recommended in the Highway Capacity Manual (2000).

Additionally, speed reductions of 3%–5%, 7%–9%, and 8%–10% for trace, light, and moderate snow, respectively, were obtained, which quantifies reduction in speeds better than recommended speed reductions of 8%–10% in the Highway Capacity Manual (2000) due to light snow only. In contrast, speed reductions of 11%–15% percent for heavy snow (more than 0.5 inches/hour) significantly differ from the recommended speed reductions (25%–35%) in the Highway Capacity Manual (2000). This larger variation in speed reductions for heavy snow from this study can be attributed to differences in freeway locations, driver familiarity, moderate occurrences of snowstorms, or better winter maintenance activities in the Twin Cities region. Thus, this difference indicates that reductions in speeds due to heavy snow might be overstated in the Highway Capacity Manual (2000) for urban freeway segments.



4(a)



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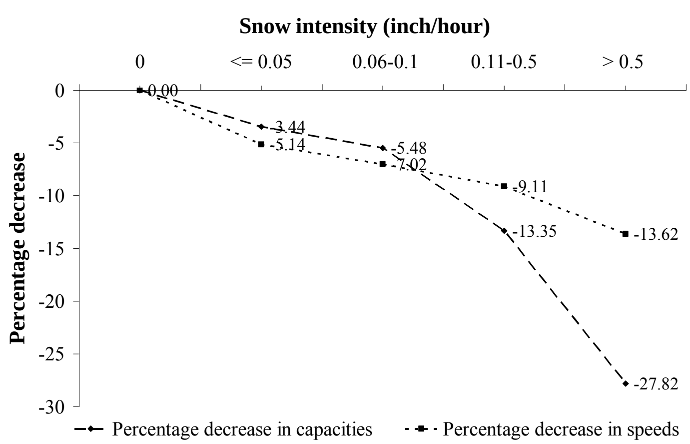

4(c)

Figure 4. Effects of varying snow intensities on capacities and speeds for freeway sites near the MSP (a), MIC (b), and STP (c) airports

Summary of Findings

The results of this research were compared with the current information found in the Highway Capacity Manual (2000), because it is important for traffic operators to understand the extent of reductions in capacity and speeds for urban freeway locations due to varying rain and snow intensities. Results indicate that the manual underestimates or overestimates the impacts, as listed below:

- This research found that light rain (0.01–0.25 inches/hour) has a significantly greater impact (5%–10%) on capacity as opposed to no reductions mentioned in the Highway Capacity Manual (2000).
- The results indicate that reductions in operating speeds due to light rain are comparable with recommended reductions in the Highway Capacity Manual (2000). However, the reductions in operating speeds due to heavy rain may be overstated in the manual.
- Light and moderate snow categories show almost similar capacity and speed reductions, which are similar to reductions by light snow as stated in the Highway Capacity Manual (2000). Thus, these two categories can be merged into one category.
- Heavy snow shows similar reductions (19%–28%) in capacity as stated in the Highway Capacity Manual (2000). However, speed reductions due to heavy snow obtained from this study were significantly lower (19%–25%) than those recommended by the manual.

CONCLUSIONS

Overall, the results of this analysis show how various weather and weather intensities affect traffic flow variables. This research shows that impacts of rain and snow on freeway traffic operations in urban regions are different than recommended in the Highway Capacity Manual (2000). Additionally, this research concluded that the quality of weather data obtained from RWIS sensors was not appropriate for the analysis. Thus, this research provides additional guidance to freeway traffic operators regarding quantitative estimates of capacity and speed decreases due to varying intensities of rain and snow.

ACKNOWLEDGMENTS

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Iowa's Long-Range Transportation Plan (Invited Presentation)

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ABSTRACT

The Iowa Department of Transportation is in the midst of updating Iowa's Long-Range Transportation Plan. Stuart Anderson, Office of Systems Planning Director, will discuss the status of the plan update and review the key directions that will be contained in the plan. Mr. Anderson will also discuss the unique partnership with Iowa's Regional Planning Affiliations (RPAs) and Metropolitan Planning Organizations (MPOs) in the development and public input efforts of the plan.

Note: Contact the presenter above for more information on this topic.

Key words: partnerships—public input—transportation plan

Overview and Status of the Long-Term Plan for Concrete Pavement Research and Technology: The CP Road Map

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EXTENDED ABSTRACT

The CP Road Map

The Long-Term Plan for Concrete Pavement Research and Technology: The CP Road Map is a comprehensive and strategic plan for concrete pavement research that will guide the investment of approximately $250 million over the next 10 years. It will result in technologies and systems that help the concrete pavement community meet the paving needs of today and the as-yet unimagined paving challenges of tomorrow. In short, the CP Road Map will result in a new generation of concrete pavements for the 21st century.

Importance of the CP Road Map

Strategic—It combines more than 250 research problem statements into 12 fully integrated, sequential, and cohesive tracks of research leading to specific products that will dramatically affect the way concrete pavements are designed and constructed.

Innovative—From the way it was developed, to its unique track structure and cross-track integration, to the plan for conducting the research, the CP Road Map introduces a new, inclusive, and far-reaching approach to pavement research.

Stakeholder involvement—This CP Road Map plan is for the Federal, State, and private concrete pavement community. Peers helped create it, so it reflects all needs.

No ties to one agency or pot of money—Stakeholders with funds and expertise will pool their resources, jointly conduct and coordinate the research, and put the results into practice.

Research implementation—The plan incorporates innovative, effective research implementation to quickly move useful new products and systems to the field.

Figure 1 illustrates the CP Road Map unifying vision.

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

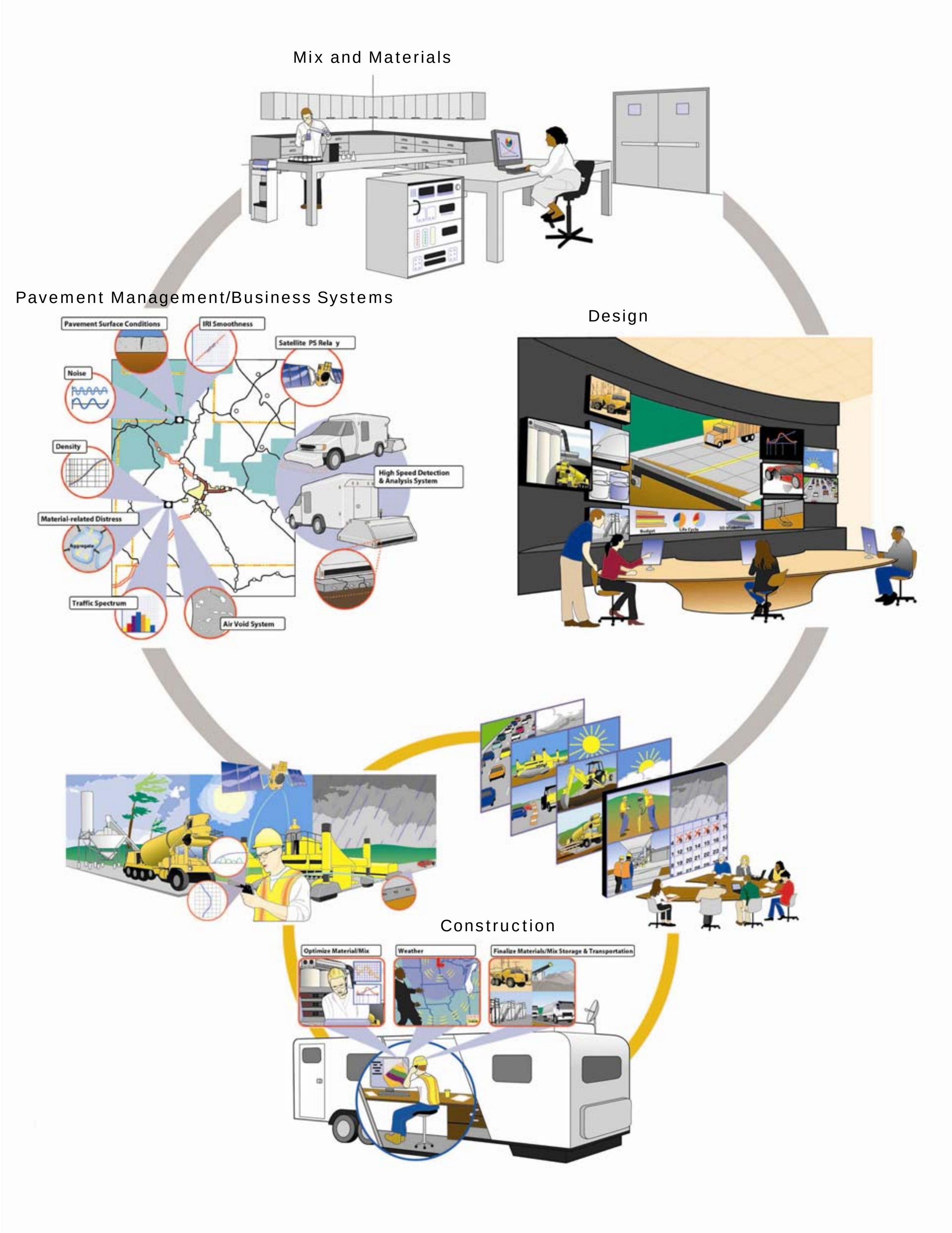

Figure 1. CP Road Map unifying vision

Research Tracks

Over 250 problem statements are grouped into 12 tracks:

Track 1. Performance-Based Concrete Pavement Mix Design System
Track 2. Performance-Based Design Guide for New and Rehabilitated Concrete Pavements
Track 3. High-Speed Nondestructive Testing and Intelligent Construction Systems
Track 4. Optimized Surface Characteristics for Safe, Quiet, and Smooth Concrete Pavements
Track 5. Concrete Pavement Equipment Automation and Advancements
Track 6. Innovative Concrete Pavement Joint Design, Materials, and Construction
Track 7. High-Speed Concrete Pavement Rehabilitation and Construction
Track 8. Long-Life Concrete Pavements
Track 9. Concrete Pavement Accelerated and Long-Term Data Collection
Track 10. Concrete Pavement Performance
Track 11. Concrete Pavement Business Systems and Economics
Track 12. Advanced Concrete Pavement Materials

Management Philosophy

- The CP Road Map is a national research plan, addressing Federal, State, and industry needs.
- The CP Road Map is not restricted to any single funding source. Coordinated sponsorship of research projects is proposed.
- It is possible—indeed, critical—for stakeholder groups to come together voluntarily. Federal, State, and industry research staff and engineers around the country are looking for more opportunities to pool their funds and other resources in win-win situations.
- The all-too-common disconnection between research results and implementation of those results must be fixed. Communication, technology transfer, and outreach activities must be elevated to the same level of importance as research itself.
- The CP Road Map is too comprehensive and too important for a part-time implementation effort. Managing the overall research program effectively and judiciously will require full-time, dedicated personnel with adequate resources.

Status of the CP Road Map

The Federal Highway Administration has indicated that it is committed to supporting execution of the CP Road Map. The American Concrete Pavement Association has also made a commitment to the plan.

Key words: concrete pavement—cooperation—management plan—research plan

Adaptive Predictive Traffic Timer Control Algorithm

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ABSTRACT

In this paper, we study the optimization of traffic light controllers and present an adaptive, predictive, and statistical optimization algorithm that performs dynamic queue length estimation. The system presented focuses on low power consumption, easy maintenance, and simple construction. The highlights of the system are (1) dynamic queue length estimation for timer delay computation and (2) the signal coordination algorithm it employs. Adaptive logic focuses on estimating the queue length during run time using sensors. The sensors need not be activated if a pattern is observed in the traffic flow. This forms the substratum for the predictive logic. Statistical data is used when the queue length exceeds a threshold. The green time for each traffic signal can be varied between a pre-estimated minimum and maximum, depending on the traffic flow. The red time for a particular signal depends on the green time of its complementary signal. The queue length detectors that we propose to use are fundamentally sensor networks that are composed of through-beam photoelectric sensors, arranged in an efficient topology. The efficiency of the algorithm has been estimated by conceptually applying the algorithm to a busy intersection in Chennai, India. The related statistical comparison with current systems has been presented. The algorithms have been simulated using a computer program written for the Turbo C++ compiler. An optimized signal coordination algorithm is presented that utilizes an online timing update technique for efficient traffic flow.

Key words: adaptive control—online timing update—signal plan optimization

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Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

A common function of a traffic control system is to minimize the delay experienced by vehicles traveling through a road intersection by manipulating the signal plans. The primary categories of a traffic control system are pre-timed and adaptive systems. Under pre-timed operation, the master traffic signal controller sets the signal parameters based on predetermined rates. These values are determined from historical data collected by observing the traffic flow. Common practice for developing pre-timed signal plans utilizes offline tools such as TRANSYT, which are based on traffic flows and queues observed from field data (McShane 1997). Pre-timed control frequently results in the inefficient usage of intersection capacity because of the inability to adjust to variations in traffic flow and actual traffic demand; this inefficiency is pronounced when flows are substantially below capacity. An adaptive controller overcomes the problem of a pre-timed controller by operating signals based on traffic demands. The green time for each approach can be varied between a minimum and maximum, depending on flows. The main feature of an adaptive controller is the ability to adjust the signal phase lengths in response to traffic flow.

Various efficient systems have been proposed. The most notable of these are SCOOT (Hunt 1982), developed in England, and SCATS (Lowrie 1982), developed in Australia. Both SCOOT and SCATS are adaptive-cyclic systems, in that they update the signal time plan at pre-specified time intervals. Other known methods under development over the last decade include PROLYN (Henry 1989), UTOPIA (Mauro 1990), OPAC (Gartner 1990), etc. These systems attempt to optimize traffic on-line without being confined to a cyclic time interval; i.e., the signal time plan may change at any time depending on the optimization algorithm. Compared to pre-timed signal control, these systems undeniably improve overall performance in terms of total delay in the controlled network. The usual improvements amount to roughly 10% (Boillot 1992).

This paper introduces an adaptive predictive signal control system that performs real time queue length estimation and employs an efficient signal coordination algorithm. The signal coordination algorithm proposed plans to optimize traffic at run time without being confined to a cyclic time interval. The real time queue length estimation is done by an array of simple through-beam photoelectric sensors placed in an efficient topology. Here the signal plan changes during each cycle based on inputs from the timer delay estimation algorithm. Though no new mechanism for detecting pedestrians has been proposed, the signal coordination algorithm takes care of pedestrian crossings as well. The timer values for pedestrians are based on historical data. The next section gives a detailed description of the proposed system architecture.

APTTCA SYSTEM ARCHITECTURE

This section presents the overall architecture of the proposed APTTCA-based system. The system consists of the following six components: (1) sensor network, (2) green time estimation module, (3) database, (4) signal plan design, (5) signal controller, and (6) traffic signal lights and timers. Figure 1 presents the architecture of the proposed system and the relationships between the various components. The bold lines indicate actual flow of control in the system. The dashed lines imply exclusive data flow.

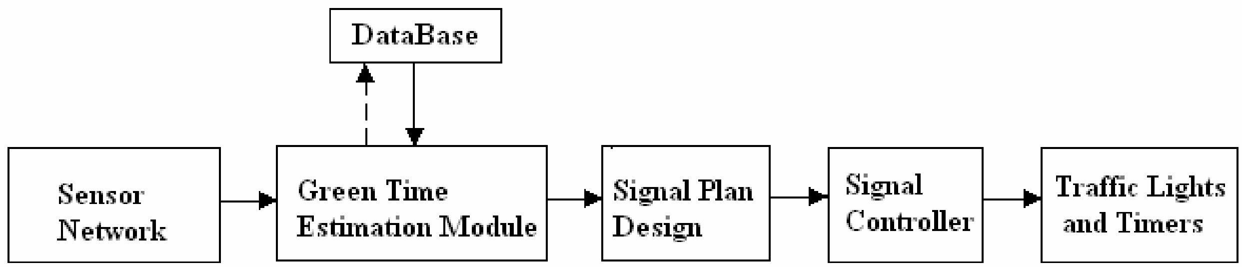

Figure 1. APTTCA system architecture

The core of the proposed system uses the queue lengths determined by the sensor network. The adaptive signal control logic attempts to respond directly to run time traffic variations. The green times are allotted based on the queue length estimated. The signal plan design module works on these values to provide an efficient traffic plan that ensures low average intersection delays and avoids starvation. The signal controller finally generates the necessary control signal for the output devices.

REAL TIME QUEUE LENGTH ESTIMATION: THE APTTC ALGORITHM

The heart of the system is its queue length estimation algorithm. The algorithm works by imposing an initial condition that when the signal is switched "on," all the lights (i.e. the traffic lights in the various directions) are red. The queue length detector is switched "on" in each direction. The queue length detectors are primarily arrays of photoelectric sensors. The number of sensors in the queue length detector array varies in each direction. The size of the sensor array (i.e. the number of sensors for a particular direction) is estimated by observing the nature of traffic in the intersection under consideration at all times of the day, distributed over a week.

All the sensors of the array are not activated at one time; they are turned "on" only when they are needed. This forms the basis of the power-efficient adaptive queue length estimation procedure. The procedure is explained below.

Adaptive Queue Length Estimation Algorithm

Initially, only the first and the last sensor pairs are activated. The reason behind the use of sensor pair outputs is that even if one sensor fails to detect a vehicle because the vehicle has stopped just before or just after a sensor, the other in the pair will surely respond. This is possible because the distance between two sensors has been designed suitably. The output of a pair is high if either of the sensors in the pair detects a vehicle. It is low if both the sensors of a pair fail to detect a vehicle. When the first and the last sensor pair outputs are examined, four cases arise.

The first case follows: if both do not detect a vehicle, then the green time is set to a minimum value, usually zero. After a small time interval, 't_{ic}' or "inter-computation time," the process is repeated.

The second case is the other extreme: both sensor pairs detect vehicles. In this case the Green time is set to a maximum value calculated from statistical data.

The third case is when the first sensor pair detects a vehicle and the last does not. This situation clearly indicates that the queue is not completely full. Here the algorithm takes a binary search-like approach. Assuming '2n' to be the number of sensors in the array, the $(n/2)^{nd}$ sensor pair is activated and its output taken. If it is high (i.e., it detects a vehicle), then the upper half becomes the area of interest with the $(n/2)^{nd}$ pair assuming the role of the first pair. Now the $(3n/4)^{th}$ sensor pair is switched "on" and the process continues until the first and the last sensor pair under consideration becomes the same. This is the length of the queue. This is used to calculate the green time.

The fourth case, namely the first pair not detecting a vehicle while the last pair detects a vehicle, is assumed to be impossible. This is because we consider sensor pair outputs, which at least theoretically do not commit such errors.

Predictive Logic

To begin, the algorithm is completely adaptive. This goes on for the first week of operation. The queue length estimated in real time over this period is stored in a database. This database now has information about the traffic flow at all times of the day, over a week. This data is sufficient to predict signal timer values in the near future.

After the first week of operation, say, on a Monday morning, the values obtained at, say, seven hours is compared with the value for Monday at seven hours in the database. If it matches, the values for the very next cycle of the signal are taken from the database and the sensors are not turned "on." The tolerance allowed is given by $t_{obs}=t_{db} \pm d$, where t_{obs} is the timer value observed at the moment, t_{db} is the database entry for the same day and time but in the preceding week, and d is the allowed deviation fixed at 10% of t_{obs}. If the above equality is not observed, t_{db} is replaced by t_{obs}. This implies a change, usually an increase in the number of vehicles passing the intersection. Hence this algorithm vouches for a significant change in traffic conditions.

Hence, the word "predictive" in the APTTCA context does not imply the conventional "predictive" commonly used in other traffic algorithms. Here we predict the timer values of the near future in the same intersection. This approach avoids the overhead due to unnecessary sensor activation and timer delay computation. This improves the efficiency of the system, reduces component wear and tear, and increases the life of the system.

Use of Statistical Data

Adaptive queue length estimation has been in consideration only because the pre-timed control systems, those using historical data, become inefficient when the flow of traffic is much less than the intersection capacity. Hence, suitably collected data obviously becomes the best choice when the queue lengths estimated are at their maximum. This data is stored in a database, which is subject to constant updating. The system is constantly observing the flow of traffic in the intersection. This information is used to update the statistical data stored in the database. Hence, a near-accurate timer delay estimation becomes feasible. If a traffic signal is constantly experiencing maximum queue lengths, it is an indication that the size of the sensor array has to be increased, since intersection usage has increased.

HARDWARE FOR REAL TIME QUEUE LENGTH ESTIMATION

The system proposed uses simple, well known, and inexpensive hardware for queue length estimation. This section is composed of subsections that deal with the technical specifications of the sensors, the topology in which they need to be arranged for effective queue length estimation, the optimal height at which they need to be placed, the optimal spacing between the sensors, and finally a comparison with the existing vehicle identification system.

Technical Specification of the Sensors

The most commonly used sensors are loop detectors, or photo sensors. The heart of our research is photo sensors. There are basically three types of photo sensors: through-beam, retro-reflective, and diffuse-reflective. The range at which the sensors need to operate is 50–100 ft. The only type that satisfies this requirement is the through-beam. Many manufacturers promise similar features. Table 1 shows a fraction of a datasheet provided by the Warner electric company.

Table 1. Sensor specification

Sensing principle	Sensing range	Input voltage	Output mode	Max cycle range	Output current	Model
Through Beam	500 ft	10 to 30 V DC	NPN	>25Hz	250 mA	MCS-629
Through Beam	50 ft	12 to 18 V DC	NPN	>250Hz	250 mA	MCS-627

Sensor Network Topology

A single photoelectric sensor cannot do the job. An array of sensors placed suitably is needed. The sensors should not interfere with pedestrians. Hence, they are placed in the intersection of the road and the pavement. We need to detect the vehicles waiting in each lane of the road. The transmitter of the first sensor is placed in the intersection of the road and the pavement, and its receiver is placed on the road, right on the lane separator line. The casing of this receiver has enough space for the transmitter of the sensor for the next lane. The receiver for this transmitter is placed on the subsequent lane separator. The final receiver (i.e., the receiver of the last lane detection array) is placed on the median. Figure 2 illustrates the sensor network topology for a four-lane intersection. Note from the figure that the number of sensors in each direction is not the same. It has been fixed based on the anticipated traffic flow and is subject to change.

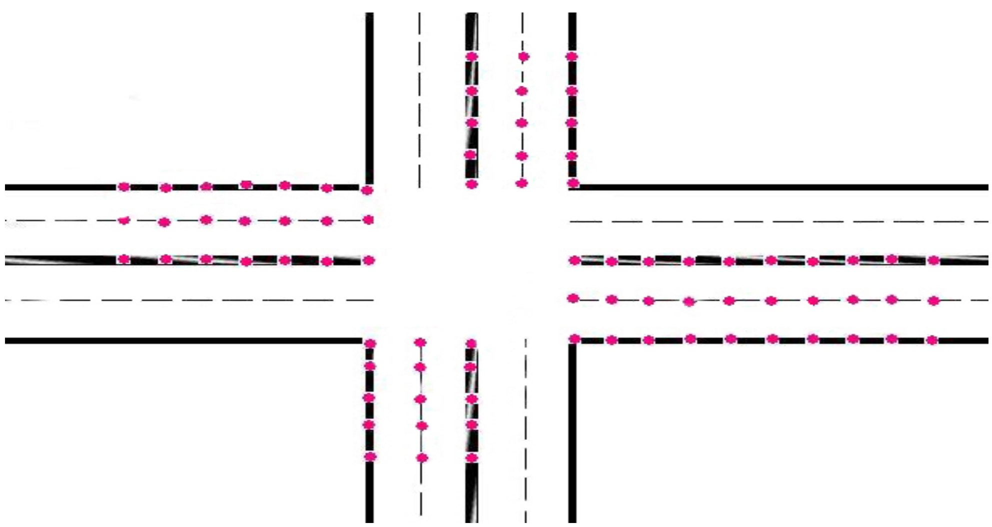

Figure 2. Sensor network topology for a four-lane intersection

Optimal Height of Placement

The sensors have to be placed at a comfortable height so that they can detect any vehicle. This has been estimated by conducting a study on various categories of commercial vehicles. The ground clearance information has been used to fix the optimal height. The highest ground clearance is found to be 18.5 inches for a dump truck. Providing a 0.5-inch tolerance, the height is identified as 19 inches. The study was conducted among 12 categories and 57 models of vehicles from 14 manufacturers around the world.

Sensor Spacing

The sensors have to be spaced so that they can effectively detect vehicles. They are also used to determine the length of the queue waiting at the signal. This information is multiplied by a suitable factor to obtain the green time for that direction. The spacing should not be so large that small vehicles that have placed themselves between the sensor pairs go undetected, nor should they be so small that more sensors will be needed for a small section of the road. This can be fixed based on a survey on various categories of vehicles in common use. The total length of the vehicle has been used as a parameter in determining the distance between the sensors. The length varies from 181 inches to 464 inches. A large number of vehicles have lengths lying between 220-230 inches. Thus, we can approximate the distance between the sensors to 240 inches. This distance has been verified manually and has an efficiency of over 65%.

Advantages

The proposed system promises a few non-performance–related advantages over current systems. Current systems generally use inductive loop detectors.

Robustness

The major advantage of an APTTCA-based system is the robustness and the ease of maintenance it offers. The queue length estimation system is not going to fail because of a single sensor malfunctioning. The job will be done by its partner. This is not the case with inductive loop detectors, whose failure leads to failure of the system. Thus an APTTCA-based system is more robust than existing vehicle detection systems.

Ease of maintenance

Another advantage of the proposed system is the ease of maintenance it offers. Inductive loop detectors are placed beneath the road, making it difficult to repair or replace the malfunctioning component. In this system, the sensors are placed 19 inches above the ground in a plastic casing, and hence replacing and repairing them is easy.

SIGNAL COORDINATION IN APTTCA

Adhering to the golden rule of traffic control, the signal coordination algorithm used in APTTCA gives priority to long queues. The inputs to the signal coordination module are the estimated timer delay values. The APTTC algorithm is applied to all directions possible at the intersection. Timer values for each direction are obtained from the estimated queue length. The queue length is added to the length of the intersection and the timer value is fixed as the time taken by a vehicle traveling at 15 kph to cover this distance. The timer values for the pedestrian crossings are obtained from historical data based on the time of the day. This means that the pedestrian delay value is picked up from the database based on the time and day of the week. The value for a pedestrian crossing on a Sunday afternoon will not be equal to that on a Monday morning.

The main aim of the coordination algorithm is to generate a signal plan that ensures efficient flow of traffic. This performs an online signal update, which means that the system is not cyclic and that no road user can predict what is going to happen next. This system is not completely dynamic. This means that if a direction has been allowed for, say, 60 seconds, it will not be considered until all other directions including the pedestrians have been given a chance. Completely dynamic systems may lead to starvation.

The Algorithm

The algorithm begins by obtaining all the green times for each direction of the signal. These are the demands of traffic. An algorithm is needed to meet this demand effectively. Each direction is associated with a green time. The input is an array of green times. The algorithm sorts this array in the descending order. Three arrays are maintained. They are the waiting array, the running array, and the completed array. The waiting array consists of the directions that are currently red, sorted in descending order. The running array is made up of those directions that are currently green. The completed array consists of those directions that have completed their turn. The timer display is turned off for those in the completed array.

Initially, the waiting array is full and the other two arrays are empty. The largest (i.e., the first) element of the array is the candidate that is made green, and the timer count is set to the estimated value. This entry is removed from the waiting array and is entered into the running array. The subsequent direction in the waiting array is taken. This is compared for compatibility with the entries in the running array; if it is not compatible, the red time for the chosen candidate is made equal to the green time of the contradicting entry. If there are no compatibility issues, then the chosen candidate is removed from the waiting array and appended to the running array. The entries in the running array are removed once their green time becomes zero. Their signal value is made red and the timer is turned off. This is repeated until the waiting array has no candidates. This ensures that there is no starvation. The pedestrians are also considered as a direction in this algorithm. Figure 3 depicts a six-lane–six-lane intersection with the various possible directions. The algorithm for the intersection shown will initially have 16 entries in its waiting array: 12 entries for vehicles and 4 for pedestrians. The diagram does not show the sensors.

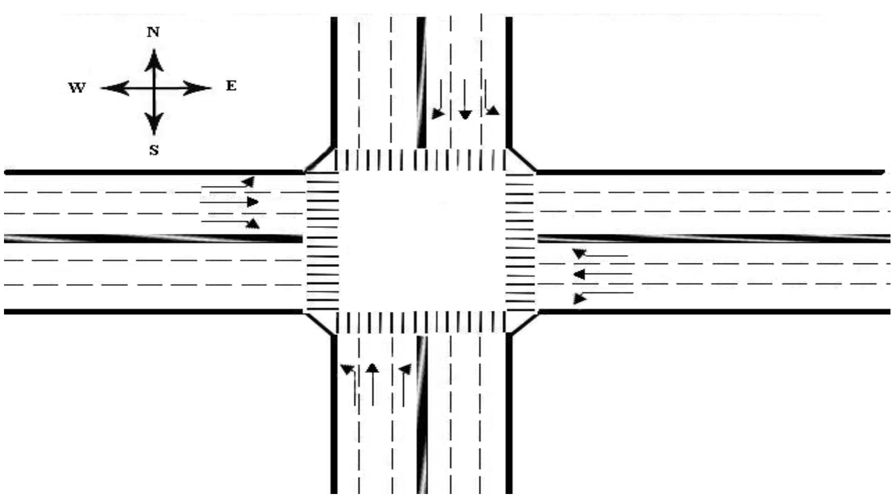

Figure 3. Six-lane–six-lane intersection

RESULTS

The algorithm has been theoretically implemented at the Mount Road-Venkatnarayana Road intersection in the city of Chennai, India. The sensors were not actually installed at the intersection, but queue lengths were determined manually at various times of the day for two full weeks. This queue length data is being used as the basis for the results derived. The algorithms were simulated using C++ programs. Table 2 shows the average values of the input data collected at the test site. This data has been used as input for the signal plan simulation program. The values shown were collected during various times of the day for two full weeks.

Table 2. Data collected at the test site

Day	Time of measurement (peak/off-peak)	Avg. queue length (meters)*	Avg. pre-timed timer value (seconds)	Avg. pre-timed timer based on time of day (seconds)	Avg. timer value APTTCA-based (seconds)
Monday	Off-peak	34.4	55	20	11.5
Monday	Peak	154.5	55	55	54
Wednesday	Off-peak	30.2	55	20	10.5
Wednesday	Peak	150.5	55	55	51
Friday	Off-peak	31	55	20	10.5
Friday	Peak	148.5	55	55	50
Sunday	N.A	64.5	55	45	24.5

* Manually estimated

The most important observation to be made is the amount of time wasted during weekends and off-peak periods using pre-timed control. Though pre-timed control that differentiates between peak and off-peak periods overcomes this, it can never be accurate as APTTCA-based systems. Thus, the results clearly indicate that APTTCA is efficient no matter the day or time. Figure 4 graphically depicts the variation in traffic flow.

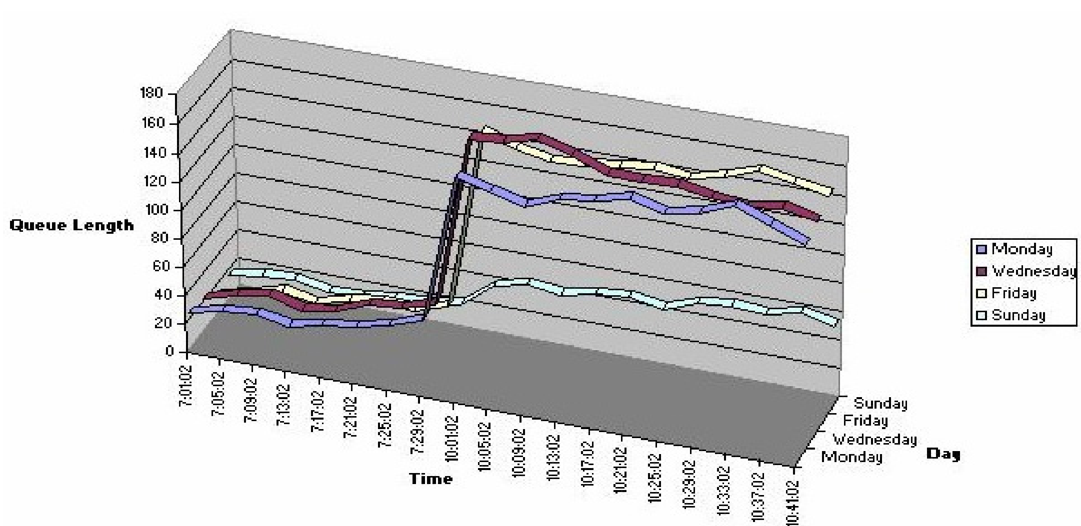

Figure 4. Variations in traffic flow

The signal plan simulation program considers the following: 1) pre-timed control, 2) pre-timed with time of day, and 3) APTTCA-based control. This program generates a signal plan and calculates parameters such as signal delay and throughput. The signal plan generation is based on the algorithm explained in the previous sections. The results are tabulated in Table 3.

Table 3. Performance results

Traffic system	Throughput per cycle (no. of vehicles)*	Average delay (sec/vehicle)	Performance improvement based on avg. delay %
Pre-timed control	235	45.5	N.A
Pre-timed control based on time of day	221	40	12.22
APTTCA-based control	218	37.5	17.58

* Includes pedestrians

The results show that the proposed system clearly outperforms the older systems. Though the throughput values are better for a pre-timed control system, the real measure of performance is the average delay. The APTTCA-based system clearly promises less waiting time. The results shown are average values from five simulation runs.

RECOMMENDATIONS

The next advancement to the system proposed is regarding pedestrians. An efficient technology that detects the number of pedestrians waiting to cross the road should be developed. This can be done using systems such as video recorders or even photoelectric sensors. The system was developed for a single intersection. A natural extension would be to address intersection coordination. Now that we have developed an efficient traffic control system, we must work to control the genesis efficiently (i.e., provide efficient control over the drivers of various vehicles). In order to do this, an efficient navigation system should be developed and communicated efficiently. A scheme compatible with APTTCA should also be developed. Another type of improvement is to use more dynamic signal coordination algorithms. These can surely improve the efficiency of the system.

ACKNOWLEDGEMENTS

First of all we would like to thank our parents and our sisters for their unconditional love and numerous sacrifices. We thank the authorities of the Chennai City Traffic Police (CCTP) for providing us with valuable information. Not only did they share their knowledge with us but also gave us permission to observe the Mount Road-Venkatnarayana Road intersection in Chennai, India throughout the day for two full weeks. We offer thanks to our friends Mr. Karthik, Mr. Sriram, Mr. Venkatesh, Mr. Karthik Mohan, Mr. Bhadri Narayanan, and Mr. Marimuthu for toiling in the hot sun, helping us collect valuable data.

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Evaluation and Modifications of the AASHTO Procedures for Flexural Strength of Prestressed Concrete Flanged Sections

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ABSTRACT

Different interpretations of the equivalent rectangular stress block approximation used by the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) and AASHTO Standard Specifications may lead to inconsistencies in the sectional response of the prestressed concrete flanged sections predicted by the two specifications. According to the LRFD specifications, the limits on the contribution of the compression flange overhangs causes the neutral axis to be located lower in the web of the section to satisfy the internal force equilibrium. This artificial overestimation of the neutral axis depth according to the LRFD Specifications as compared to the Standard Specifications, leads to inconsistencies in the provisions that depend on neutral axis depth, such as whether or not the section is over-reinforced, as well as differences in nominal moment capacity.

In this paper, provisions of the AASHTO LRFD and Standard Specifications are compared to the results from the nonlinear strain compatibility analyses of prestressed concrete nonrectangular sections. The purposes of this research are to illustrate the discrepancies that exist in the sectional response predicted by these specifications and to identify design procedures that more accurately represent the actual behavior of such members.

Modifications to the AASHTO LRFD procedure are proposed to correct for errors in determining the contribution of compression flange overhangs. Improvements in the accuracy of predicted sectional response and maximum reinforcement limits are demonstrated through a set of examples. Flexural strengths predicted by the specifications, the proposed modified LRFD procedure, and the strain compatibility analyses were also compared to measured flexural strengths of prestressed concrete I-beams found in the literature to validate the proposed modifications.

Key words: equivalent stress block—load and resistance factor design—prestressed concrete—standard specifications—T-section

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PROBLEM STATEMENT

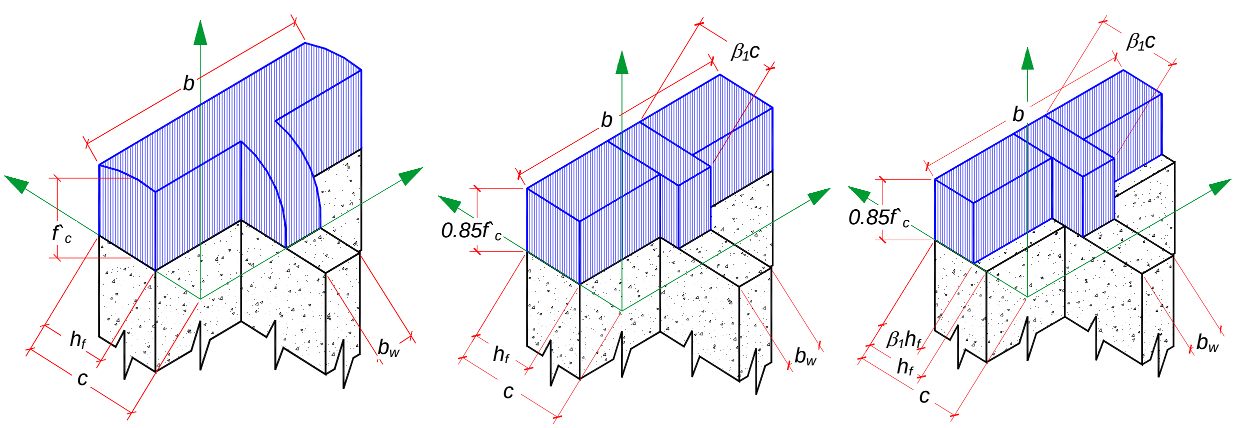
Inconsistencies in the sectional response of prestressed concrete flanged sections, as predicted by the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications (2002) and the AASHTO Load and Resistance Factor Design (LRFD) Specifications (1998), may arise due to different interpretations of the equivalent compressive stress block idealization (Badie and Tadros 1999; Seguirant 2002; Naaman 2002; Naaman 2002a; Rabb 2003; Girgis, Sun, and Tadros 2002; Weigel, Seguirant, Brice, and Khaleghi 2005; Seguirant, Brice, and Khaleghi 2005). For sections with T-section behavior at ultimate capacity, an artificial overestimation of the neutral axis depth according to the LRFD Specifications, as compared to the Standard Specifications, leads to inconsistencies in the provisions that depend on neutral axis depth, such as whether or not the section is over-reinforced, as well as differences in nominal moment capacity.

To identify the inconsistencies, the sectional responses of several prestressed concrete nonrectangular sections were predicted by the AASHTO LRFD and Standard Specifications (Baran, Schultz, and French 2005). The sections were also analyzed using strain compatibility analysis that incorporated nonlinear material properties. This paper presents a comparison of the sectional response that the specifications predicted and that determined by the nonlinear strain compatibility analysis. A comparison of the measured flexural strengths of prestressed concrete I-beams found in the literature with those predicted by the specifications and the proposed procedure is also included.

BACKGROUND

Equivalent Stress Block Approximation

Both the AASHTO Standard Specifications and the AASHTO LRFD Specifications approximate the actual nonlinear concrete compressive stress distribution at nominal capacity (Figure 1) with the Whitney rectangular stress block, which has an average compressive stress of $0.85f'_c$ "uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a line parallel to the neutral axis at a distance $a=\beta_1 c$ from the extreme compression fiber." However, there are two differences in the implementation of the equivalent rectangular compression block according to the two specifications. As illustrated in Figure 1, the LRFD Specifications limit the depth of the compression flange overhangs contributing to the internal compressive force to $\beta_1 h_f$. In the case of AASHTO Standard Specifications, on the other hand, a full compression flange depth of h_f can contribute to the internal compressive force.



(a) Actual Nonlinear Stress Distribution (b) AASHTO Standard Specifications (c) AASHTO LRFD Specifications

Figure 1. Compressive stress distributions for a T-section

The second difference between the two specifications is in the limiting value of the neutral axis depth at which the transition from rectangular section behavior to T-section behavior occurs. According to the Standard Specifications, T-section behavior starts when $a=\beta_1 c$ becomes equal to the compression flange depth, h_f, while the LRFD Specifications consider the section as a T-section as soon as c reaches h_f.

The differences in the way the two specifications treat the overhanging portions of the top flange of nonrectangular sections result in overestimation of the neutral axis depth, according to the LRFD Specifications, which leads to inconsistencies in the determination of whether the section is considered over-reinforced, as well as in the resulting flexural capacity.

Reinforcement Limits

The amount of tensile reinforcement that can be placed in a prestressed concrete section is limited in both the LRFD and the Standard Specifications. The two specifications define the maximum reinforcement limits in terms of different parameters. AASHTO Standard Specifications limit the maximum value of reinforcement index (Equation 1-a), ω_w, while the LRFD Specifications limit the value of the ratio of neutral axis depth, c, to effective depth, d_e (Equation 1-b).

$$\omega_w = \frac{A_{sr} f_{ps}}{b_w d f'_c} \leq 0.36\beta_1 \qquad (1\text{-a})$$

$$\frac{c}{d_e} \leq 0.42 \qquad (1\text{-b})$$

Even though the Standard and the LRFD Specifications define the maximum reinforcement limit using different criteria, two parameters (the reinforcement index, ω_w, and the c/d_e ratio) are related to each other through the equilibrium of internal tensile and compressive forces.

Limiting the maximum tensile reinforcement in flexural members dates back to the 1971 edition of the ACI Code (1971), which placed an upper limit of 0.30 on the reinforcement index, ω, which is directly proportional to the amount of tensile reinforcement. The first appearance of a limit on the reinforcement index equal to 0.30 is found in the report titled “Tentative Recommendations for Prestressed Concrete” by the ACI-ASCE Joint Committee 323 (1958). The justification for such a limit was expressed as the need “to avoid approaching the condition of over-reinforced beams for which the ultimate flexural strength becomes dependent on the concrete strength.”

In both the LRFD and Standard Specifications, the sections with tensile reinforcement exceeding the maximum reinforcement limit are termed “over-reinforced.” By preventing the use of the full flexural capacity of over-reinforced sections, the specifications impose an additional safety margin to account for the limited ductility of those sections. In other words, the specifications permit the use of prestressed concrete sections with steel amounts exceeding the maximum limit, but with a usable flexural strength that is less than the actual strength of the section.

Even though the specifications penalize the use of over-reinforced sections by making a trade-off between ductility and strength, Provision 5.7.3.3.1 of the AASHTO LRFD Specifications states that “Over-reinforced sections may be used in prestressed and partially prestressed members only if it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved.” This statement effectively penalizes the design and use of prestressed and partially prestressed over-reinforced sections more severely than provisions that simply limit the flexural resistance, as is done in the Standard Specifications.

Strength Considerations

As mentioned earlier, AASHTO Standard Specifications assume that T-section behavior begins when the depth of the equivalent rectangular compressive stress block, $a=\beta_1 c$, drops below the top flange of the section. In this case, Equation 2-a is the expression for the internal equilibrium of the compressive force in the concrete and tensile force in the prestressing steel. As seen in the second term of the left-hand side of Equation 2-a, the Standard Specifications allow the full flange depth of h_f to contribute to the total compressive force carried by the section. In this case, the contribution of the top flange overhangs to the total internal compressive force is $0.85 f'_c (b - b_w) h_f$.

$$\left(0.85 f'_c\right)\left(\beta_1 c\right) b_w + \left(0.85 f'_c\right) h_f \left(b - b_w\right) = A_{ps} f_{ps} \tag{2-a}$$

In the case of AASHTO LRFD Specifications, the expression for the internal force equilibrium of a T-section is the one given in Equation 2-b. As seen in the second term of the left-hand side of the equilibrium expression, the LRFD Specifications limit the depth of the equivalent rectangular stress block acting on the flange overhangs to $\beta_1 h_f$. This implicitly means that, in the case of a T-section, the full depth of the top flange never contributes to the total compressive force when the equivalent rectangular stress block assumption is used, regardless of the magnitude of c. This assumption results in an overestimation of the neutral axis depth, c, for the LRFD Specifications in comparison with the Standard Specifications, since the web contribution must increase to compensate for the portion of the top flange that is being neglected.

$$\left(0.85 f'_c\right)\left(\beta_1 c\right) b_w + \left(0.85 f'_c\right)\left(\beta_1 h_f\right)\left(b - b_w\right) = A_{ps} f_{pu} \left(1 - k \frac{c}{d_p}\right) \tag{2-b}$$

Strand Stress

The LRFD and Standard Specifications use different procedures to predict the stress in the prestressing steel at nominal capacity. In the procedure used by the Standard Specifications, the strand stress is predicted by Equation 3-a, which is independent of the neutral axis depth.

$$f_{ps} = f_{pu}\left[1 - \left(\frac{\gamma}{\beta_1}\right)\left(\rho \frac{f_{pu}}{f'_c}\right)\right] \tag{3-a}$$

In the LRFD procedure, on the other hand, the location of the neutral axis is determined first using Equation 2-b, which implicitly includes an assumed value for the strand stress. With this estimate of the neutral axis location, strand stress is computed using Equation 3-b.

$$f_{ps} = f_{pu}\left(1 - k \frac{c}{d_p}\right) \tag{3-b}$$

As seen, the equation used by the Standard Specifications (Equation 3-a) is independent of the neutral axis depth. At the ultimate capacity of a prestressed concrete section, any change in the neutral axis location will result in a change in the strand strain, and hence in the strand stress. The procedure used by the Standard Specifications to determine the strand stress cannot represent this behavior. The LRFD procedure, on the other hand, satisfactorily takes the changes in neutral axis location into account when determining the strand stress at ultimate capacity.

Nominal Moment Capacity

Both the AASHTO Standard and AASHTO LRFD Specifications use formulas for computing the flexural strength of over-reinforced sections that differ from those used for under-reinforced sections. The AASHTO Standard and LRFD Specifications use Equations 4-a and 4-b, respectively, for the calculation of moment capacity of under-reinforced prestressed concrete sections.

$$M_n = A_{sr} f_{ps} d\left[1 - 0.6\left(\frac{A_{sr} f_{ps}}{b_w d f'_c}\right)\right] + 0.85 f'_c (b - b_w) h_f (d - 0.5 h_f) \tag{4-a}$$

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85 f'_c (b - b_w)(\beta_1 h_f)\left(\frac{a}{2} - \frac{h_f}{2}\right) \tag{4-b}$$

For the moment capacity of over-reinforced sections, on the other hand, the Standard and LRFD Specifications recommend using Equations 5-a and 5-b, respectively.

$$M_n = \left(0.36\beta_1 - 0.08\beta_1^{\ 2}\right) f'_c b_w d^2 + 0.85 f'_c (b - b_w) h_f (d - 0.5 h_f) \tag{5-a}$$

$$M_n = \left(0.36\beta_1 - 0.08\beta_1^{\ 2}\right) f'_c b_w d_e^{\ 2} + 0.85 f'_c (b - b_w)(\beta_1 h_f)(d_e - 0.5 h_f) \tag{5-b}$$

The last two equations are obtained by substituting into Equations 4-a and 4-b, respectively, the maximum amount of tensile reinforcement allowed by Equations 1-a and 1-b. Through the use of Equations 5-a and 5-b, in effect, the flexural strength of over-reinforced sections is limited to the value of the moment capacity corresponding to the maximum limit of tensile reinforcement. Any additional capacity that may be provided by having more steel than allowed by the reinforcement limits is neglected. This limitation on moment capacity is intended to ensure that sections with limited ductility have reserve moment capacity.

RESEARCH METHODOLOGY

This paper summarizes some of the findings of a study conducted at the University of Minnesota to investigate precast concrete T-section behavior at nominal strength. The study included a comparison of sectional analyses of several reinforced and prestressed concrete nonrectangular sections, following the procedures given in the AASHTO Standard Specifications, the AASHTO LRFD Specifications, a nonlinear strain compatibility analysis, and a modified LRFD procedure (Baran, Schultz, and French 2005).

In addition, analytical results were compared to the test results of prestressed concrete I-beams found in the literature, which were identified as over-reinforced and as having neutral axis depths within the web at nominal strength.

Strain compatibility analyses were conducted using RESPONSE-2000, a sectional analysis program by Bentz and Collins (2001) incorporating a nonlinear stress-strain material model. A computer code utilizing internal force equilibrium and strain compatibility between steel and concrete was also developed to verify the results obtained using RESPONSE-2000. For this paper, the results of sectional analyses performed using RESPONSE-2000 were used to make comparisons between the sectional responses predicted by the AASHTO Specifications. When the term “strain compatibility” is used, it refers to the strain compatibility analyses performed using RESPONSE-2000 with nonlinear material models for the concrete and the steel.

ANALYSIS OF SECTION BEHAVIOR

Limits of T-Section Behavior

Analyses were conducted to reproduce the results given in Figure C5.7.3.2.2-1 of the LRFD Specifications (Figure 2 in this paper). This chart, originally developed by Naaman (1992), is used to indicate the difference in neutral axis depth with the amount of steel for the AASHTO LRFD and AASHTO Standard (and ACI) Specifications for the T-section shown. Note that this figure as given in the LRFD Specifications contains an incorrect interpretation of the neutral axis depth for the ACI and AASHTO Standard Specifications.

To compare the neutral axis locations calculated according to the specifications with those from the strain compatibility study (superimposed on the chart with triangle symbols), the plot has been divided along the x-axis into three regions to compare the neutral axis depths. Region I represents those cases for which c is always smaller than h_f, and both the LRFD and Standard Specifications indicate rectangular section behavior. As shown, in this region the neutral axis depth values calculated according to both specifications with the equivalent rectangular stress block assumption agree with the values obtained from the nonlinear strain compatibility analyses.

Region II in Figure 2 covers those cases for which the neutral axis begins to drop below the flange. In this region, the LRFD Specifications indicate T-section behavior while the Standard Specifications still indicate that the section behaves as a rectangular section. In Region II, the Standard Specifications approximate the location of the neutral axis with acceptable accuracy compared to the strain compatibility analyses, while the procedure in the LRFD Specifications grossly overestimates the neutral axis depth.

In Region III, T-section behavior is predicted by both the LRFD and Standard Specifications. As shown in Figure 2, for a specific value of steel area, both specifications overestimate the neutral axis depth compared to the strain compatibility analyses, but with much larger errors for neutral axis depth calculations using the LRFD Specifications.

The dashed line in Figure 2 is used in provision C5.7.3.2.2 of AASHTO LRFD Specifications to show the inconsistency between the AASHTO LRFD and the AASHTO Standard and ACI Specifications. This dashed line is actually a misinterpretation of the neutral axis depth in the ACI and Standard Specifications. The plot is based on the assumption that T-section behavior initiates when $c \geq h_f$, taking the total depth of the flange as effective in compression at that point, and thereby indicating a negative neutral axis depth in the web for equilibrium. The correct assumption, denoted by the solid line in Figure 2, is that T-section behavior initiates when the depth of the equivalent rectangular stress block exceeds the flange depth (i.e., $a = \beta_1 c \geq h_f$).

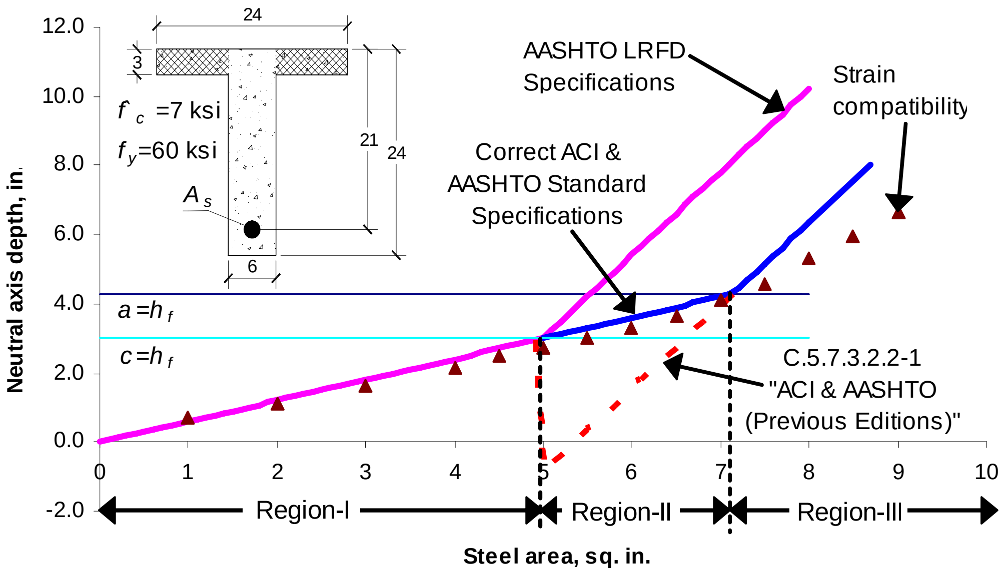


Figure 2. Change in neutral axis depth with amount of steel in a T-section

MODIFICATION OF LRFD PROCEDURE

Proposed Changes

As noted earlier, the LRFD Specifications overestimate the neutral axis depth for two reasons; the first reason is the use of $c=h_f$ as the limit for T-section behavior, and the second reason is the use of the $\beta_1 h_f$ limit for the maximum flange overhang contribution to the internal compressive force once T-section behavior begins. A modification, which overcomes both of these problems, was proposed to the procedure outlined in the LRFD Specifications to indicate that T-section behavior begins when $a = \beta_1 c = h_f$, rather than when $c = h_f$ (Baran, Schultz, and French 2005). This modification also fixes the second problem mentioned above by enabling the entire flange depth to become effective when $a \geq h_f$.

Because the neutral axis depth and moment capacity of under-reinforced sections and the moment capacity of over-reinforced sections depend on the amount of flange overhang contribution to the internal compressive force, the corresponding equations in the LRFD Specifications (Equations 5.7.3.1.1-3, 5.7.3.2.2-1, and C5.7.3.3.1-2) were also modified to remove the $\beta_1 h_f$ limit on the contribution of the flange overhangs. After the modification, Equations 5.7.3.1.1-3, 5.7.3.2.2-1, and C5.7.3.3.1-2 in the LRFD Specifications (Equations 2-b, 4-b, and 5-b, respectively, in this paper) take the forms of Equations 6, 7, and 8, respectively.

$$(0.85 f'_c)(\beta_1 c) b_w + (0.85 f'_c)(h_f)(b - b_w) = A_{ps} f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad (6)$$

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right) \quad (7)$$

$$M_n = \left(0.36\beta_1 - 0.08\beta_1^{\,2}\right) f'_c\, b_w d_e^{\,2} + 0.85 f'_c (b - b_w) h_f \left(d_e - 0.5 h_f\right) \quad (8)$$

The procedure in the LRFD Specifications using Equations 6, 7, and 8 instead of Equations 5.7.3.1.1-3, 5.7.3.2.2-1, and C5.7.3.3.1-2 is referred to as the modified LRFD procedure in the rest of this paper.

Verification of Proposed Changes

The Minnesota Department of Transportation (Mn/DOT) Type 63 section shown in Figure 3 was analyzed for various amounts of prestressed steel to investigate the relationship between the sectional response predicted by the LRFD and Standard Specifications, strain compatibility analyses, and the modified LRFD procedure. This section is currently being used in Minnesota for prestressed concrete through-girder pedestrian bridge construction with typical spans on the order of 135 ft. Because of the through-girder–type construction, no composite deck exists on top of the girders, and there is interest in a more accurate evaluation of the strength and ductility of these girders.

The large span length requires the use of a large number of strands to control deflections. In addition, the section has a narrow top flange ($b_{flange}/b_{web} = 5$). Because neither a composite deck nor a wide top flange are provided to help carry the compressive part of the internal couple, the neutral axis is located within the web of the section, causing the section to be over-reinforced according to the LRFD Specifications, and consequently causing it to fail to meet the required strength. With the narrow top flange, the difference between the response of the section predicted by the LRFD Specifications and Standard Specifications is less significant than it would be for a section with a wider top flange. From this aspect, this section provides a lower bound for the difference between the sectional quantities predicted by the LRFD and Standard Specifications.

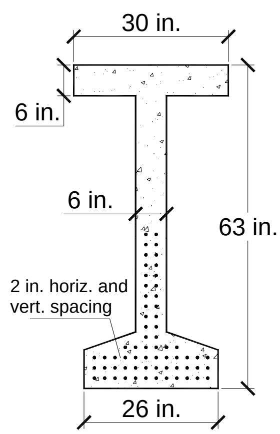


Figure 3. Mn/DOT type 63 section

The Mn/DOT Type 63 section was analyzed assuming an 8.2-ksi concrete strength and 0.5-in. diameter strands with an effective prestress of 162 ksi ($0.60f_{pu}$). The number of strands varied from 20 to 60, and the strands were placed in the typical pattern used for this type of section, that is, spaced two inches on-center in the horizontal and vertical directions. Thus, the depth to the center of gravity of strands was lowered as the number of strands increased. The results of the analyses are shown in Figure 4.

Figure 4a is similar to Figure 2, which was for a 24-in. deep reinforced concrete T-section. For the present case, the LRFD Specifications assume that T-section behavior starts when c exceeds h_f, and, as shown in Figure 4, the LRFD Specifications begin to overestimate the neutral axis depth when there are 20 strands in the section. The Standard Specifications, on the other hand, indicate almost the same neutral axis location as the strain compatibility analysis until the number of strands is increased to 32. For this amount of prestressed steel, the Standard Specifications begin to treat the section as a T-section, and overestimate *c* in comparison to the strain compatibility analysis.

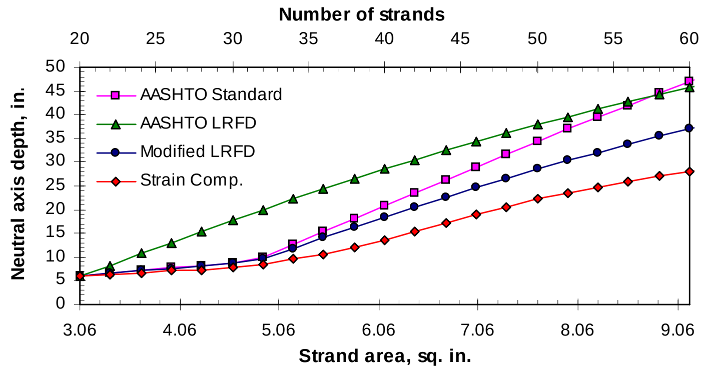
The neutral axis depth values computed with the modified LRFD procedure are also shown in Figure 4a. As for the Standard Specifications, T-section behavior starts at 32 strands for the modified LRFD procedure. As seen in Figure 4, there is better agreement between the neutral axis depth values computed with the modified LRFD approach and the strain compatibility results than there is with either the Standard or the LRFD Specifications.

The change in strand stress at the ultimate capacity of the section is shown in Figure 4b. As illustrated, the LRFD Specifications underestimate the strand stress compared to the strain compatibility analysis, while the Standard Specifications slightly overestimate it. When the LRFD Specifications are modified, as mentioned previously, the results fall into close agreement with those obtained by the strain compatibility analysis.

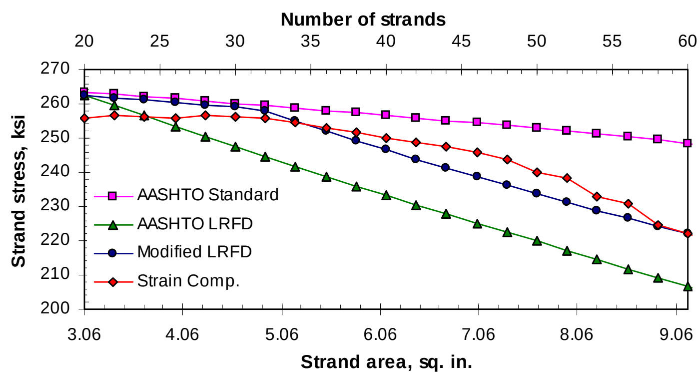
Figure 4c shows the variation among the nominal bending resistances calculated according to the LRFD, Standard Specifications, modified LRFD procedures, and strain compatibility analysis as the amount of prestressed steel is varied. Once T-section behavior begins (i.e., when there are 20 strands according to the LRFD Specifications) the LRFD Specifications begin to underestimate bending capacity. As the number of strands increases, the depth of the web participating in the internal compressive force increases until the section becomes over-reinforced at 38 strands.

After the section becomes over-reinforced (i.e., at 44 strands according to the Standard Specifications, at 38 strands according to the LRFD Specifications, and at 46 strands according to the modified LRFD procedure) Equations 5-a, 5-b, and 8 are used to compute the nominal bending resistance according to the Standard Specifications, the LRFD Specifications, and the modified LRFD procedure, respectively. These equations include only the geometric terms of the section and the concrete strength, and are independent of the amount of steel present in the section. As seen in Figure 4-c, the moment capacity of the section decreased after the section became over-reinforced according to the specifications. This occurred because the LRFD and Standard Specifications use the distance from the extreme compression fiber to the centroid of the strands to compute the moment capacity of over-reinforced sections, and this distance decreases with increasing number of strands as the strands were distributed through the depth of the section in this example.

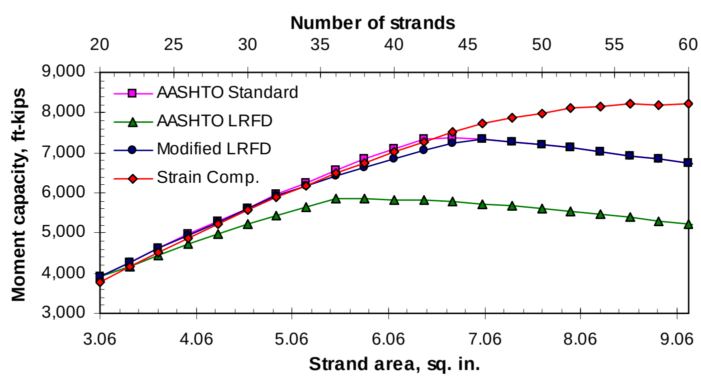
Note that once the section became over-reinforced according to the Standard Specifications and the modified LRFD procedure, both specifications (modified LRFD and Standard) indicated the same moment capacity values for an increasing number of strands. This is so because, according to both procedures, the moment capacity of over-reinforced sections is computed based solely on the compressive portion of the internal couple.



(a) Neutral axis depth, *c*.



(b) Strand stress at nominal capacity, f_{ps}.



(c) Moment capacity, M_n.

Figure 4. Variation in section response with amount of prestressed steel (63-in. deep section)

The inconsistency described above between the moment capacities of over-reinforced sections severely limits practitioners' choices, as the LRFD Specifications penalize the use of these so-called over-reinforced sections in two ways: (1) by placing a conservative limit on nominal bending resistance, and (2) by requiring additional analyses and experimentation to show that there is sufficient ductility. As shown in Figure 4, modifying the LRFD procedure as described earlier minimizes the inconsistency between the moment capacities calculated according to the LRFD and Standard Specifications.

Validation with Experimental Data

Measured flexural strengths of 38 12 in.-deep prestressed concrete I-beams tested in flexure by Hernandez (1958) were compared to the strengths predicted by the LRFD and Standard Specifications, the proposed modified LRFD procedure, and the strain compatibility analysis. Reported material properties were used for each beam and the tensile strength of the concrete was neglected. In the experimental study, two different reinforcement ratios were used with two different cross sections, as shown in Figure 5. Even though the nominal dimensions of all of the beams were the same, the web and flange dimensions varied slightly. In the analyses, the reported actual dimensions were used.

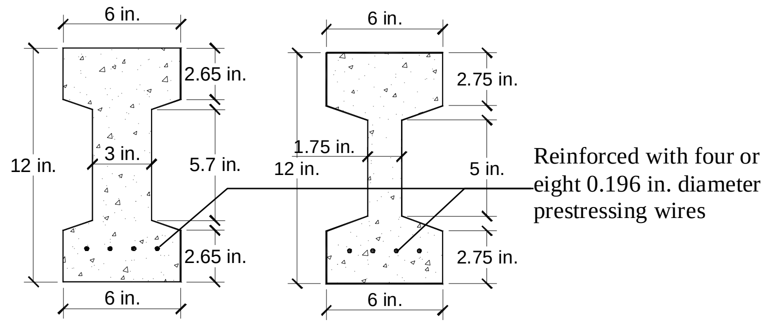


Figure 5. Nominal dimensions of beam sections tested by Hernandez (1958)

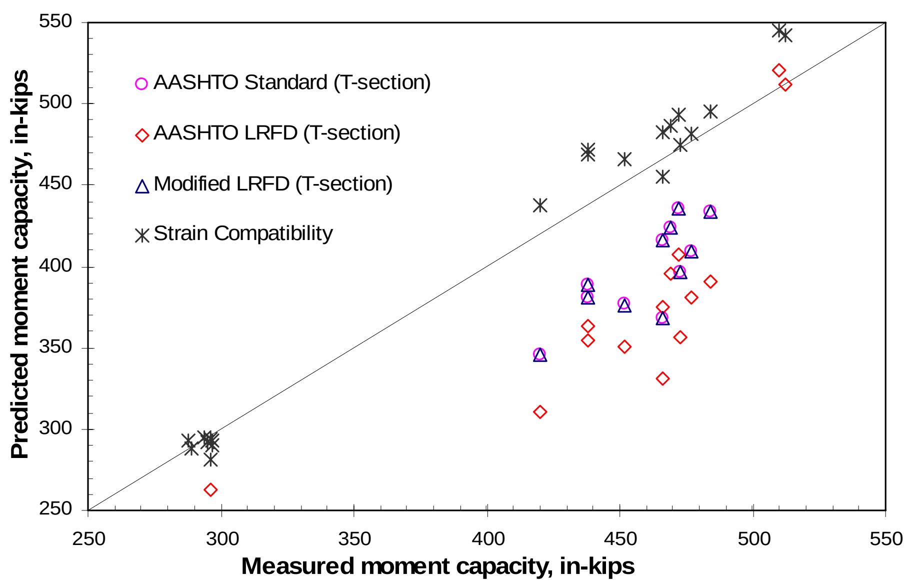


Figure 6. Comparison of predicted and measured moment capacities

Figure 6 provides a comparison of the predicted and measured moment capacities. For the sake of clarity, only the sections with T-section behavior at ultimate capacity according to the specifications are included in the figure. As shown in Figure 6, moment capacities predicted by the strain compatibility analysis were in agreement with the test results.

The reason that the Standard Specifications and the modified LRFD procedure predicted the same moment capacities for the beams is because all of the beams shown in the figure were considered over-reinforced according to the LRFD, Standard Specifications, and the modified LRFD procedure and because the Standard Specifications and the modified LRFD procedure use identical equations (Equations 5-a and 8, respectively) for the moment capacity of over-reinforced sections.

As evident in Figure 6, using a maximum tensile reinforcement limit and computing the moment capacity of over-reinforced sections with a different formula, which is based on the amount of tensile reinforcement corresponding to the maximum reinforcement limit, implicitly places a safety factor on the computed moment capacity of sections. Figure 6 also shows that the AASHTO LRFD Specifications grossly underestimated the nominal moment capacities, therefore resulting in a larger safety factor.

CONCLUSIONS

The following conclusions were drawn from this study:

1. Different interpretations of the equivalent rectangular stress block idealization used by the AASHTO LRFD and AASHTO Standard Specifications resulted in inconsistencies in the sectional response of nonrectangular prestressed concrete sections predicted according to these specifications.
2. Limiting the contribution of top flange overhangs to the internal compressive force caused an overestimation of the neutral axis depth of T-sections according to the LRFD Specifications, which in turn leads to the section being prematurely considered as over-reinforced. The tendency to classify some sections prematurely as over-reinforced results in large differences between the moment capacities predicted by the AASHTO LRFD Specifications and the other methods.
3. Limiting the maximum amount of tensile reinforcement to be used in determining the moment capacity, as used in the AASHTO LRFD and Standard Specifications, provides an additional safety margin to account for the poor flexural ductility of sections with large amounts of tensile reinforcement.
4. Modifying the procedure of the AASHTO LRFD Specifications by changing the T-section limit from $c = h_f$ to $a = h_f$ reduces the inconsistencies between the sectional response and the maximum reinforcement limits predicted by the AASHTO LRFD Specifications and the other methods (AASHTO Standard Specifications and the strain compatibility analyses). With this modification, the $\beta_1 h_f$ maximum limit for the depth of the top flange overhang contribution to the internal compressive force in the LRFD Specifications is automatically removed.
5. The AASHTO Standard Specifications does not take into account the effect of changes in the neutral axis location caused by changes in top flange depth when calculating the strand stress at ultimate moment capacity. In this respect, the LRFD procedure for strand stress provides a more realistic sectional response. Thus, it is proposed that the LRFD strand-stress relation be used with the modified procedure.

ACKNOWLEDGMENTS

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NOTATION

a = depth of equivalent rectangular stress block, in.
A_{ps} = area of prestressing steel, sq.in.
A_s = area of nonprestressed tension reinforcement, sq.in.
A_{sr} = prestressing steel area required to develop the ultimate compressive strength of the web of the section, sq.in.
b = width of the compression face of the member, in.
b_w = web width, in.
c = distance from the extreme compression fiber to the neutral axis, in.
d = distance from extreme compression fiber to centroid of the prestressing force, in.
d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement, in.
d_p = distance from extreme compression fiber to the centroid of the prestressing tendons, in.
f'_c = specified compressive strength of concrete, ksi
f_{ps} = average stress in prestressing steel at ultimate load, ksi
f_{pu} = specified tensile strength of prestressing steel, ksi
h_f = compression flange depth, in.
k = factor for type of prestressing tendon
= 0.28 for low-relaxation steel
= 0.38 for stress-relieved steel
= 0.48 for bars
M_n = nominal flexural resistance, in.-kips
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone
γ, γ_p = factor for type of prestressing tendon
= 0.28 for low-relaxation steel
= 0.40 for stress-relieved steel
= 0.55 for bars
ρ = prestressed reinforcement ratio
= A_{ps}/bd_p
ω_w = reinforcement index considering web of flanged sections

$$= \frac{A_{sr} f_{ps}}{b_w d f'_c}$$

Rapid Pavement Backcalculation Technique for Evaluating Flexible Pavement Systems

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ABSTRACT

This study focuses on the use of artificial neural network (ANN)-based pavement backcalculation tools for analyzing falling weight deflectometer (FWD) data collected from flexible pavement sections. Some of the pavement sites have been part of the Long-Term Pavement Performance (LTPP) program, and a history of pavement materials testing and FWD deflection data already exist in the LTPP database. Pavement backcalculation tools developed in this study have been utilized to predict the pavement layer moduli and critical pavement responses of flexible pavement layers under typical highway pavement loading conditions. Unlike the linear elastic layered theory commonly used in pavement layer backcalculation, nonlinear subgrade soil response models were used in the ILLI-PAVE finite element program to account for the softening nature of fine-grained subgrade soils and hardening behavior of the unbound base materials under increasing stress states. Preliminary investigations have shown that the ANN-based backcalculation models were capable of rapidly predicting the layer moduli and critical pavement responses with low average errors when compared to the models obtained directly from the finite element analyses. In addition to the analyses of large amounts of FWD data using the backcalculation tools developed in this study, predicted pavement layer moduli values were compared with the traditional backcalculation software solutions; the comparison results are presented in this paper. The advantages of using an ANN-based rapid pavement layer backcalculation tool are also discussed.

Key words: artificial neural networks—falling weight deflectometer—flexible pavements—long-term pavement performance—pavement layer backcalculation

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INTRODUCTION

The falling weight deflectometer (FWD) is a commonly used device for non-destructively assessing the structural properties of flexible pavement systems. Evaluation of FWD test results entails the backcalculation of in situ pavement layer moduli from measured deflections. The use of artificial neural networks (ANNs) to backcalculate the elastic layer moduli and critical pavement responses are investigated using data from FWD test sites, including Iowa LTPP sections.

Elastic layered programs (ELPs) used in asphalt pavement analysis assume linear elasticity. Pavement geomaterials do not, however, follow a linear-type stress-strain behavior under repeated traffic loading. Rather, the nonlinear stress-sensitive response of unbound aggregate materials and fine-grained subgrade soils (herein referred to as geomaterials) has been well established (Brown and Pappin 1981; Thompson and Elliott 1985; Garg et al. 1998). Unbound aggregates exhibit stress hardening or stiffening, whereas fine-grained soils show stress softening type behavior. When these geomaterials are used as pavement layers, the layer stiffnesses (i.e., moduli) are no longer constant, but are functions of the applied stress state. Pavement structural analysis programs that take into account nonlinear geomaterial characterization, such as the ILLI-PAVE finite element program (Raad and Figueroa, 1980), need to be employed to predict pavement response needed more realistically for mechanistic-based pavement design.

Recent research at Iowa State University has focused on the development of artificial neural network (ANN)-based backcalculation flexible pavement analysis models to predict critical pavement responses and layer moduli, respectively. In the field, pavement deflection profiles are obtained from FWD measurements, which require the use of backcalculation structural analysis to determine pavement layer stiffnesses, and as a result estimate pavement remaining life. For this purpose, the ILLI-PAVE finite element program was utilized to generate a solution database for developing ANN-based structural models to predict pavement deflection basins accurately, and determine pavement layer moduli from realistic pavement surface deflection profiles or synthetic FWD data. Such a use of ANN models is described in this paper.

NON-DESTRUCTIVE TESTING WITH THE FALLING WEIGHT DEFLECTOMETER

The FWD, shown in Figure 1, is a non-destructive pavement loading device capable of exerting a load impulse similar in magnitude and duration to moving truck and aircraft wheel loads. The FWD unit can produce loads from 1,500 to 25,000 pounds of force. The load is applied to a loading plate by dropping a weight package on a dampening system, as illustrated in Figure 2. The force applied to the loading plate is measured by a load cell. The resulting pavement deflection is measured by a series of seven seismic deflection sensors positioned along the pavement surface at pre-determined intervals from the loading plate. Signals from the load cell and deflection sensors are fed into the system processor, which selects peak values and transfers this information to an onboard computer. A computerized system in the tow vehicle monitors and controls the testing cycle. A typical test sequence is approximately one minute long, so testing proceeds very rapidly down a street, highway, or airfield. The deflection data, as well as pavement stationing and operator comments, are stored on a floppy disk for analysis after uploading to a personal computer.

Deflection testing has numerous applications for the analysis and design of highway and airfield pavements. FWD data can be used to estimate subgrade and pavement layer elastic moduli values, determine the structural adequacy of a pavement and identify causes of failure, determine uniformity of support along a project and identify weak areas, determine overlay thickness requirements, and develop cost-effective maintenance and rehabilitation alternatives.

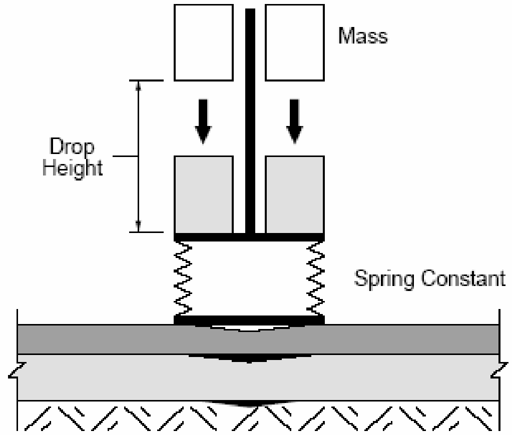


Figure 1. Falling weight deflectometer **Figure 2. Load applied to pavement**

BACKGROUND ON THE LTPP DATABASE

The Long-Term Pavement Performance (LTPP) program is primarily designed to provide state of the art information to the state highway agencies to build and maintain longer lasting pavements (FHWA 1995). The LTPP database is a nationwide effort to collect pavement information over a long period of time. It contains a wide variety of information about pavement materials, climate, traffic, maintenance, field tests, etc. frequently collected for a particular pavement section. Thus, it provides a unique opportunity for pavement researchers to develop modeling tools.

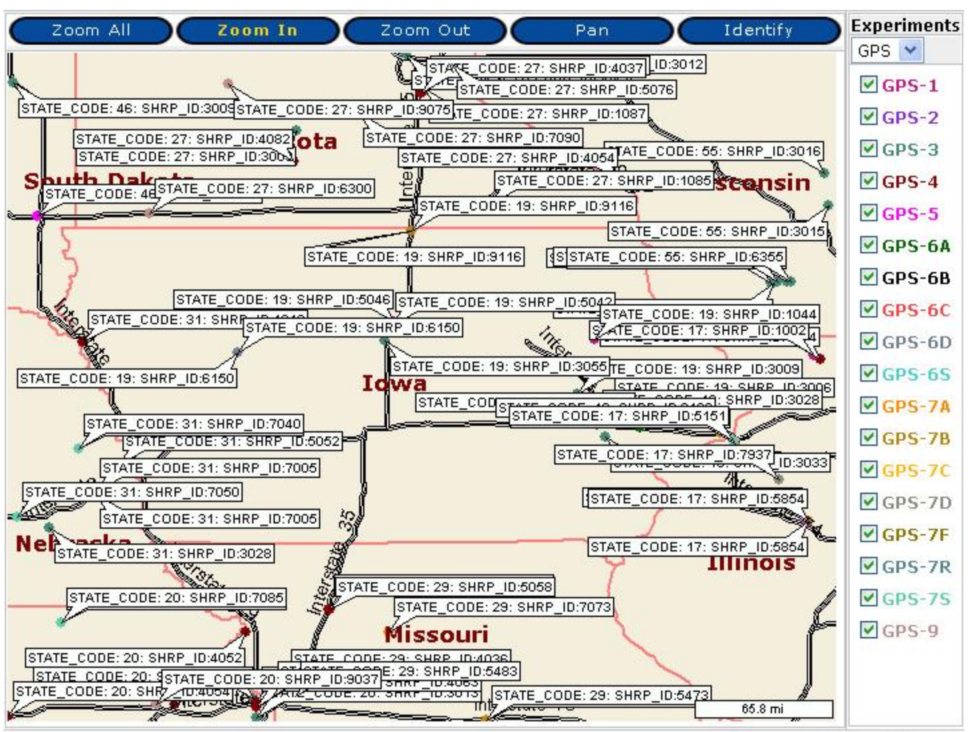


Figure 3. General pavement studies (GPS) sites in Iowa

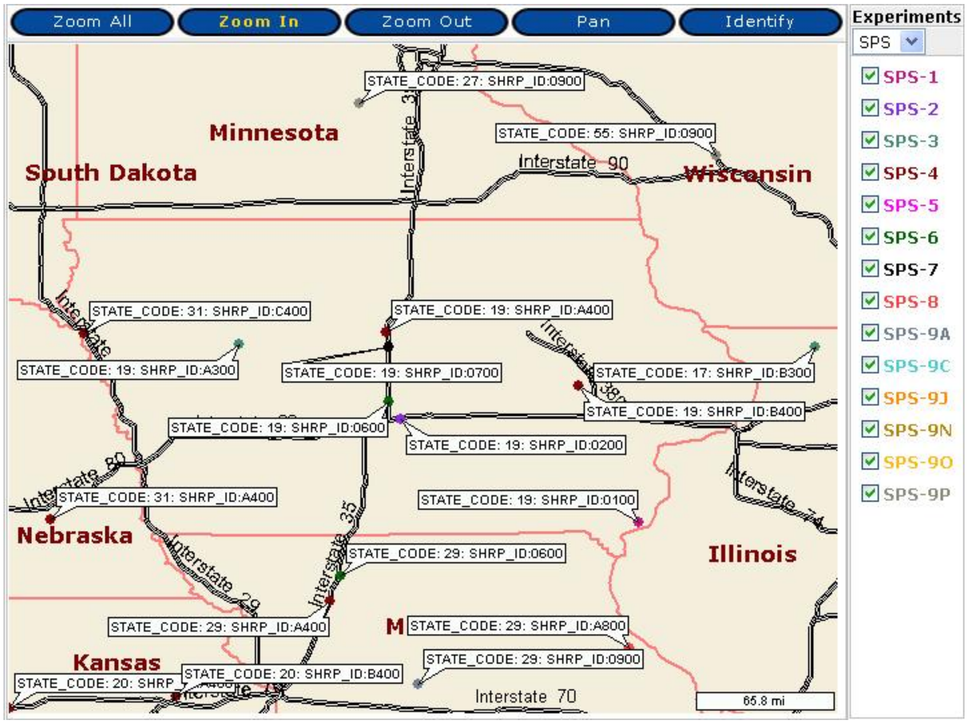


Figure 4. Specific pavement studies (SPS) sites in Iowa

The LTPP program started in 1987 with a comprehensive 20-year study of in-service pavements and a series of rigorous long-term field experiments monitoring more than 2,400 flexible and rigid pavement test sections across the United States and Canada. LTPP database is mainly divided into the major categories of general pavement studies (GPS) and specific pavement studies (SPS). The GPS category consists of nearly 800 in-service pavement test sections throughout the United States and Canada. The SPS category includes intensive studies of specific variables involving new construction, maintenance treatments, and rehabilitation activities (LTPP 2004). Even though it contains a vast amount of information related to pavements, it is organized in a user-friendly format. The GPS and SPS sites in Iowa are shown in Figures 3 and 4, respectively.

OVERVIEW OF ARTIFICIAL NEURAL NETWORKS

Imitating the biological nervous system, artificial neural networks are information processing computational tools capable of solving nonlinear relations in a specific problem. Like humans, they have the flexibility to learn from examples by means of interconnected elements, namely neurons. Neural network architectures, arranged in layers, involve synaptic connections amid neurons that receive signals and transmit them to the others via activation functions. Each connection has its own weight and learning is the process of adjusting the weight between neurons to minimize error between the predicted and expected values. During the learning process, node biases are also adjusted, similar to the connection weights. Since interconnected neurons have the flexibility to adjust the weights, neural networks have powerful capacities for analyzing complex problems. Artificial neural networks, motivated by the neuronal architecture and operation of the brain, contribute to our understanding of several complex, non-linear pavement engineering problems with various pavement and soil variables. In Figure 5, a typical structure of ANNs that consists of a number of neurons that are usually arranged in layers, which are the input layer, hidden layers, and output layers. A comprehensive description of ANNs is beyond the scope of this paper.

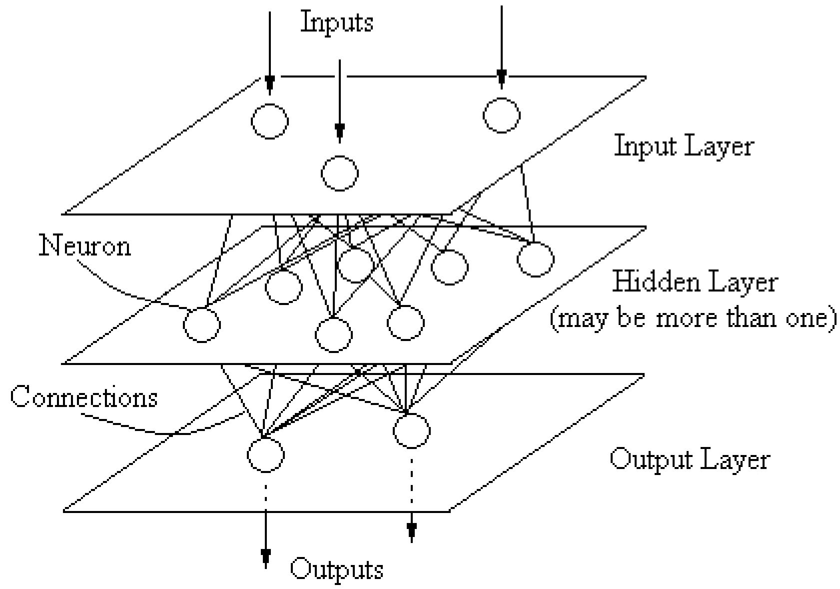


Figure 5. General schematic view of artificial neural networks

NONLINEAR GEOMATERIAL CHARACTERIZATION

Under the repeated application of moving traffic loads, most of the pavement deformations are recoverable and thus considered elastic. It has been customary to use resilient modulus (M_R) for the elastic stiffness of the pavement materials, defined as the repeatedly applied wheel load stress divided by the recoverable strain. Repeated load triaxial tests are commonly employed to evaluate the resilient properties of unbound aggregate materials and cohesive subgrade soils. Therefore, emphasis should be given in structural pavement analysis to realistic nonlinear material modeling in the base/subbase and subgrade layers primarily based on repeated load triaxial test results (AASHTO T307-99, European CEN Std EN 13286-7).

Simple resilient modulus models are often suitable for finite element programming and practical design use, such as in the following equations:

K-θ Model (Hicks and Monismith 1971): $$M_R = K\left(\theta/p_o\right)^n \quad (1)$$

Universal Model (Uzan et al. 1992): $$M_R = K_1\left(\theta/p_o\right)^{K_2}\left(\tau_{oct}/p_o\right)^{K_3} \quad (2)$$

Where $\theta = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_1 + 2\sigma_3$ = bulk stress, τ_{oct} = octahedral shear stress = $\sqrt{2}/3*\sigma_d$ (where $\sigma_d = \sigma_1 - \sigma_3$ = deviator stress in triaxial conditions), p_0 is the unit reference pressure (1 kPa or 1 psi) used in the models to make the stresses non-dimensional, and K, n, and K_1 to K_3 are multiple regression constants obtained from repeated load triaxial test data on granular materials. The simpler K-θ model often adequately captures the overall stress dependency (bulk stress effects) of unbound aggregate behavior under compression-type field loading conditions. The universal model (Uzan et al. 1992) additionally considers the effects of shear stresses and handles very well the modulus increase (unbound aggregates) or decrease (fine-grained soils) with increasing stress states, even for extension field loading conditions.

The resilient modulus of fine-grained subgrade soils is also dependent upon the stress state. Typically, soil modulus decreases in proportion to the increasing stress levels, thus exhibiting stress softening behavior. As a result, the most important parameter affecting the resilient modulus becomes the vertical deviator stress on top of the subgrade due to the applied wheel load. The bilinear or arithmetic model (Thompson and Elliot 1985) is a commonly used resilient modulus model for subgrade soils, expressed by the modulus-deviator stress relationship given in Figure 6. As indicated by Thompson and Elliot (1985), the value of the resilient modulus at the breakpoint in the bilinear curve, E_{Ri}, (see Figure 6) can be used to classify fine-grained soils as being soft, medium, or stiff.

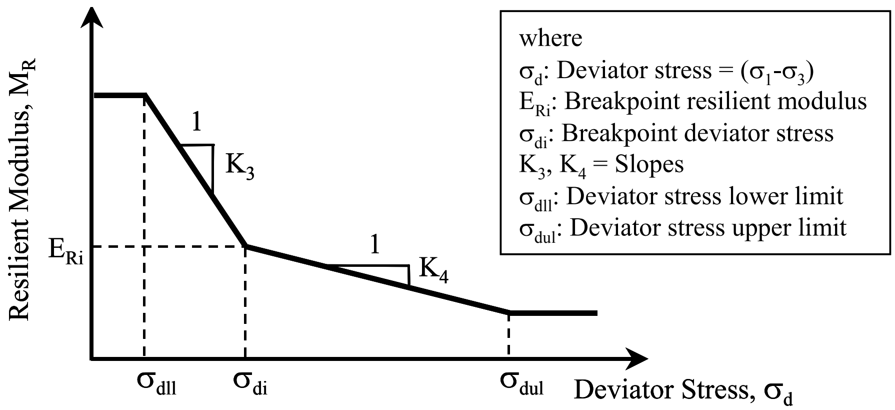


Figure 6. Stress dependency of fine-grained soils characterized by the bilinear model

PAVEMENT ANALYSIS USING THE ILLI-PAVE FINITE ELEMENT PROGRAM

Developed at the University of Illinois (Raad and Figueroa 1980), ILLI-PAVE is an axisymmetric finite element program commonly used in the structural analysis of flexible pavements. The nonlinear, stress-dependent resilient modulus geomaterial models summarized in the previous section are already incorporated into ILLI-PAVE. Numerous studies have validated that the ILLI-PAVE model provides a realistic pavement structural response prediction for highway and airfield pavements (Thompson and Elliot 1985; Thompson 1992; Garg et al. 1998). Recent research at the Federal Aviation Administration's Center of Excellence established at the University of Illinois also supported the development of an updated version of the program, now known as ILLI-PAVE 2000 (Gomez-Ramirez et al. 2002).

The ILLI-PAVE 2000 finite element program was used in this study as the main validated nonlinear structural model for analyzing flexible pavements. The goal was to establish a database of ILLI-PAVE response solutions that would eventually constitute the training and testing data sets for developing ANN-based structural models for rapid backcalculation analyses. For this purpose, a convergence study was performed to determine the domain size extent for the finite element mesh discretization. A radial boundary placed at 25 times the contact area radius was sufficient to obtain convergence of deflections.

The top surface asphalt course was characterized as a linear elastic material with Young's Modulus, E_{AC}, and Poisson's ratio, ν. Due to its simplicity and ease in model parameter evaluation, the K-θ model (Hicks and Monismith 1971) was used as the nonlinear characterization model for the unbound aggregate layer. Based on the work of Rada and Witczak (1981) with a comprehensive granular material database, K and n model parameters can be correlated to characterize the nonlinear stress-dependent behavior with only one model parameter using the following equation (Rada and Witczak, 1981):

$$Log_{10}(K) = 4.657 - 1.807 \cdot n \qquad R^2 = 0.68;\ SEE = 0.22 \qquad (3)$$

Accordingly, good quality granular materials, such as crushed stone, show higher K and lower n values, whereas the opposite applies for lower quality aggregates. Following the study by Rada and Witczak

(1981), the K-values used typically ranged from 20.7 MPa (3 ksi) to 62.0 MPa (9 ksi) and the corresponding n-values were obtained from Equation 3.

Fine-grained soils were considered to be no-friction rather than cohesion-only materials and were modeled using the bilinear or arithmetic model (see Figure 6) for modulus characterization. The breakpoint deviator stress, E_{Ri}, was the main input for subgrade soils. The K_3 and K_4 slopes shown in Figure 6 were taken as constants, 1,100 and 200, respectively, corresponding to medium soils given by Thompson and Elliott (1985). According to a comprehensive Illinois subgrade soil study by Thompson and Robnett (1979), the breakpoint deviator stress, σ_{di}, was taken as 41.4 kPa (6 psi), and 13.8 kPa (2 psi) was used for the lower limit deviator stress, σ_{dll}. The soil's unconfined compressive strength, Q_u, or cohesion was used to determine the upper limit deviator stress, σ_{dul} (see Figure 6), as a function of the breakpoint deviator stress, E_{Ri}, using the following relationship (Thompson and Robnett 1979):

$$\sigma_{dul}(psi) = 2 \times cohesion(psi) = Q_u(psi) = \frac{E_{RI}(ksi) - 0.86}{0.307} \tag{4}$$

Therefore, the asphalt concrete modulus, E_{AC}, granular base K-θ model parameter K, and the subgrade soil break point deviator stress, E_{Ri}, in the bilinear model were used as the layer stiffness inputs for all the different conventional flexible pavement ILLI-PAVE runs. The 40-kN (9-kip) wheel load was applied as a uniform pressure of 552 kPa (80 psi) over a circular area of radius 152 mm (6 in.). The thickness and moduli ranges used are also summarized in Table 1.

Table 1. Pavement geometry and material property/model inputs for ILLI-PAVE solutions

Material type	Layer thickness	Material model	Layer modulus inputs	Poisson's ratio
Asphalt concrete	h_{AC} = 76 to 381 mm (3 to 15 in.)	Linear elastic	E_{AC} = 690 to 13,800 MPa (100 to 2,000 ksi)	$\nu = 0.35$
Unbound aggregate base	h_{GB} = 102 to 559 mm (4 to 22 in.)	Nonlinear K-θ model	$M_R = K\theta^n$ "K" = 20.7 to 62.0 MPa (3 to 9 ksi) "n" from Equation 3	$\nu = 0.35$ for K ≥ 34.5 MPa (5 ksi) $\nu = 0.40$ for K < 34.5 MPa (5 ksi)
Fine-grained subgrade	7,620 mm (300 in.) minus total pavement thickness	Nonlinear bilinear model	$M_R = f(E_{Ri})$; E_{Ri} = 6.9 to 96.5 MPa (1 to 14 ksi)	$\nu = 0.45$

ARTIFICIAL NEURAL NETWORKS AS PAVEMENT ANALYSIS TOOLS

Backpropagation artificial neural network models were trained in this study with the results from the ILLI-PAVE 2000 finite element model and were used as rapid analysis design tools for predicting stresses and strains in flexible pavements. Backpropagation ANNs are very powerful and versatile networks that can be taught to map from one data space to another using a representative set of patterns/examples to be learned. "Backpropagation network" actually refers to a multi-layered, feed-forward neural network trained using an error backpropagation algorithm. The learning process performed by this algorithm is called backpropagation learning, which is mainly an error minimization technique (see Haykin 1999).

ANNs are valuable computational tools that are increasingly being used to solve resource-intensive complex problems as an alternative to using more traditional techniques. Meier et al. (1997) trained backpropagation ANNs as surrogates for ELP analysis in a computer program for backcalculating pavement layer moduli and realized a 42-fold increase in processing speed. In a recent successful application at the University of Illinois, Ceylan (2002) employed ANNs in the analysis of concrete pavement systems and developed ANN-based design tools that incorporated state-of-the-art finite element solutions into routine practical design in ways that were several orders of magnitude faster than the sophisticated finite element programs. Most recently, the project team working on the development of the new mechanistic-based AASHTO pavement design (NCHRP1-37A) in the United States has also recognized ANNs as nontraditional but very powerful computing techniques, and took advantage of ANN models in preparing a mechanistic concrete pavement analysis package.

A total of 24,093 ILLI-PAVE FE runs were conducted by randomly choosing the pavement layer thicknesses and input variables within the given ranges in Table 1 to generate a knowledge database for ANN trainings. The total analysis depth of the pavement system was taken as 7,620 mm (300 in.). The subgrade thicknesses were calculated by subtracting the thicknesses of the asphalt concrete (AC) layer and the base from the total analysis depth. The outputs recorded were the pavement surface deflection basin and the critical pavement responses, the radial strain at the bottom of the AC layer (ε_{AC}), the vertical strain on top of the subgrade (ε_{SG}), and the deviator stress on top of the subgrade layer (σ_D). To maintain a high level of accuracy in the results from all finite element analyses, very similar ILLI-PAVE meshes were employed to have 266 to 494 elements with a total of 20 nodes used in the horizontal direction and 15 to 27 nodes used in the vertical direction. Ceylan (2002) recently highlighted the need to choose such consistent meshes for generating accurate finite element solutions and, as a result, successfully training ANN structural analysis models.

Backpropagation neural networks were used to develop three ANN structural models with different network architectures for predicting the pavement layer moduli (E_{AC}, K_{GB}, and E_{Ri}) and critical pavement responses (ε_{AC}, ε_{SG}, and σ_D) using the FWD deflection data (see Table 2). The FWD surface deflections (D_0, D_8, D_{12}, D_{18}, D_{24}, D_{36}, D_{48}, D_{60}, and D_{72}) are often collected at several different locations, at the drop location (0) and at radial offsets of 203 mm (8 in.), 254 mm (12 in.), 457 mm (18 in.), 610 mm (24 in.), 914 mm (36 in.), 1219 mm (48 in.), 1524 mm (60 in.), and 1829 mm (72 in.). For the modeling work, surface deflections at these FWD sensor radial offsets were obtained from the ILLI-PAVE solutions and used as synthetic data to train ANNs.

Table 2. Conventional flexible pavement ANN backcalculation models

ANN models	Input parameters	Output variables	ANN architecture
BCM-1	h_{AC}, h_{GB}, D_0, D_{12}, D_{24}, D_{36}	E_{AC}, E_{Ri}	6 – 20 – 20 – 2
BCM-2	h_{AC}, h_{GB}, D_0, D_8, D_{12}, D_{24}, D_{36}, D_{48}, D_{60}, D_{72}, E_{AC}, E_{Ri}	K_{GB}	12 – 20 – 20 – 1
BCM-3	h_{AC}, h_{GB}, D_0, D_{12}, D_{24}, D_{36}	ε_{AC}, ε_{SG}, σ_D	6 – 20 – 20 – 3

The first backcalculation model, BCM-1, was designed to predict E_{AC} of the AC layer and the E_{Ri} value of the subgrade using only four pavement surface deflections, D_0, D_{12}, D_{24}, and D_{36}, and two layer thicknesses, h_{AC}, h_{GB}. The ANN BCM-1 model, therefore, had six input parameters and two outputs, E_{AC} and E_{Ri}. A training data file was formed using the 24,093 ILLI-PAVE runs. One thousand of these runs were set aside for use as an independent testing set to conduct proper training and validate the performance of the trained ANN BCM-1 model. A neural network architecture with two hidden layers was exclusively chosen in accordance with the satisfactory results obtained previously with such networks, considering their ability to better facilitate the nonlinear functional mapping (Ceylan 2002).

Several network architectures with two hidden layers were trained. Overall, the training and testing mean squared errors (MSEs) decreased as the networks grew in size with increasing numbers of neurons in the hidden layers. The testing MSEs for the two output variables were, in general, slightly lower than the training ones. The error levels for both the training and testing sets matched closely when the number of hidden nodes approached 20, as in the case of 6-20-20-2 network architecture (6 input, 20 and 20 hidden, and 2 output nodes, respectively).

Figure 7 depicts the prediction ability of the 6-20-20-2 network at the 10,000th training epoch. Average absolute errors (AAEs) were calculated as the sum of the individual absolute errors divided by the 1,000 independent testing patterns. The AAE for the AC layer moduli was a low 1.22%, while the AAE for the subgrade breakpoint moduli E_{Ri} was only 3.27%. As shown in Figure 7, all 1,000 ANN predictions fell on the line of equality for the two-pavement layer moduli, thus indicating proper training and excellent performance of the ANN BCM-1 model.

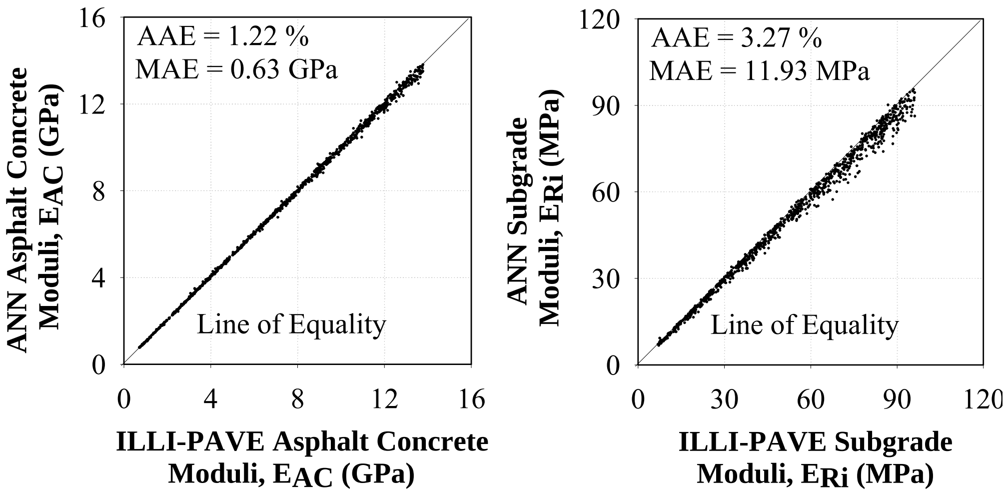


Figure 7. Prediction performance of the 6-20-20-2 BCM-1 model for 10,000 learning cycles

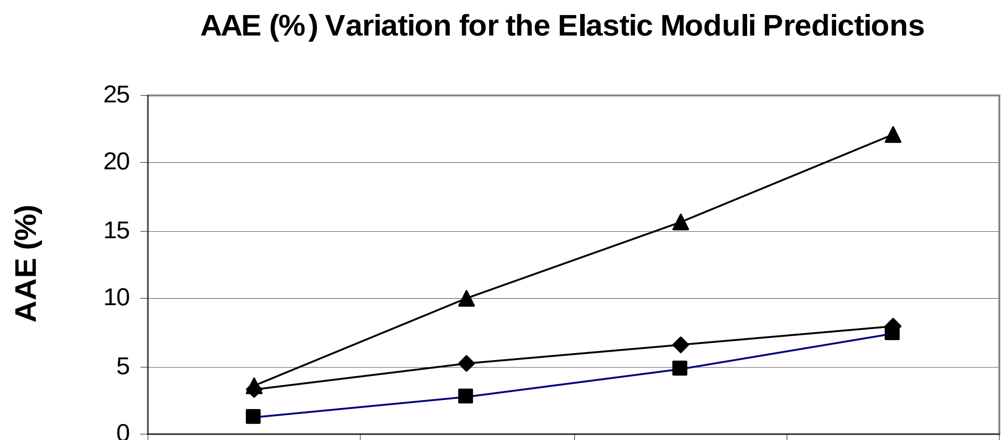
The development of a second backcalculation model, ANN BCM-2, was deemed necessary for accurately predicting the K parameter of the $K\theta^n$ granular base model. The E_{AC} and E_{Ri}, already computed from the ANN BCM-1 model, were used as additional input variables in the BCM-2 model. The BCM-2 network architecture had 12 input variables (h_{AC}, h_{GB}, D_0, D_8, D_{12}, D_{24}, D_{36}, D_{48}, D_{60}, D_{72}, E_{AC}, and E_{Ri}) and a single output, for the K parameter. The trained ANN BCM-2 also had 2 hidden layers with 20 hidden nodes in each layer, and successfully predicted the K values with a low AAE value of 3.53% after 10,000 learning cycles.

Next, using ILLI-PAVE solutions, a third backcalculation model, ANN BCM-3, was developed with the intention of directly predicting the critical pavement responses, ε_{AC}, ε_{SG}, and σ_D, from the deflection basins. This approach eliminates the need for first predicting the pavement layer moduli and then using a forward calculation structural analysis model to compute the critical pavement responses. The directness of this approach can save time and effort in analyzing the structural adequacy of field pavement sections from FWD data. Once validated with field data, the ANN model can predict ε_{AC} for AC fatigue condition evaluation in the field.

The ANN BCM-3 network architecture had 6 input variables (inputs similar to those of the ANN BCM-1 model), 2 hidden layers with 20 hidden nodes in each layer, and 3 critical pavement responses, ε_{AC}, ε_{SG}, and σ_D, in the output layer. The AAE values from the ANN BCM-3 predictions were 0.46% and 2.03% for the asphalt radial strains (ε_{AC}) and the vertical compressive subgrade strains (ε_{SG}), respectively. The AAE value for the predicted subgrade deviator stresses (σ_D) was 1.36%. Such low errors indicate the proper training and excellent prediction performance of the ANN BCM-3 backcalculation model trained for 10,000 learning cycles.

To develop more robust networks that can tolerate the noisy or inaccurate pavement deflection patterns collected from FWD field tests, several network architectures were trained with varying levels of noise in them. Applied noise levels in deflection basins and pavement layer thicknesses ranged from ± 2% to ± 10% to train the robust ANN models that can account for the variations in deflection measurements and pavement layer thicknesses due to poor construction practices. The AAE comparisons for the virgin data and noise-introduced data are given below.

AAE variations for the asphalt layer moduli predictions were investigated. The minimum AAE was obtained in the ANN training that used virgin deflection data. As can be seen from Figure 8, when the noise level introduced into the deflection data was increased, the percent AAE value also increased, as expected. After noise was introduced to the deflection data, the highest AAE value increase was found in the K_{GB} predictions. AAE variations for predicting the critical pavement responses were also investigated. Similar trends in AAE increase in the asphalt layer moduli predictions were observed in predicting the critical pavement responses. The percent AAE value was increased again when more noise was introduced into the deflection data (see Figure 9).



	1	2	3	4
EAC	1.22	2.77	4.85	7.41
ERI	3.27	5.19	6.57	8.03
KGB	3.53	10.04	15.62	22.12
	Virgin Def.	±2% Noisy Def.	±5% Noisy Def.	±10% Noisy Def.

Figure 8. AAE variations for predicting the asphalt layer moduli for different noise levels

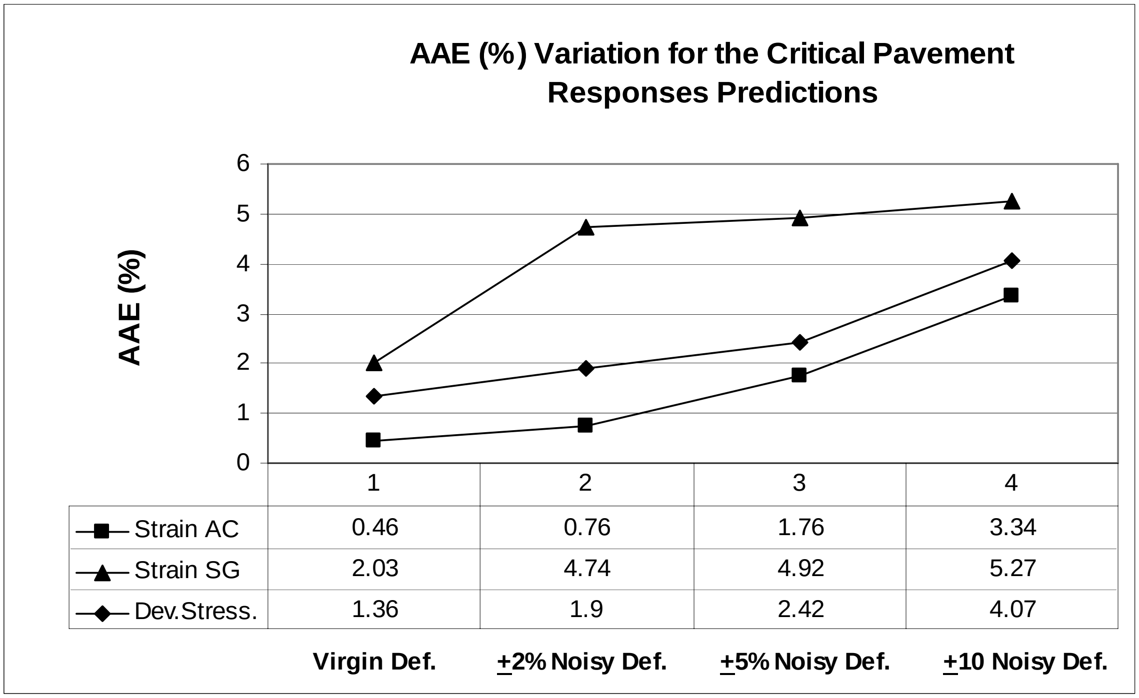


Figure 9. AAE variations for predicting critical pavement responses for different noise levels

PERFORMANCE OF ANN PAVEMENT BACKCALCULATION MODELS

Six conventional flexible pavement sections were selected to further evaluate the performances of the ANN backcalculation models, BCM-1, BCM-2, and BCM-3. All pavement sections had a 102-mm (4-in.) AC underlain by a 305-mm (12-in.) granular base layer and applied with the 40-kN (9-kip) wheel load and a 552-kPa (80-psi) uniform tire pressure. The AC layer moduli were kept constant at 1,379 MPa (200 ksi) with a constant Poisson's ratio of 0.35.

The pavements were first analyzed with the ILLI-PAVE finite element program. Two aggregate base K values of 27.6 and 62.1 MPa (4 and 9 ksi) and three subgrade E_{Ri} values of 20.7, 41.4, and 62.1 MPa (3, 6, and 9 ksi) were considered for a total factorial of 6 pavement sections analyzed. The rest of the nonlinear model parameters in the base and subgrade layers were assigned in accordance with the properties shown in Table 3. The pavement surface deflections obtained by ILLI-PAVE at 0, 203, 305, 610, 914, 1,219, 1,524, and 1,829 mm (0, 8, 12, 24, 36, 48, 60, and 72 in.) were treated as field-measured FWD deflections and used for validating the ANN backcalculation model performances. These deflections were also used in an ELP-based backcalculation program, BAKFAA, developed by the Federal Aviation Administration, to backcalculate the pavement layer moduli (http://www.airtech.tc.faa.gov/naptf/download/).

The top section of Table 3 presents the inputs and output results of the ILLI-PAVE analyses. Given next in Table 3 are the ANN backcalculation model predictions for the AC layer modulus, E_{AC}, nonlinear parameters, K and E_{Ri}, and the critical pavement responses, ε_{AC}, ε_{SG}, and σ_D. Note that all ANN models produced very close results to the ILLI-PAVE input values and critical pavement responses. The AAE values were 2.1% for the six E_{AC} predictions and 1.7% for the six E_{Ri} predictions from the ANN BCM-1 model, 3.4% for the six K predictions from the ANN BCM-2 model, and 0.5%, 1.5%, and 1.3% for each of the six predicted critical pavement responses for ε_{AC}, ε_{SG}, and σ_D, respectively, from the ANN BCM-3 model. Clearly, the developed ANN models were quite successful in accurately mapping the nonlinear analysis ability of the ILLI-PAVE finite element program into their connection weights and node biases. The AAE values for the predicted critical pavement responses were even lower than the ones obtained for the backcalculated layer properties, which suggests that it would be feasible to estimate, for example, pavement fatigue life directly from the field-measured FWD deflection data.

Table 3. Summary analysis results for the six pavements with unbound aggregate bases

	Pavement section					
ILLI-PAVE calculations	**1**	**2**	**3**	**4**	**5**	**6**
E_{AC} (MPa) – input	1,379	1,379	1,379	1,379	1,379	1,379
K (MPa) – input	27.6	27.6	27.6	62.1	62.1	62.1
E_{Ri} (MPa) – input	20.7	41.4	62.1	20.7	41.4	62.1
ε_{AC} ($\mu\varepsilon$) – output (tension "+")	426	411	400	355	346	340
ε_{SG} ($\mu\varepsilon$) – output (compression "–")	-1,058	-925	-792	-929	-798	-698
σ_D (kPa) – output (compression "–")	-38	-42	-49	-34	-39	-43
ANN backcalculation model predictions	**1**	**2**	**3**	**4**	**5**	**6**
E_{AC} (MPa) – (ANN BCM-1)	1,333	1,328	1,373	1,387	1,428	1,389
K (MPa) – (ANN BCM-2)	28.1	28.6	27.3	64.5	65.2	64.9
E_{Ri} (MPa) – (ANN BCM-1)	20.7	40.6	61.7	21.1	42.8	63.7
ε_{AC} ($\mu\varepsilon$) – (ANN BCM-3)	425	408	400	352	344	339
ε_{SG} ($\mu\varepsilon$) – (ANN BCM-3)	-1,009	-923	-763	-925	-797	-697
σ_D (kPa) – (ANN BCM-3)	-38	-40	-48	-34	-39	-43
BAKFAA ELP-based results	**1**	**2**	**3**	**4**	**5**	**6**
E_{AC} (MPa) – backcalc. output	1,498	1,579	1,497	1,553	1,551	1,555
E_{GB} (MPa) – backcalc. output	129.3	127.9	148.0	158.2	167.5	174.6
E_{SG} (MPa) – backcalc. output	60.4	86.7	103.7	66.9	91.5	115.7
ε_{AC} ($\mu\varepsilon$) – forward calc. output	520	505	473	444	425	411
ε_{SG} ($\mu\varepsilon$) – forward calc. output	-861	-688	-608	-748	-614	-525
σ_D (kPa) – forward calc. output	-52	-59	-63	-50	-56	-60
BAKFAA forward calculations with avg. nonlinear layer moduli	**1**	**2**	**3**	**4**	**5**	**6**
E_{AC} (MPa) – avg. from ILLI-PAVE	1,379	1,379	1,379	1,379	1,379	1,379
E_{GB} (MPa) – avg. from ILLI-PAVE	193.4	201.0	207.5	223.4	229.2	233.7
E_{SG} (MPa) – avg. from ILLI-PAVE	40.4	57.7	76.6	42.1	59.3	77.2
ε_{AC} ($\mu\varepsilon$) – forward calc. output	420	407	396	366	357	351
ε_{SG} ($\mu\varepsilon$) – forward calc. output	-935	-788	-688	-857	-728	-635
σ_D (kPa) – forward calc. output	-38	-45	-52	-36	-43	-49

The ELP-based BAKFAA program was used next to backcalculate the average elastic layer moduli from pavement surface deflections. Table 3 also summarizes the BAKFAA backcalculation and forward calculation results. The AAE values were rather high: 12%, 170%, and 78% for each of the six predicted moduli for E_{AC}, E_{GB}, and E_{SG}, respectively. These layer moduli were then used as inputs for the forward calculation option of the BAKFAA program to compute critical pavement responses. The AAE values were 22%, 23%, and 39% for the average ε_{AC}, ε_{SG}, and σ_D values, respectively, when compared to the actual ILLI-PAVE pavement responses.

Also presented in the bottom section of Table 3 are the results of additional BAKFAA forward calculation analyses. For all six pavement sections, the subgrade and base layer moduli, E_{SG} and E_{GB}, were computed by averaging the actual ILLI-PAVE computed nonlinear (pavement centerline) modulus distributions with depth in the subgrade and base layers. These averaged moduli values, tabulated in the bottom section of Table 3, were then used as inputs in the forward calculation BAKFAA analyses to predict the critical pavement responses. In this case, the AAE values were calculated as 2%, 11%, and 8% for the average ε_{AC}, ε_{SG}, and σ_D values, respectively, much lower than the previous AAE values of 22%, 23%, and 39% from the ELP-based backcalculation. Again, the nonlinear pavement geomaterial behavior must be properly accounted for to backcalculate layer moduli and predict critical pavement responses accurately.

SUMMARY AND CONCLUSIONS

ANN-based pavement backcalculation tools for analyzing the FWD data collected from flexible pavement sections have been developed in this study. Some of the pavement sites have been part of the LTPP program, and a history of pavement materials testing and FWD deflection data already exist in the LTPP database. Three ANN backcalculation models were developed using approximately 24,000 nonlinear ILLI-PAVE finite element solutions. Unlike the linear elastic layered theory commonly used in pavement layer backcalculation, realistic nonlinear unbound aggregate base (UAB) and subgrade soil modulus models were used in the ILLI-PAVE program to account for the typical stiffening behavior of UABs and the fine-grained subgrade soil moduli decreasing with increasing stress states. The ANN models developed successfully predicted the layer moduli and critical pavement responses computed by the ILLI-PAVE finite element solutions and were superior to the linear elastic layered backcalculation analyses due to the nonlinear material characterization employed. Also, in order to develop more robust networks that can tolerate the noisy or inaccurate pavement deflection patterns collected from FWD field tests, several network architectures were trained with varying levels of noise; the noise-introduced ANN models successfully predicted the pavement layer moduli and critical pavement responses. Such ANN structural analysis models can provide pavement engineers and designers with sophisticated finite element solutions, without the need for a high degree of expertise in the input and output of the problem, to rapidly analyze the large number of pavement deflection basins needed for routine pavement evaluation.

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Automated Performance Measurement of Winter Road Maintenance Results

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ABSTRACT

Snow Belt state and local transportation agencies have been raising the winter maintenance service level for a number of years. Also, motorists have developed high expectations for unimpeded travel fairly soon after a winter storm. A 2002 survey by the Iowa DOT of licensed Iowa drivers found that more than 44% of respondents expected interstate highways to be completely clear within six hours after a winter storm. There are several methods to determine the length of time to achieve a clear roadway. Employee observations are routinely used for determining road conditions. This paper explores the feasibility of using existing automated systems to provide objective, quantifiable data to measure driver perceptions of winter road conditions in comparison to physical measurements of pavement surface conditions. Automatic traffic recorders (ATRs) provide vehicle speed data at 70 locations on the Iowa primary highway system. There are 52 Road Weather Information System (RWIS) locations on the same system.

Hourly changes in average speed at ATR sites before, during, and after a winter storm indicate motorist perceptions of winter driving conditions. RWIS data include changes in road surface status, temperature, chemical factors, and precipitation. In places where ATR and RWIS sites are in reasonable proximity on the highway, the two data sets can be compared to determine the ways in which motorist perceptions align with physical road surface measurements concerning the time that must elapse after a storm for the road to return to the motorists' expectations of a completely clear highway.

Speed and road surface data for major winter storms at 15 rural interstate highway locations have been analyzed to demonstrate the data's utility for measuring performance from the customer's perspective.

Note: This research was still in progress at the time of publication; contact the author above for more information.

Key words: performance measurements—winter road maintenance

Ultra High Performance Concrete Highway Bridge

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ABSTRACT

Wapello County and the Iowa Department of Transportation were granted funding through the TEA-21 Innovative Bridge Construction Program to demonstrate the use of ultra high performance concrete (UHPC) in a bridge replacement project. The UHPC in the prestressed concrete beams is expected to achieve a 28-day compressive strength of up to 30,000 psi. The use of this innovative product in a three-beam cross section is intended to take advantage of the superior strength and to optimize design. The beams will be pretensioned using 0.6-inch diameter strands and without mild reinforcing steel, except to provide composite action with the cast-in-place deck.

In Phase I of the multi-phase project, a 71-foot–long test beam will be tested to verify shear and flexural capacities, along with shear testing of smaller beams. If testing efforts are successful, Phase II will include the casting of 111-foot–long beams, followed by the construction of the single span bridge in the spring/summer of 2005. After construction, a monitoring program will be implemented to document the performance of this innovative product.

A discussion of the design efforts and the current progress of this research project is the focus of this paper.

Key words: bulb-tee—ductal concrete—reactive powder concrete—steel fibers—ultra high performance concrete

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

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 and low permeability can be achieved, depending on the curing process. To improve ductility, steel or fiberglass fibers (approximately 2% by volume) are added, replacing the mild reinforcing steel. For this project, a patented mix (Ductal) developed by LaFarge North America has been used.

Research is currently being conducted at Ohio University, Michigan Technological University, Iowa State University, and Virginia Polytechnic Institute and State University to help better understand UHPC properties. Testing is under way at the Turner-Fairbanks Laboratory near Washington, DC on a prototype prestressed, pretensioned section. In addition, a TEA-21 Innovative Bridge Construction Program (IBRC) project, using UHPC in prestressed beams for a highway bridge, is underway for the Virginia Department of Transportation.

PROJECT BACKGROUND

In 2003, Wapello County, Iowa and the Iowa Department of Transportation were granted funding through the IBRC for a project utilizing UHPC. UHPC will be used in pretensioned, prestressed concrete beams in a bridge replacement project in southern Wapello County (see Figures 1, 2, and 3).

The beams will be pretensioned using 0.6-inch diameter low relaxation strands. No mild reinforcing steel, except an amount to provide composite action between the beam and cast-in-place deck, will be used. To verify shear and flexural capacity of the beam, 10-inch and 12-inch shear beams and a 71-foot–long test beam have been cast. Testing is currently underway at Iowa State University and the Center for Transportation Research and Education (CTRE) in Ames, Iowa. Currently, the service capacity under flexure has been verified by testing, and casting of the 111-feet production beams has been scheduled for June 26, 2005. Contract letting for a separate project for the bridge construction is scheduled for June 20, 2005.

BRIDGE DESCRIPTION

The replacement bridge for Wapello County will be a 110-foot simple span bridge with a three-beam cross section. The abutments will be integral and an 8-inch cast-in-place deck will be used. Beam spacing will be 9 feet 7 inches with 4-foot overhangs. See Figure 4 for additional details.

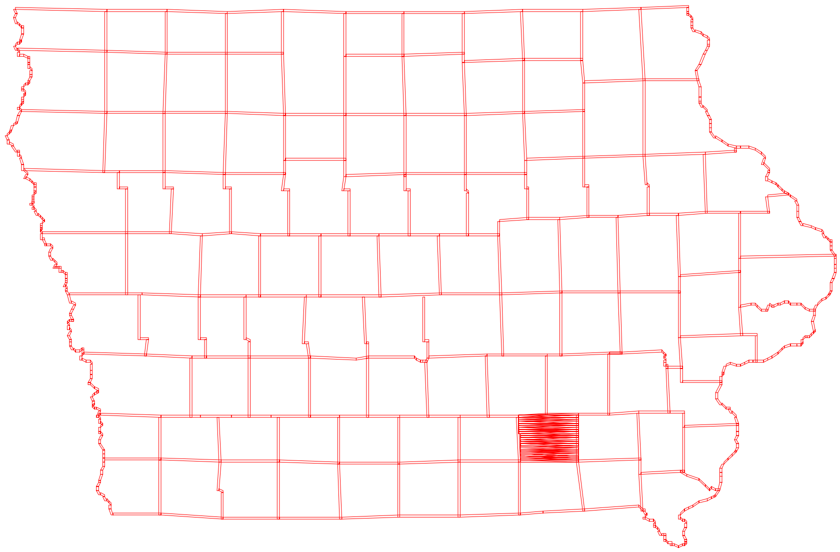

Figure 1. Wapello County, Iowa

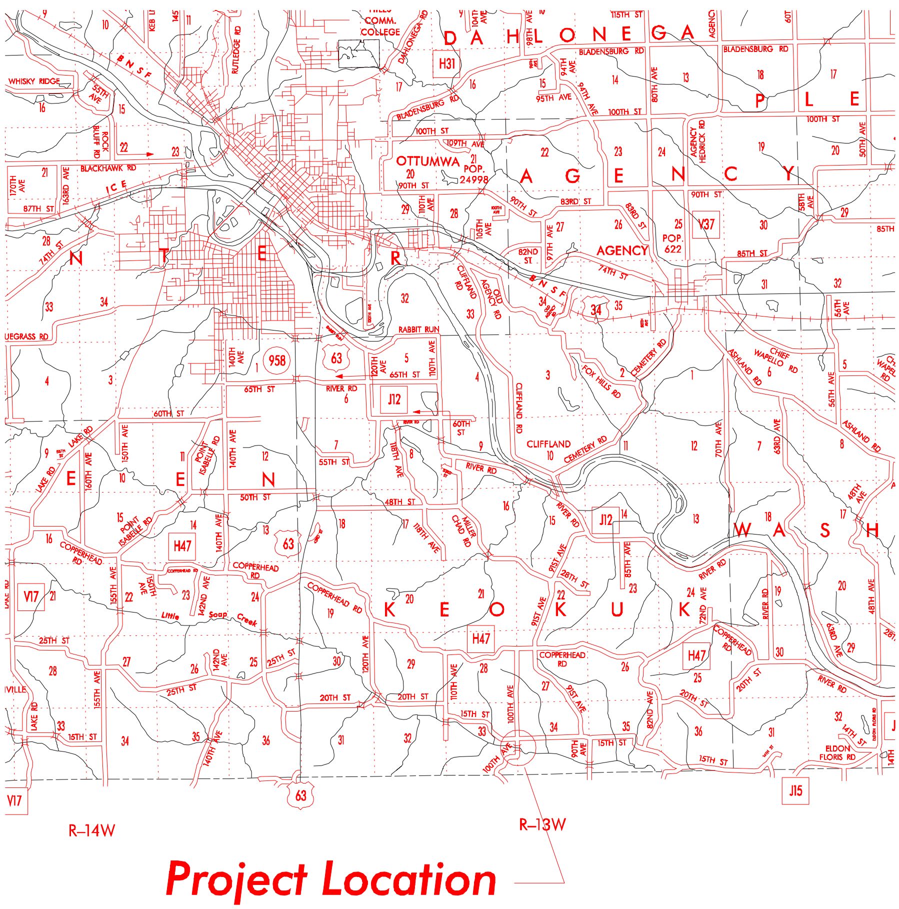

Figure 2. Project location in Wapello County, Iowa

Figure 3. Bridge site

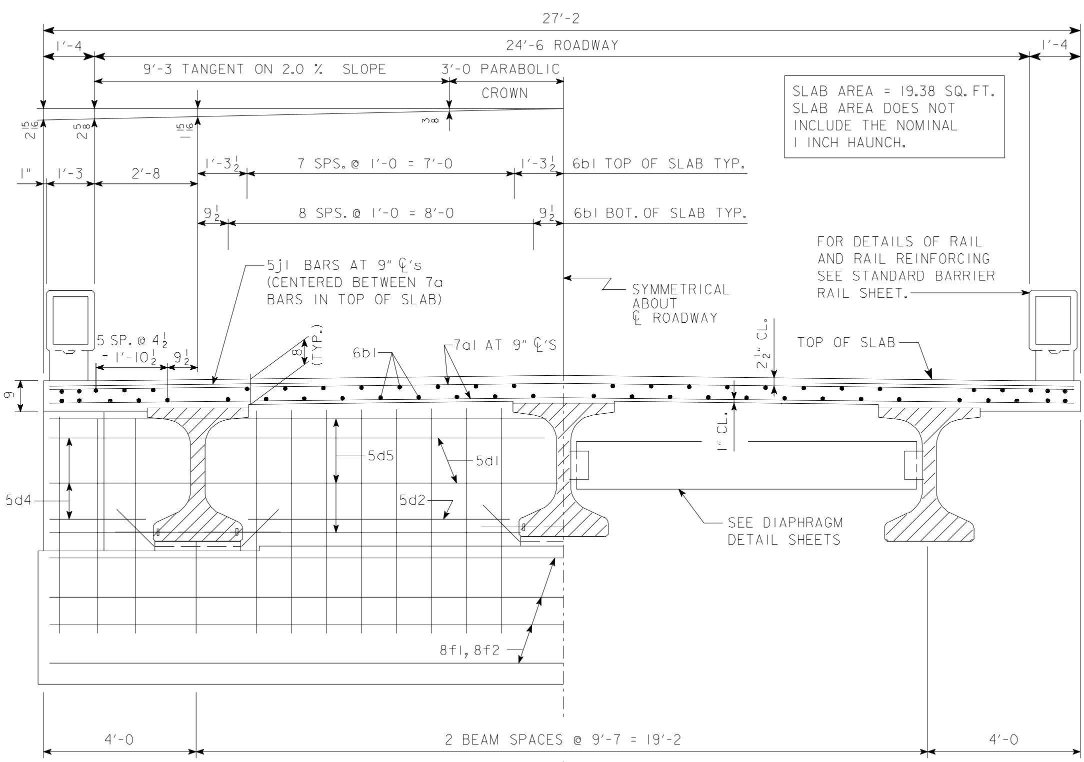

Figure 4. Proposed bridge cross section

STAGES OF PROJECT

Because of the uniqueness of UHPC and the special requirements for designing, mixing, casting, and curing, this project was organized into the stages listed below to gain experience and confidence for all parties. Listed below are stages, completion dates, and the current status of each stage.

1. Ultra-High Performance Concrete Design Seminar (completed 8-12-03)
2. Test batch at Iowa DOT Materials Laboratory in Ames (completed 12-11-03)
3. Review of precasting plants (completed 12-11-03)
4. Additional test batch at Materials Laboratory in Ames (completed 1-26-04)
5. Test batch at precasting plants (completed 4-12-04)
6. Casting of shear beam specimens (completed 1-24-05)
7. Casting of 71-ft test beam (completed 2-23-05)
8. Flexure testing of 71-foot test beam (completed 5-12-05)
9. Shear testing of 71-foot test beam (scheduled 6-9-05)
10. Shear testing of shear beam specimens (pending)
11. Casting of three 111-foot production beams (scheduled to begin 6-26-05)
12. Construction of replacement structure (contract letting scheduled for 6-20-05)
13. Two-year evaluation of finished bridge after construction

PRELIMINARY WORK

Design Seminar

On August 12, 2003, the Iowa DOT and CTRE organized a seminar on ultra high performance concrete to provide information to people who would be involved in the project. The seminar was sponsored by the Federal Highway Administration (FHWA) and attended by individuals from the FHWA, the state of Iowa, the precast industry, and academia. Speakers and topics are listed below:

1. Joey Hartmann, P.E., Turner-Fairbanks Highway Research Center, FHWA
   (Research Program)
2. Eugene Chuang, Ph.D., P.E., Garg Consulting Services, Inc.; formerly from MIT
   (Design Issues and Section Optimization)
3. Ben Graybeal, PSI, Inc.
   (Material Testing)
4. Chris Hill, Prestress Services of Kentucky
5. (Design Issues and Precasting)
6. Vic Perry, P. Eng., LaFarge North America
7. (Material Overview and Precasting Issues)
8. Brent Phares, Ph.D., CTRE
   (Overview of IBRC Project)

Test Batch at Materials Laboratory in Ames, Iowa

On December 11, 2003, a test mix was produced at the Iowa DOT Materials Laboratory in Ames, Iowa. Personnel from the precasting industry, Iowa DOT, Iowa State University, and CTRE attended. LaFarge provided the test mix and Gavin Geist from LaFarge demonstrated the mixing procedure (see Table 1 for mix proportions). For the demonstration, a 1958 Lancaster mixer with a two-cubic ft capacity was used to produce a one-cubic ft batch (see Figure 5). Three-inch by six-inch test cylinders were cast along with four-inch by four-inch by 18-inch beams. Specimens were cast on a vibrating table using a small plastic tremie tube. Curing of the specimens took place in sealed metal containers placed in ovens at 140° F for 72 hours. Results of the test cylinder compressive strengths are shown in Table 2.

Table 1. Test Mix Proportions

Description	Quantity
Ductal mix	137 lbs
Water	8.03 lbs
3000 NS (super plasticzer)	850 g
Steel fibers	9.7 lbs

Figure 5. Mixing of UHPC

Table 2. Compressive strengths of 12-11-03 mix

Cylinder	Compressive strength (psi)
1	15,896
2	16,123
3	20,004
4	15,943

Lower than expected compressive strengths (30,000 psi was expected) were found when the cylinders were tested. The following reasons may have contributed to the reduced strengths:

1. Steam curing was started 24 hours after casting and before initial set had taken place. Without accelerators, initial set can take up to 40 hours.
2. There was difficulty in achieving plane ends of test cylinders for uniform compressive loading. The ends of the cylinders were trimmed with a concrete saw to provide square ends.
3. Visual inspection of a cylinder that was cut lengthwise showed higher than expected air voids.

Because of these problems, and to gain more experience working with the mix, the Iowa DOT Materials Lab produced a second test batch on January 26, 2004. Three-inch by six-inch test cylinders and two-inch cubes were prepared. Casting of the two-inch cubes provided a test specimen with plane sides that did not require end preparation. Specimens were cured in sealed steel containers in ovens at 195° F with 95% humidity for 40 hours (see Tables 3 and 5) and in water (see Table 4). Compressive strengths of the cylinders improved, but were still lower than expected for the cylinders. Difficulty in achieving plane surfaces for uniform compression loading was believed to be the main cause of the lower strength values.

Table 3. Specimens at 95% humidity

two-inch cubes	Compressive strength (psi)
1	29,930
2	27,540
3	30,610

Table 4. Water-cured specimens

Two-inch cubes	Compressive strength (psi)
1	31,210
2	30,750
3	27,640

Table 5. Specimens cured at 95% humidity

Three-inch x six-inch cylinders	Compressive strength (psi)
1	23,820
2	24,570
3	22,510

Certification of Local Precasting Plants

Two local precasting plants expressed an interest in casting the beams for the project and were certified by LaFarge to mix and cast the Ductal mix. Inspections of the plants were made as part of a certification process and test batches were performed at each plant.

Concerns expressed by precasters on the use of the Ductal mix are listed:

1. High cost for patented Ductal mix
2. Longer time needed to batch the mix (possibly 15 to 30 minutes per batch) and additional cleaning time for mixers because of the steel fibers and fine aggregate
3. Increased chances of damaging mixing equipment, due to the high mixing energy required
4. Proper placement in forms and the requirement to produce the complete concrete quantity before placement can be started
5. Shrinkage values estimated to be twice the amount normally expected from standard mixes, because of the large amount of cement in the mix. Modifications of forms may be required to compensate for the additional shrinkage. Larger shrinkage will require properly timed release of the strands and removal of forms as well.
6. Long setting and curing time (40 hours set time/48 hours of 195° F steam cure) and the lost production time in the casting beds
7. Lack of testing equipment required to do the following:
   - Evaluate the UHPC mix
   - Prepare the three-inch by six-inch test cylinders
   - Compressive testing of the two-inch cubes

Plant Selection

Bids were received from the local precastors for the casting of the 71-foot test beam, three 111-foot production beams, and additional smaller beams for shear test. The bids submitted by local precastors were higher than expected, due to the concerns listed above and limited experience with UHPC mixes. The precaster selected was LaFarge Canada, Inc., of the Greater Winnipeg Precast Division, Winnipeg, Canada.

Beam Design and Plan Preparation

CTRE, Wapello County, and the Iowa DOT Office of Bridges and Structures jointly designed the test beam, production beams, and plans for the bridge. A modified Iowa 45-inch bulb tee was used. To save material in the beam section, the web width was reduced by two inches, top flange by one inch, and the bottom flange by two inches (see Figures 6 and 7). Because of the work on UHPC been done by Franz-Josef Ulm of the Massachusetts Institute of Technology, he provided a final review of the beam design.

The design of the beam was a challenge for the staff involved because of lack of approved specifications. Design guidelines have been developed by France, and design recommendations were available from reports that have been done. However, there are no specifications currently available in the United States. A review of the service and ultimate strength checks recommended by the French design guide and the research model developed by Dr. Ulm were used as a guide for design. The following additional design data were also used:

1. Release compressive strength: 14,500 psi
2. Release modulus of elasticity: 5,800 psi
3. Final design compressive strength: 24,000 psi
4. Final modulus of elasticity: 8,000 psi
5. Allowable tension stress at service: 600 psi
6. Allowable compression stress at service: 14400 psi
7. LRFD HL-93 loading
8. Grillage analysis for distribution factors

The final beam design section used 49 0.6-inch strands stressed to 72.6% of ultimate. To reduce end-beam stresses, five strands were draped along with debonding (see Figure 8 for strand layout). The 71-foot test beam used an identical strand layout to verify release stresses.

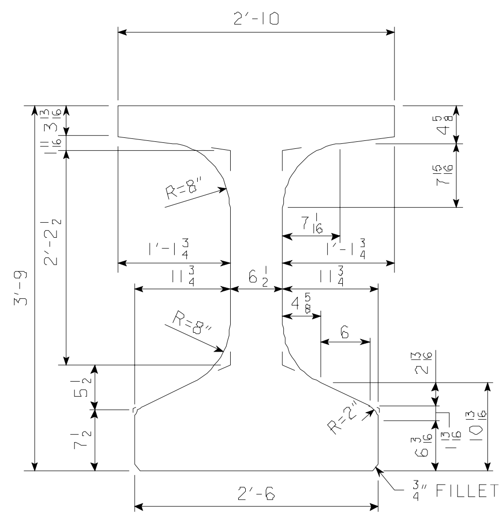

Figure 6. Iowa 45-inch bulb tee section

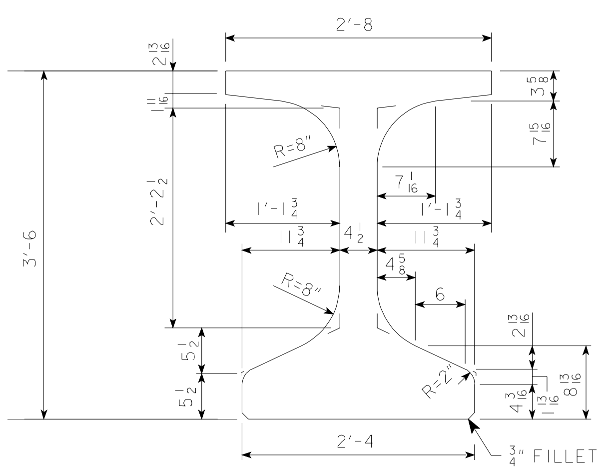

Figure 7. Modified section for UHPC

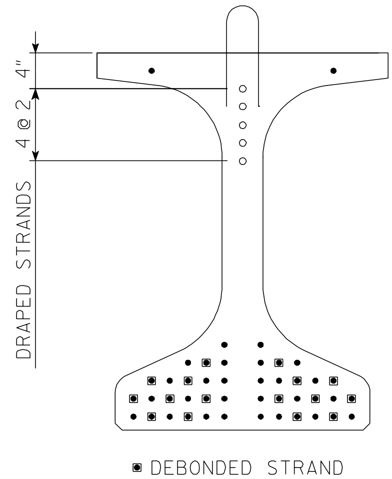

Figure 8. Strand layout

OVERVIEW OF TEST BEAM

Casting and Release of Strands

Composite Connection between the Beam and Cast-in-Place Deck

The test beam was cast with three options for developing the composite connection between the beam and deck (see Figure 9). These options were studied due to the requirement that the top of the beam be covered with plastic immediately after placement of the concrete to prevent shrinkage cracks and the need for the plastic be placed directly on the concrete. Based on discussions after the casting, the use of the mild steel U-bar option was selected. The selection was based on the simplified detail and ease of installation during casting of the test beam.

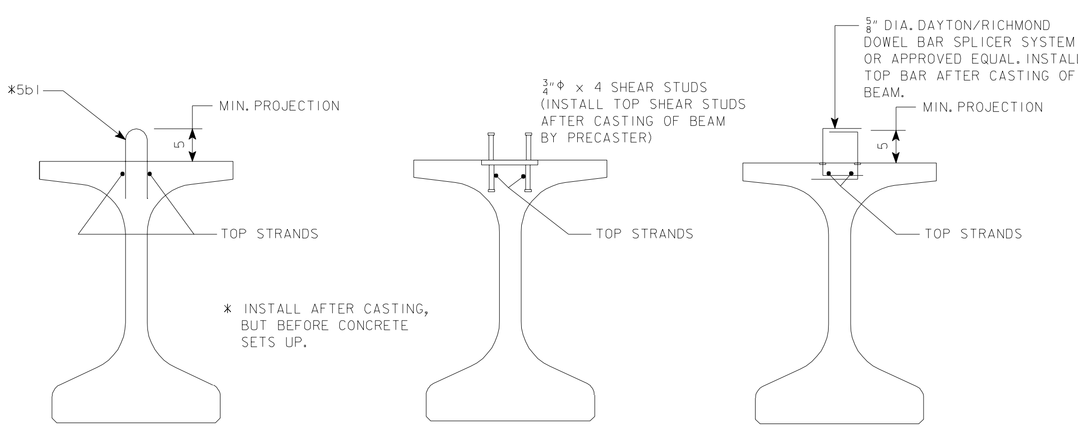

Figure 9. Composite connections

Strand Anchorage and Transfer of Prestressing Force

Research completed at Ohio University, "Bond Performance Between Ultra-High Performance Concrete and Prestressing Strands," showed improved bond strength using UHPC. Because of the improved bond and transfer, there was concern that the reduced transfer lengths (possibly less than 12 inches) may cause a concentration of release forces at the interface between the bottom flange and web. To reduce these forces, both debonding and draping of the strands were provided (see Figure 8 for details). Under inspection, no visible cracks were found at the interface after release of the strands for the test beam.

Short-Term and Long-Term Losses

To attempt to measure losses in the beam, fiber optic strain gauges were attached to the bottom row of strands on the test beam before casting. Based on the changes in strain measured at release and the final strains after curing, the release losses and total losses at midspan were calculated. Final losses were calculated to be approximately 27% higher than those estimated in design.

Camber and Growth

Release camber for the 71-foot test beam was 1-3/8 inches. After curing, the measured camber was 3-1/8 inches. No additional growth was noted in the Iowa State University lab after shipment.

Release and Final Compressive Strengths (Percent Difference)

Beam strands were released at initial concrete strengths of 14,500 psi. Final compressive strengths from three-inch x six-inch test cylinders varied from 20,400 psi to 33,700 psi, with an average of 28,976 psi. Lower values of compressive strengths may have been due to poor end-cylinder preparation.

Flexure Test

The initial flexure test was limited to just over the concrete cracking load. There was concern that flexure testing to failure might adversely effect the ultimate shear test at the beam ends. The test was performed on May 12, 2005 in the structures lab at Town Engineering, Iowa State University. Four jacks were placed symmetrically at midspan, spaced 2.6 feet and 4.5 feet from the centerline of the span. See Figure 10. The estimated cracking load for the beam was between 240 kips and 280 kips, based on the loss estimates. Actual cracking was noted at 64 kips per jack or 256 kips total. See Figure 11. The maximum load applied was 264 kips with 3¼ inches of deflection. See Figure 12 for a load displacement diagram.

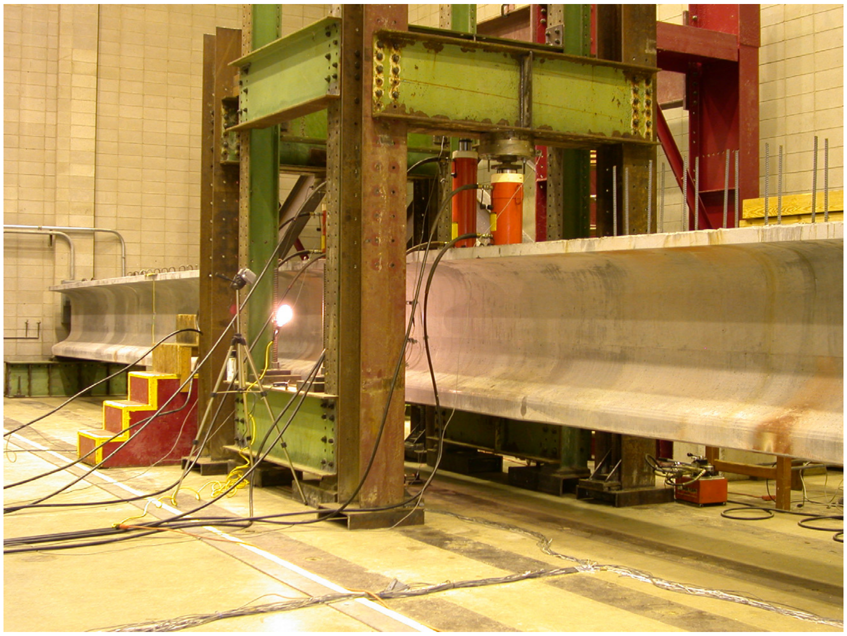

Figure 10. Flexure test

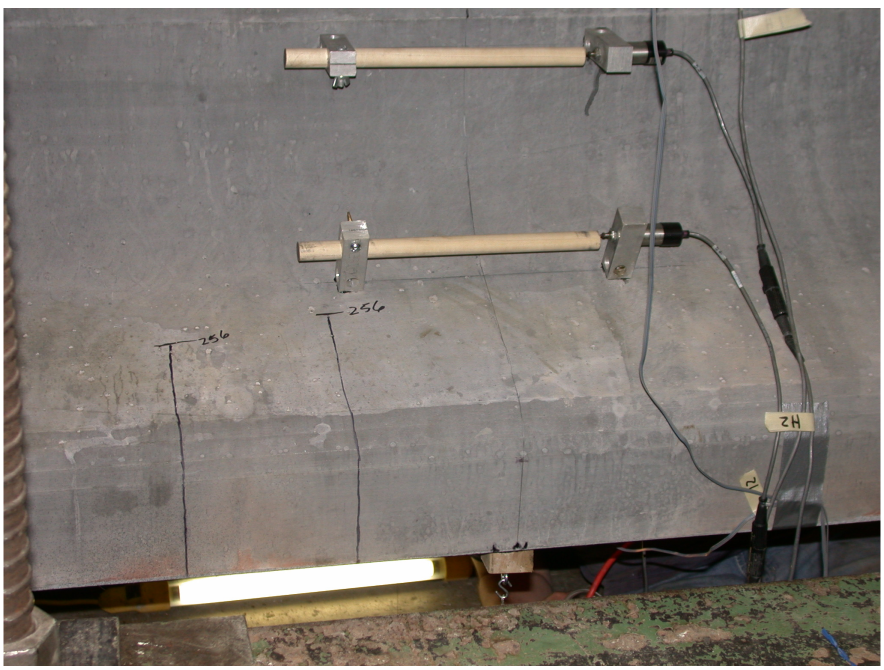

Figure 11. Measured flexure cracking at midspan

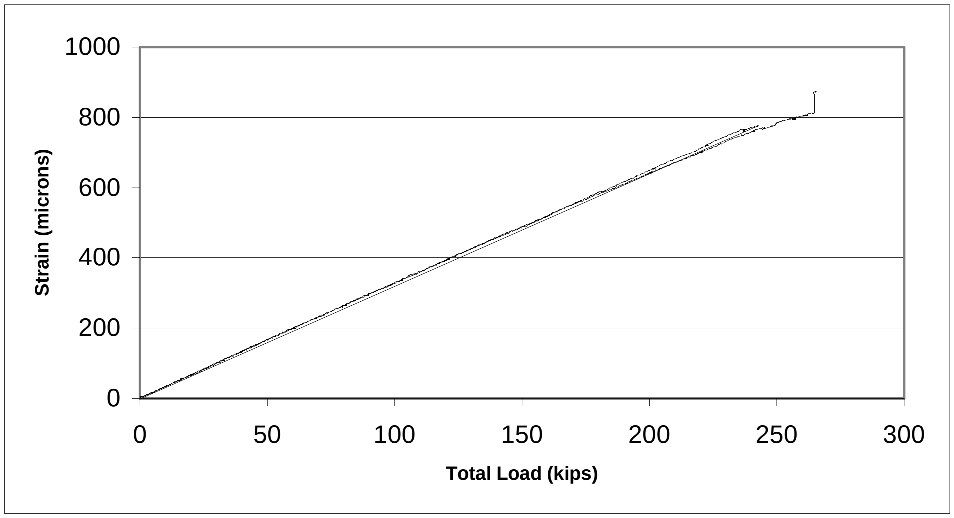

Figure 12. Midspan fiber optic gauge

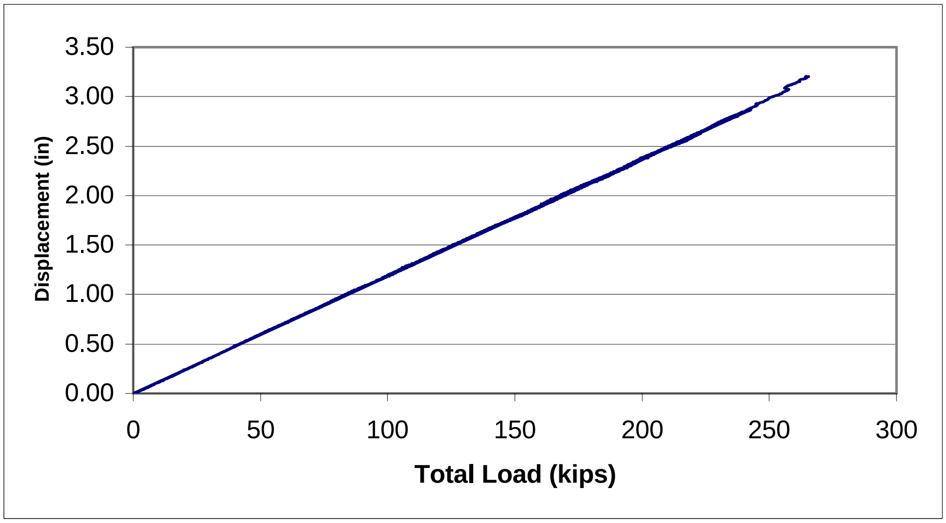

Figure 13. Midspan deflection gauge

Shear Test

At the writing of this report, shear testing had not taken place, but was scheduled for June 9, 2005. An estimated failure load of 750 kips has been calculated.

To help develop a better understanding of the shear capcacity of the UHCP mix, additional shear testing was included as part of the research. Shear tests will be performed on a series of smaller beam shapes (10-inch deep by 54-inch long and 12-inch deep by 64-inch long) with web widths from 1½ to 2 inches. See Figures 14 and 15 for the dimensions and a photo of the test specimens.

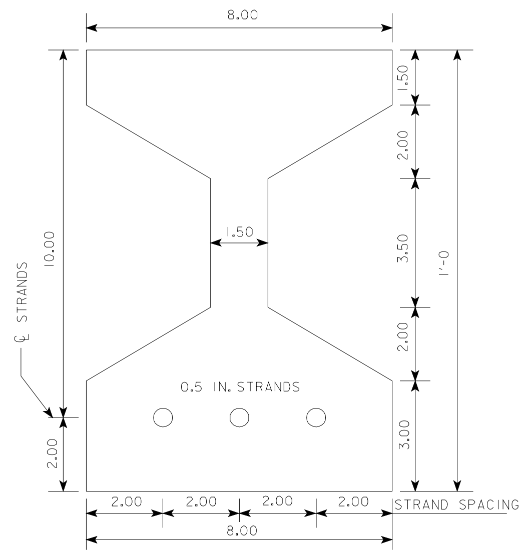

Figure 14. Twelve-inch shear beam cross section

Figure 15. Ten-inch and 12-inch shear beams at Iowa State University

CONCLUSIONS

This IBRC project has given Wapello County, the Iowa DOT, CTRE, and Iowa State University the opportunity to gain valuable experience designing, testing, mixing, and casting ultra high performance concrete. Additional research in the future should address current design and production concerns, and develop more efficient beam designs to maximize the unique structural properties of UHPC. Additional research is already underway to use UHPC in precast replacement paving notches as part of rapid approach slab replacement work repair.

Training Opportunities in Transportation Asset Management

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Many states and municipalities are currently implementing transportation asset management techniques and strategies. To adopt the institutional change required for such a systemic overhaul of transportation agency business, agencies must participate in training opportunities. While many of these training opportunities, offered by various providers, currently use the phrase transportation asset management as the catch, most do not conform to the AASHTO-led efforts. Since many public agencies now seek training in transportation asset management, it is imperative to address the quality and substance of the training programs. Some states and localities create their own training materials, some rely on vendors, and others rely on public providers such as Local Technical Assistance Program (LTAP) centers. The Midwest Regional University Transportation Center at the University of Wisconsin-Madison is compiling a comprehensive inventory of available training. Since asset management-termed training covers a wide range of topics, short abstracts and analyses will be written for each training course identified. Courses will be categorized by major type. A primary focus will be whether a training course is primarily concerned with broadly defined asset management, pavement or data management, or some other topic. Since many individual components of transportation asset management exist, courses must be catalogued based on their relative strengths and specialties.

This project utilizes an external advisory group to comment on and react to the materials collected. In addition, since no registry exists of asset management trainers, the project relies heavily on people close to asset management activities to suggest agencies, institutions, and vendors to be included in the survey. A likely outcome of the effort is an identifiable list of trainers. The survey will provide data on target audiences, focused strategies, tools and techniques, and related content. The end review, evaluation, categorization, and abstracting will benefit the transportation community. This paper will attempt to make sense of what exists, identify glaring deficiencies in courses, and objectively analyze current programs.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: asset management training—training program inventory—transportation asset management

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

Improving Multi-Use Recreational Trail Safety through a Coordinated 911 Sign Project

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ABSTRACT

The objective of the Cedar Valley Trails 911 Signs Project (911 Signs) is the design and implementation of a comprehensive method to georeference trail locations for emergency response and asset management purposes. The Cedar Valley Trail System and the Cedar Valley Nature Trail encompass 95 miles of paved trail within Black Hawk County Iowa. This recreational trail system serves over 200,000 trail users annually. Using GIS software and local GIS data, a new map grid system was devised to communicate a location to within one-tenth of one square mile. This scheme of two numbers, one letter, and two numbers (e.g., 22 C 99) provides a short identification number that is meaningful on both computer and printed maps. A GPS trail survey was conducted by Iowa Northland Regional Council of Governments (November 2004) to collect locations, attributes, and photographs of benches, signs, and other trailside features. A new 911 sign, with "911" clearly visible, was designed and approved. The new 911 signs, with the specific ID number for location reference, are attached to the georeferenced trail features. The 911 sign location data is integrated into the Black Hawk Consolidated Public Safety Dispatch Center's system to enable dispatchers to “see” the location on the Dispatch Center's GIS computer map. Thus, the 911 Signs Project provides a practical solution to location communication in emergency situations and serves as an asset inventory of all features along this transportation corridor.

Key words: alternate transportation—recreation trail—safety

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

PROBLEM STATEMENT

The metropolitan Cedar Valley Trail System and the rural Cedar Valley Nature Trail (collectively referred to as the Trail) encompass nearly 95 miles of paved trail within Black Hawk County (BHC), Iowa. This recreational trail system serves over 200,000 trail users annually. Trail use includes bicycle, in-line skating, and pedestrian traffic; charity walks and rides; Trail festivals; and other events. As Trail usage increased and the Trail length was extended over time, it became apparent that an improved means of communicating trail location information was needed for both public safety and asset management purposes. Since Trail expansion plans are in place, any location communication improvements must fit both the current Trail and future extensions. An added benefit is an inventory of all assets along this transportation corridor.

Public Safety

The original Trail location project, known as the Trail Emergency Access System (TEAS), was developed in 1999. The TEAS was designed to assist emergency responders in locating Trail users in need of help. The TEAS included 130 signs that were placed along the then-existing Trail. The signs, made of a material that degrades in the sunlight, were not permanently fixed in the ground. Some have been destroyed, others moved, and some are hidden in trail-side vegetation (see Figure 1).

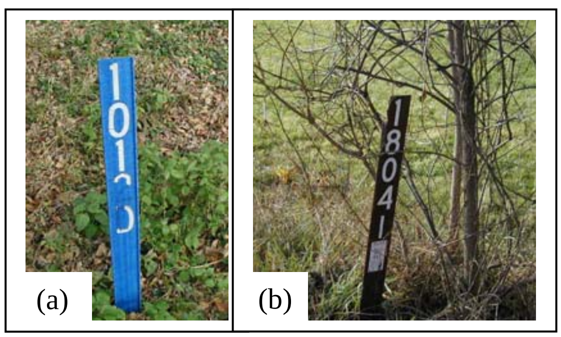

Figure 1. Two of the 79 remaining TEAS signs. (a) deterioration of the decals; (b) a sign that will be hidden when the shrub leafs out

Of the original signs, only 79 remain today. Decals on each sign provide a unique location ID code. The purpose of the signs, however, is not clearly indicated. The TEAS code is composed of numbers and a letter. The first two to three numbers were derived from the east-west street address range, with the following two to three numbers coming from the north-south street address range. The east-west street address ranges start at 00 in the center of BHC and again in the center of each city, as do the north-south street address ranges. The address range numbers increase to the east and west and to the north and south of the city or county center. Thus, the number on a TEAS sign gives no apparent clue as to location. The letter designates a particular jurisdiction or area within a jurisdiction. TEAS signs along the Trail in southeastern BHC have as many as seven characters. For bicyclists and other trail users, a seven-character sign that may be partially hidden in grass is difficult to read, let alone to recall if the sign is no longer in view. With the TEAS, all of the numbers and the letter need to be communicated to describe even the general area of the location. At the Black Hawk Consolidated Public Safety Dispatch Center, the TEAS sign locations are related to the street or the Trail access point nearest to the sign, rather than to the sign location on the Trail. The post-1999 trail segments lack TEAS signs. Thus, the TEAS is no longer adequate to meet present public safety needs related to location communication and emergency dispatch.

Asset Management

Today there are over 1700 trailside features, including signs, benches, and shelters. The Trail crosses through many jurisdictions (Cedar Falls, Evansdale, Hudson, Waterloo, BHC, and the State of Iowa). The Iowa Northland Regional Council of Governments (INRCOG) coordinates transportation planning, grant writing, and Trail mapping. Thus, rather than each jurisdiction devising an approach to Trail asset data collection, a common method useful to all the jurisdictions to provide uniformity is needed. The resulting method must facilitate annual asset condition reporting and maintenance, be useful for Trail planning, and accommodate future addition of new assets.

PROJECT OBJECTIVES

The main objective of the Cedar Valley Trails-911 Signs Project (911 Signs) is the design and implementation of an up-to-date, comprehensive method to georeference trail locations. Specific objectives are to do the following:

- Develop a location identification system that is consistent across all jurisdictions in the county
- Use a location identification code that is meaningful to emergency responders even if only part of the identification code is communicated
- Design signs that are easy to see, read, remember, and maintain
- Use GPS for initial and future Trail-related field data collection and for future asset condition assessment
- Use GIS software for data management, sharing, viewing, and updating
- Install 911 signs on existing permanent trail-side features so that there is at least one 911 sign every one-quarter mile along the entire length of the Trail in BHC
- Incorporate the 911 sign location data into the local computer-aided dispatch system
- Replicate the 911 Signs Project in other counties that have multi-use recreation trails

The 911 Signs Project is the first such project implemented in Iowa.

PROJECT DEVELOPMENT

The 911 Signs Project is a cooperative project involving multiple agencies. Meetings began in August 2004 to discuss needs and initiate the design of the 911 Signs Project. County and city parks departments, the State of Iowa Department of Natural Resources, city engineers, the BHC Emergency Management Agency, public safety offices, the Black Hawk Consolidated Public Safety Communications Center, INRCOG, and BHC Information Technology were represented at this and subsequent meetings.

911 Sign Location Identification Code

Representatives agreed that using a number-letter-number format would be easier to remember that an entirely numeric sequence and that the code should be as short as possible. A new 911 sign location identification code with a maximum of five alpha-numeric characters was chosen from among a variety of possibilities presented to the group. Using GIS software and local GIS data layers as a base, a new map grid system was devised. The location code is based on a new map grid system that breaks BHC into one-square-mile blocks, assigning numerals 1–25 to the X-axis (west-east) and letters A–X to the Y-axis (north-south) of the grid (see Figure 2). Thus, the X value and the Y value alone designate a particular square mile (see Figure 3). Each one-square-mile block is further broken into 100 blocks one-tenth of a square mile. The one-tenth square mile blocks are sequentially numbered 00–99 from left to right (i.e., 00–09, 10–19, 20–29, etc.). The final two digits identify a location to within one-tenth of a square mile.

This scheme of two numbers, one letter, and two numbers (e.g., 22 C 99) provides a short identification code that is meaningful on both computers and printed maps.

X value = one or two number(s)
Y value = letter,
1/10-mile grid value = two numbers
For example: 24 C 99
X value = 24
Y value = C
1/10 mile gird value = 99

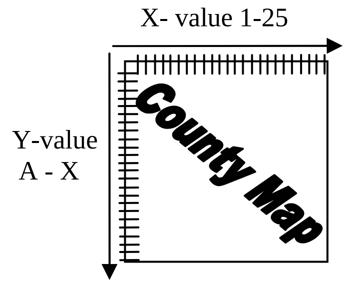

Figure 2. Overview of identification code

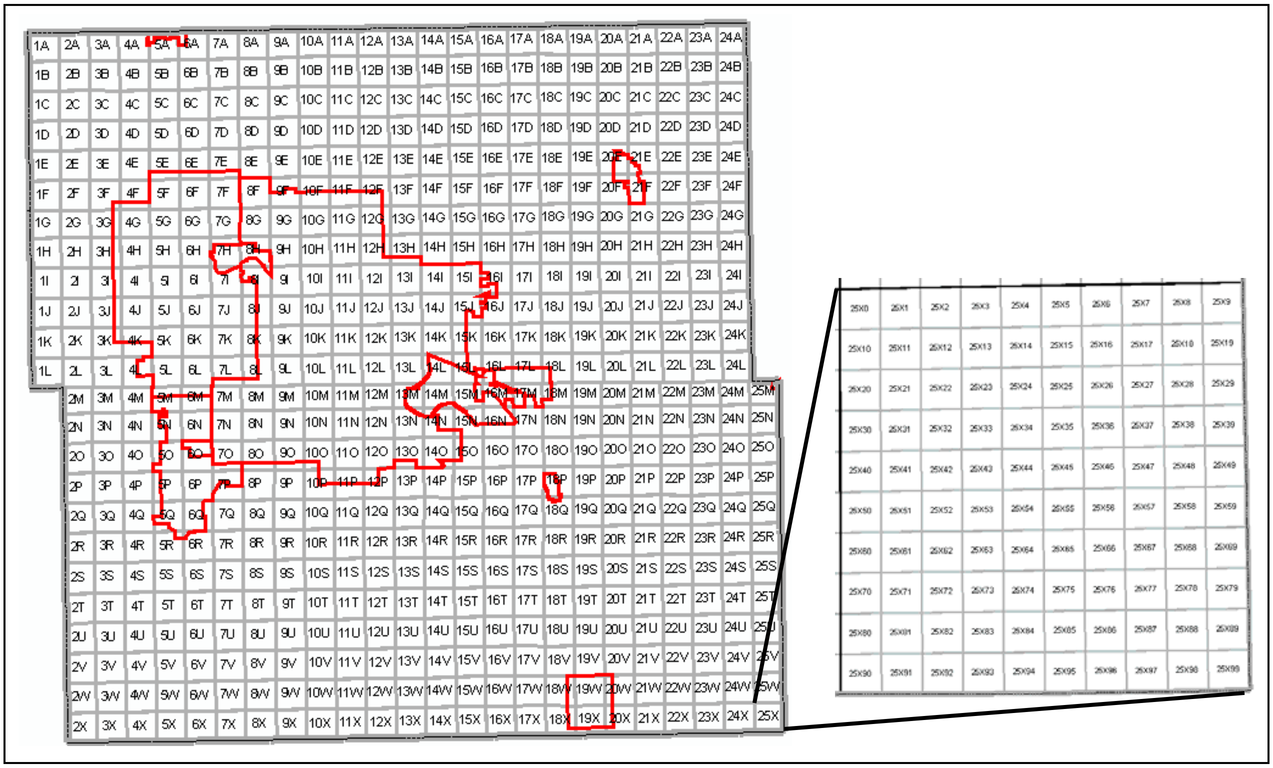

Figure 3. One-mile grid with code, marked county boundary and corporate boundaries, and an expanded (25X) area to illustrate the 1/10 mile grid

Sign Design

Following approval of the new identification code and location grid system, a new 911 sign with “911” clearly visible was designed and approved (see Figure 4). The signs are made of metal. Identification code decals are attached to the sign. The feature that a sign is attached to dictates sign shape and orientation (horizontal, vertical, or square). The horizontal and vertical signs measure 18 inches by 4 inches. The square sign measures 12 inches by 12 inches.

Figure 4. 911 sign design

Trail Survey

INRCOG staff conducted a GPS trail survey to capture location and attribute information for each trail feature, including benches, shelters, drinking fountains, bridges, and transportation signs. This was done in late fall in leaf-off conditions. A Trimble GeoXT GPS receiver was used for field data collection. The data dictionary included an auto-filled date and time entry. Field data was differentially corrected post-process to improve spatial accuracy, using Pathfinder Office software and the files downloaded from the North Liberty CORS GPS base station. The differentially corrected trail feature files were exported as shapefiles for use in ESRI© ArcGIS9 (ArcView) software. Using ArcGIS 9, the trail feature shapefiles were overlaid on BHC's high-resolution orthophotography to compare spatial accuracy. Many of the trail features are more recent than the orthophotography, but those that did correspond to visible features in the orthophotography were on the feature or within five feet of it. This level of accuracy is suitable for the purposes of this project. A photograph of each feature was also taken for quality control and asset management purposes. Each photograph was time-stamped.

911 Signs Project Maps and Attributes in GIS

A GIS project was set up for asset mapping and attribute management purposes (see Figure 5). The Trail-related data sets (shapefiles) were symbolized. Layer (.lyr) files were created to preserve the symbology for ease of use in other GIS projects. A hyperlink attribute field was included in each of the point data sets so that the photograph of a particular feature could be linked to the point representing the feature. The time stamps of the photo and the date/time from the GPS data dictionary were used to ensure that each photograph is properly assigned to its corresponding feature (see Figure 6). The photographs served as a quality control and quality assurance measure initially to verify that feature attributes were correct. The photographs also provided a baseline against which future feature conditions and 911 sign condition will be gauged. Mapping the features provides a visual overview of the location of 249 benches. The attributes reveal the material the bench is composed of, the memorial name on the bench, the side of the trail the bench is on, and the sign identification code that would apply to the bench.

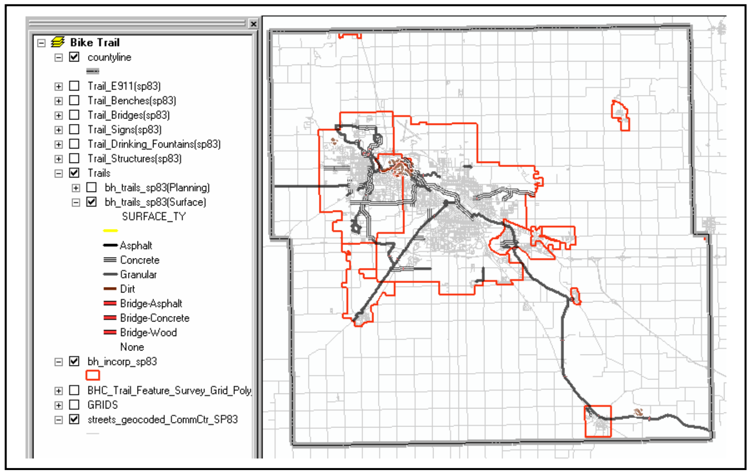

Figure 5. Trail Survey 2004 Project in ArcMap, illustrating Trail location and Trail surface type (1:300,000 scale)

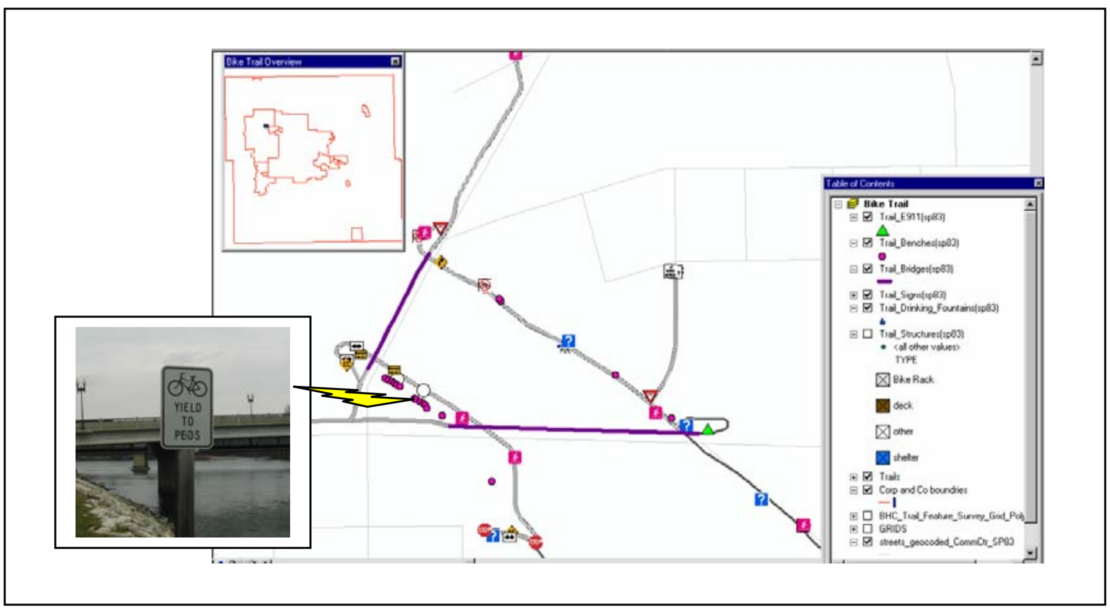

Figure 6. Trail Survey 2004 Project in ArcMap, illustrating variety of Trail features and a photograph linked to a particular feature (1:3,500 scale)

Other Trail assets include bridges, structures, drinking fountains, and existing TEAS signs. There are 1,363 trail signs providing information to Trail users. These include the typical caution, warning, yield, stop, and bike trail signs, as well as signs designating Trail names, funding sources and partnerships, other named places, boundaries lines, information marquees, local area maps, and directions. Table 1 illustrates the wide variety of signs along the Trail. These signs are valuable assets. Knowing Trail sign location, type, and jurisdiction aids in accounting for and tracking these government assets. The photograph of each asset also ensures that missing signs are replaced with the correct sign type.

Table 1. Partial list of sign types along the trail

All unauthorized vehicles and horses prohibited	House address markers	Roadside prairie restoration area
Begin one way	Lawcon funding sign	Rough trail
Bike symbol	Memorial marker	RR X-ing
Bike X-ing	Mile marker	Sharp left curve
Bikeway narrows	Names of trail segments	Sharp right curve
Bird house	Names places (cities, parks, wildlife area, lakes, etc.)	Side road - left
Bird observation blind / no bicycles	Narrow bridge	Side road - right
Bird sanctuary	No bicycles	Sidewalk users walk bikes
Bulletin board	No drugs zone increased penalties	Slip when wet
Campground	No dumping	Slow
Campground direction	No dumping $500 Fine	Soft trail
Caution	No horses on recreation trail	Steep grade
City map	No hunting	Stop
Cross country skiing	No hunting wildlife refuge	Stop ahead
Curve 90	No minibikes motorcycles snowmobiles	Street or road name
Curve left	No motor vehicle	T intersection
Curve right	No parking beyond here	Trail begins
Curve S	No snowmobiling	Trail courtesy
Cyclist warn pedestrians when passing	No swimming	Trail end
Danger water	No target shooting	Trail ends 200 feet
Dead end	Park closing time	Tree plantings
Directional sign	Park or preserve boundaries	Two way traffic
Dismount walk	Park preserve - no dumping	Unauthorized vehicles and horses prohibited on dike
Do not enter	Park rules	Use ped signal
End one way	Ped X-ing	Warning orange diamond
Entering <named place>	Please place trash in receptacles	Warning walk bike across cattle guards
Exit only / directional sign	Prairie trail	Watchable wildlife
Funded by REAP	Public hunting area	Wildlife refuge
Hiking trail	Rec trail	Wrong way
Hiking trail / cross country skiing	Residential address sign	Y intersection
Hiking trail / no snowmobile	Restroom direction	Yield
Hiking, no smoking, dogs on a leash	Riverview recreation area/ rules and regulations	Yield to pedestrians
Hospital direction	Road closed	

When the 911 Signs GIS project was completed, the locations of the various trailside features were assessed in the GIS mapping to determine the features to which to attach the new 911 signs. Benches were the primary targets for the new signs. In a few situations, the 911 sign was attached to a vertical sign post or to a structure. To meet the criteria of having a 911 sign every one-quarter mile, new permanent posts were installed in a few locations to support the 911 sign. Once the final determination was made, a new data set consisting of 267 points illustrating features with new 911 signs was derived (see Figure 7). This data set is used in the dispatch center's mapping system.

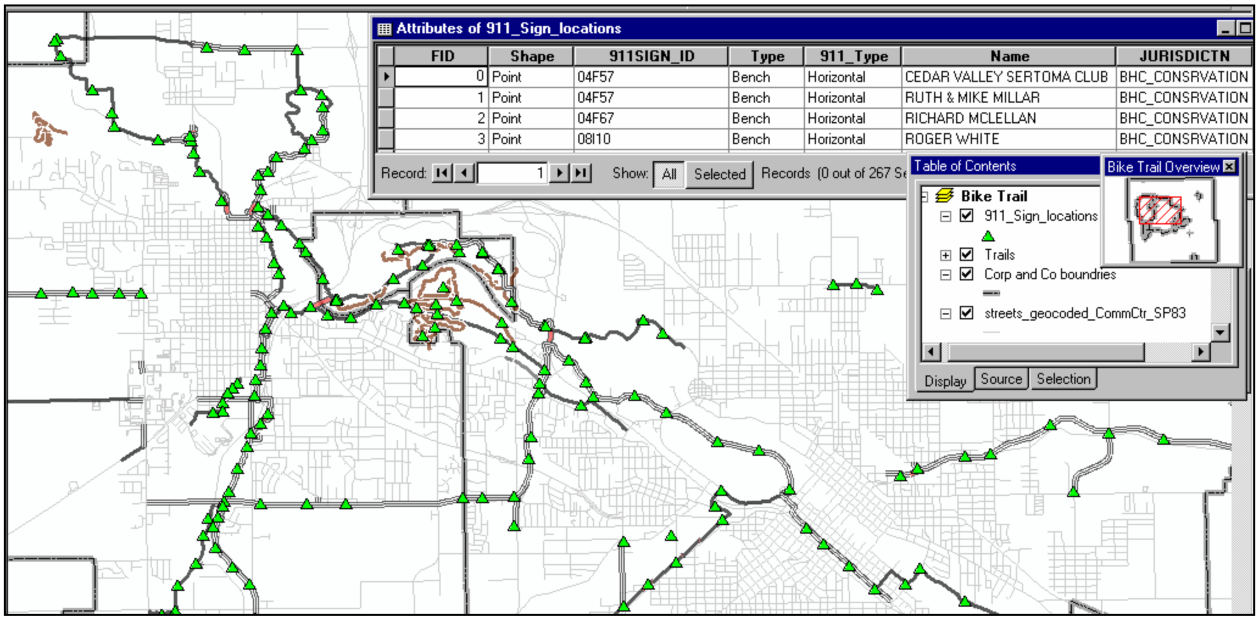

Figure 7. ArcGIS map view illustrating the location of new 911 signs, an attribute table for the 911_Sign _locations data set, and an overview map showing portion of county in the map view

Incorporation of 911 Signs Project Data into the Computer-Aided Dispatch System

Integrating the Trails and 911_Sign_locations data sets into the dispatch center's computer-aided dispatch (CAD) system enables 911 dispatchers to "see" the location communicated by a trail user on the center's CAD map, which is an ESRI© MapObjects application. This visualization helps dispatchers guide the appropriate responders to the emergency site. For the Trail data to work within the CAD system, the line data set representing the Trail was incorporated into the dispatch center's geocoded street centerline shapefile using ArcGIS tools. The first step was to merge all of the individual features representing Trail segments from the original bh_trails_sp83(Surface) shapefile into one line feature, and then output this as a new shapefile named "merged_trail" using the merge tool in the edit tool set. Next, this single feature representing the entire trail system in the county was broken into segments by intersecting the one-square-mile map grid shapefile with the merged_trail shapefile and joining the attributes of the map grid shapefile with the merged_trail attributes in the process. The single merged_trail was broken into 83 line features representing trail segments, and then output as another new shapefile, "intersected_trail." Due to the attribute joining, the intersected_trail shapefile included the X-number and the Y-letter attribute fields inherited from the map grid shapefile. Each of the 83 line features in intersected_trail corresponds spatially to and is located within a particular one-mile grid map area. Any attribute fields not needed for incorporation into the CAD system were deleted. Next, one road from the geocoded street centerline shapefile was selected and exported as a new shapefile (biketrail_geocoded_XY_SP83) to use as a centerline attribute template. An editing session was begun with the biketrail_geocoded_XY_SP83 shapefile set as the target. All of the 83 line features of the intersected_trail shapefile were copied and pasted to the biketrail_geocoded_XY_SP83 shapefile, resulting in 84 records. The record representing the road, that is, the only record with an actual street name, was deleted, leaving only the 83 line features with the same exact attribute table structure, albeit not populated with data, as used in the dispatch center's geocoded street centerline file.

The next step was to populate the 83 records of the biketrail_geocoded_XY_SP83 shapefile via calculations, addition and deletion of fields, and a join. The NAME field was calculated equal to BIKE, and the TYPE field was calculated equal to TRL. Then, the intersected_trail shapefile was spatially joined to the biketrail_geocoded_XY_SP83 shapefile in order to append the X_Number and Y_Letter fields of the intersected_trail shapefile to the biketrail_geocoded_XY_SP83 attribute table (see Figure 8). The PRE_TYPE field was populated by calculation (X_Number and Y_Letter) to preserve the information for the CAD system. A Y_Number field was added as a text field. An X_Y_Number field was also added as a text field. All fields right of PRE_TYPE were eventually deleted. Since the CAD system uses standard LEFTFROM and LEFTTO fields for address numbers, the Y-letter was converted to a number corresponding to the letter position in the alphabet. Then the X_Y_Number field was calculated (X_Number and Y_Number). A zero was appended to the end of the X_Y_Number field. Thus, for example, 406 became 4060, 506 became 5060, and so on.

butes of biketrail_geocode_XY_SP83

ST_TIP	ST_LABEL	PRE_TYPE	X_Number	Y_Letter	Y_Number	X_Y_Number
		4F	4	F	06	406
		5F	5	F	06	506
		6F	6	F	06	606
		7F	7	F	06	706
		5G	5	G	07	507
		6G	6	G	07	607
		7G	7	G	07	707
		4H	4	H	08	408
		5H	5	H	08	508

Figure 8. Portion of biketrail_geocode_XY_SP83 attribute table with X_Number, Y_Letter fields from the join and two fields added for calculations

After appending ”0” to the X_Y_Number field, the LEFTFROM field, which is a numeric field, was set equal to the X_Y_Number field. The other three address range fields were derived from the LEFTFROM field (see Figure 9). Unnecessary fields were then deleted.

eocode_XY_SP83

LEFTFROM	LEFTTO	RIGHTFROM	RIGHTTO	PREDIR	NAME
25240	25249	25241	25249		BIKE
4060	4069	4061	4069		BIKE
4080	4089	4081	4089		BIKE
5060	5069	5061	5069		BIKE
5070	5079	5071	5079		BIKE

Figure 9. Portion of biketrail_geocode_XY_SP83 attribute table illustrating address fields used in geocoding

Once the biketrail_geocode_XY_SP83 attribute table was populated and in the same format as the geocoded street centerline file, an editing session was started with the centerline file as the target. The 83 features in the biketrail_geocode_XY_SP83 file were copied and pasted to the geocoded street centerline file. The CAD system recognized a street named BIKE TRL, and an address range related to a section of that street (see Figure 10). No direct data entry was done to incorporate the trails into the centerline file.

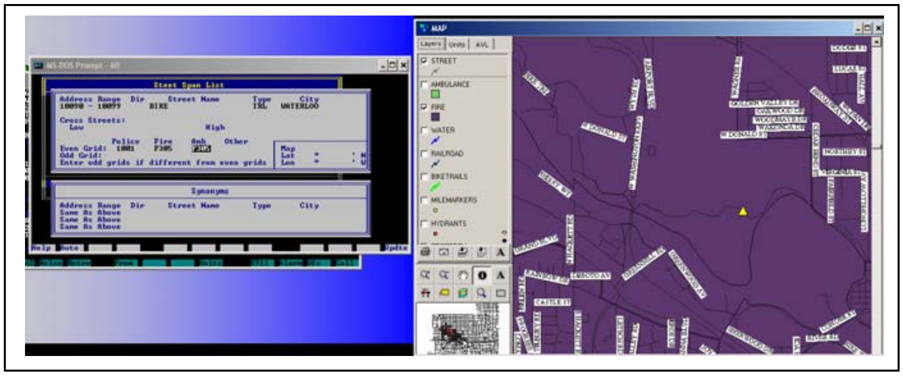

Figure 10. CAD screen showing BIKE TRL address range and a triangle on CAD Map representing the center of that street segment.

Fortunately, the CAD systems allows for the use of alias names. For example, if a caller in need of assistance identifies his or her location as Waterloo Wal-Mart, the dispatcher can enter that information because it is cross-referenced to the Wal-Mart address, and thus to the appropriate address range in the CAD system. That is exactly how the sign identification code is used. The sign identification codes are entered as aliases and are related to the appropriate address range. So, each of the 267 features that will have a 911 sign attached to it, with its particular sign identification code, will be related to 1 of the 83 BIKE TRL street segments. At present, the CAD system allows for only three alias entries. The use of the alias will bring the view on the CAD map to the appropriate BIKE TRL street segment. The 911_sign_locations point shapefile is used in the CAD map to identify the feature with a specific sign identification code.

KEY FINDINGS

While the 911 Signs Project does improve location communications, future updates of the CAD system software will remove some present limitations. The data fields for address numbers (as LEFTFROM and LEFTTO) can hold up to 12 digits in the ArcObjects application. However, the address fields in the CAD database are currently limited to five digits. This issue will be resolved within the next year by the CAD software developer. In some cases, there are more than three 911 signs within a square mile area of the map grid, so allowing more than three alias entries would also be beneficial. However, when a Trail user dials calls 911 and communicates a sign identification code, the dispatcher enters the identification code into the CAD system, which triggers the CAD map to center a particular BIKE TRL segment in the CAD map view. The 911_Sign_location shapefile, with the map tip set to the 911_SignID field, allows the dispatcher to visualize the location of the feature quickly in relation to the nearest Trail access point. Overall, the project has met the intended goals. Cooperative input from different viewpoints proved to be helpful in generating ideas and protocols. The ease of annual review to assess sign condition and readability will be beneficial. The project was accomplished at a very low cost.

CONCLUSIONS

This project provides a practical solution for georeferencing multi-use recreational transportation routes and related assets. The use of GPS and GIS technologies greatly simplified the task of assembling the data needed for improved trail location communication for both public safety, and asset management. Using a Trimble GeoXT GPS receiver, collecting location and attribute information for over 1,700 permanent trailside features, along with related digital photographs for each feature along the100-plus miles of paved and dirt trail in BHC, was accomplished in less than three weeks. Incorporating these georeferenced point features into a GIS project provides easy and quick access to the information needed for location communication and asset management. A map grid system designed with GIS software provides a means to assign an identification code to selected features, primarily benches, along the Trail. The first three characters of the five-character alphanumeric identification code on the newly designed 911 sign locate a feature to within a particular square mile on the map grid. The last two characters define the location to a particular one-tenth square mile area. With all data in digital format, any additional trails and trail features can be incorporated easily into the existing data sets. Basic computational and analysis tools in ArcGIS (ArcView) made processing the shapefiles for incorporation into the dispatch center's CAD system an easy task. Annual review of asset conditions will be accomplished efficiently by loading the GIS data back into the GPS receiver. Cooperation between numerous agencies involved in this project, along with grant funding and in-kind contributions, enabled speedy completion of this project. The entire project, including sign installation and location communication testing, was completed within one year of its original conception.

ACKNOWLEDGMENTS

The 911 Signs Project is a multi-agency cooperative project. Project funding comes from the Cedar Trails Partnership grant and in-kind local donations. The authors would like to acknowledge the Cedar Trails Partnership organization (http://www.cedartrailspartnership.org/) for their generous support of this project through grant funding.

Market Analysis of Alternative Crop Production in Iowa

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ABSTRACT

Large retail outlets convert market fluctuations for fresh fruits and vegetables into consistent shipments by distributing them over a large customer base as close to end use as possible; increasingly large stores link increasingly large volumes to increasingly large farms. In this regard, reliable access to fresh fruits and vegetables has become heavily dependent on the physical movement of freight from increasingly centralized and distant locations.

At the same time, reliance on long-distance, over-the-road truck activity is becoming problematic in terms of fuel use, traffic congestion, air quality, roadway deterioration, driver shortages, security issues, and other concerns. Domesticating and reducing demand for transportation energy is becoming a key political topic; agri-tourism, direct marketing, and local identity labeling are gaining cultural popularity among consumers who are becoming intrigued with rural development and sustainability. Under appropriate conditions, localized sourcing could represent an alternative to a growing reliance on transportation fuel.

On the supply side, new crop varieties and controlled environment techniques are expanding the ability of producers to grow some crops year-round in non-traditional locations. Concepts linking automated equipment to electronic scanning and control technologies are emerging to offset the cost and yield advantages produced by a surplus of migrant laborers in central production areas. Forecasting, organizational, and inventory tracking technologies are advancing at an accelerated pace to virtually link a diverse array of upstream suppliers to downstream demand.

The "Iowa Fresh Fruit and Vegetable Planning Spreadsheet" was developed to illustrate how demand aligns with supply at the county level in Iowa and to identify objectives for minimizing transportation and handling requirements, reducing revenue outflow from the state, and positioning Iowa closer to becoming self-sustaining in fresh fruit and vegetable production. The intent is to provide a tool for producers, distributors, and policy makers to identify volume and infrastructure goals for products that can currently be grown in the state, as well as to position longer term targets for improving local yields with research into non-traditional horticultural methods. The spreadsheet is intended to provide general direction in the context of strategic decision making; results are not intended to be precise.

Note: This research was still in progress at the time of publication; contact the author above for more information.

Key words: alternative crop production—decentralized production—freight transportation

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Alternatives to Truck Engine Idling

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ABSTRACT

This report describes activities and observations gathered during a multi-year project to develop awareness of long-duration truck idling in Iowa and the Midwest. Near-term approaches to address long duration idling include a number of technologies that are generally classified as either mobile or stationary. Mobile technologies refer to truck mounted devices designed to offset use of the main engine to support necessary heating, ventilating, and air conditioning and other "hotel" loads. Stationary technologies provide some form of interface between the truck and the electric grid; because trucks frequently (but not always) park at truck stops, grid-based approaches are commonly referred to as truck stop electrification (TSE).

Based on the insights that emerged from this study, we have determined that implementing a practical TSE demonstration in Iowa in the near future will be limited by several factors that are explained more thoroughly in the report. Iowa is currently meets all air quality standards relevant to truck idling, and ranks just below average (30th) in terms of projected truck parking demand. This implies that resources to advance stationary TSE facilities would more appropriately be targeted to other areas of the country with higher traffic and more urgent air quality concerns.

Because it was determined that Iowa is a net supplier of truck services, it was recommended that Iowa's near term contribution to advancing idle reduction could more cost effectively be served by focusing on the adoption of mobile technologies by Iowa based trucks that operate primarily out of state, and, in particular, in non-attainment or other regulated areas. We therefore proposed to wait and see whether unsubsidized TSE concepts prove commercially successful in other states, whether they become less expensive, or whether more mobile alternatives (including hybrids) emerge to service idle reduction across broader geographic and functional contexts.

Key words: auxiliary power unit—mobile idle reduction technology—truck idle reduction—truck stop electrification

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PROBLEM STATEMENT

The hours of service (HOS) rules enforced by the Federal Motor Carrier Safety Administration (FMCSA) generally require commercial vehicle drivers to rest for a ten-hour (consecutive) off-duty period after accumulating eleven hours of driving time or being on-duty for fourteen hours (Freund 2004). These ten-hour off-duty periods generally occur when drivers are likely to engage in long-duration idling.

Drivers that are beyond returning to a home base commonly spend off-duty periods in truck mounted sleeper berths, parked at private truck stops, public rest areas, freight terminals, or other locations. During heating or cooling days, they typically have few options other than to idle the large 400–500 horsepower engines to power loads that typically require less than 10 horsepower to operate. This causes the main engine to run for long periods of time at low efficiency, and with disproportionately high emissions.

The need to power heating, ventilating, and air conditioning (HVAC) and other "hotel" loads for several hours at a time and to maintain functional engine and fuel temperatures during cold weather is generally what differentiates long-duration from short-duration idling. Short-duration idling is generally more likely to occur while the drivers are on-duty, for example while they are conducting vehicle checks, refueling, waiting to load or unload, taking lunch breaks, or while they are stuck in traffic.

Near-term approaches to address idling include a range of technologies that are generally classified as either mobile or stationary. Mobile technologies refer to truck mounted devices that offset use of the main engine to support HVAC, fuel warming, and other "hotel" loads. Stationary technologies provide some form of interface with the electric grid; because trucks frequently (but not always) park at truck stops, grid-based technologies are commonly called truck stop electrification (TSE). Other approaches may include regulatory and fleet programs that either punish or reward drivers for unnecessary idling. However, because these by themselves do not accommodate the HVAC needs of the drivers or the fuel and engine needs of the truck, these programs were generally ignored within the context of this project.

Current mobile technologies include engine timers that start or stop the main engine whenever a specified parameter is reached (such as time, temperature, or battery voltage); direct-fired heaters that provide cab, fuel, and engine heat, but not air conditioning or electricity; extra batteries that charge while the truck is running to power specially installed electric HVAC and appliances after the truck shuts down; and auxiliary generators, powered by a second diesel engine more appropriately sized for hotel loads, that provide HVAC and other capabilities. Several of these devices may be used in various combinations.

Direct-fired heaters and engine-off systems generally cost about $1,200/truck. However the frequent cold starts that result when engine timers are used are generally hard on the main engine and disruptive to a driver's rest period; direct-fired heaters do not accommodate air conditioning during peak ozone months, which is a key objective of idle reduction policy. Current battery technology is generally inadequate to support long duration load requirements within acceptable volume and weight limits.

For the purposes this report, mobile systems that are capable of providing fully functioning HVAC, electric power, and system support, with minimal limitations, are referred to as auxiliary power units (APU); current retrofit versions generally cost between $6,000 and $8,000/truck. Future versions are likely to evolve under the context of hybrid truck development.

TSE options are generally categorized as either full service or shorepower. Full service TSE integrates all of the HVAC and auxiliary functions into a completely land-based system. Shorepower requires compatible (electric) HVAC and support systems on the truck that are powered by land-based receptacles installed where trucks frequently park. Depending on the number of spaces at a site, installation costs for full service TSE are generally between $15,000 and $18,000 per parking space. Commercial shorepower generally costs about $6,000 per parking space; compatible electric HVAC retrofits can vary from $100 for a resistance heater to $3,000 for an inverter and fully functioning system.

Although truck manufacturers are starting to integrate shorepower as a factory option, most trucks operating today are not compatible. This is largely because there are few shorepower-equipped parking spaces available, and without them, there are few opportunities for truck owners to recover the cost of the added investment; this in turn limits the commercial viability of installing stationary infrastructure. A long-term strategy of IdleAire, one of two full service providers (the only one with facilities installed) is eventually to remove the stationary HVAC components from its full service platform and incrementally convert to less expensive shorepower as a larger number of compatible trucks enter the market (Doty 2004).

The key advantages of TSE relative to most fully functioning on-board options is that it completely eliminates site-specific emissions, facilitates the documentation of emission reduction credits (when applicable), and either eliminates or reduces the up-front investment needed by truck owners. The primary drawbacks are that it substantially limits the locations and conditions under which idle reduction can occur and currently requires the addition of a service fee and/or public funding to offset installation and maintenance costs of the stationary infrastructure. It is also unlikely that TSE will be practical for short-term idling that does not warrant the time and cost required to hook up and log into a TSE facility.

Fully functioning mobile devices allow idle reduction almost anywhere and anytime a truck idles, however, current options do not completely eliminate on-site noise or emissions, and are either functionally limited or require a payback period that exceeds what is acceptable to most truck owners. In regard to the objectives of this project, a significant difference between TSE and the mobile approaches revolve around the up-front involvement needed to advance further development.

TSE generally requires indirect investment by third parties, a payback that balances the price of diesel fuel against the grid price of electricity (along with added labor and other operating costs), and coordinated development across a spectrum of service providers, site owners, truck owners, regulatory bodies, and others. Completely mobile approaches require cost reduction by manufacturers, a straightforward investment by truck owners, and a payback that is directly tied to the price of diesel fuel.

PROJECT OBJECTIVES

The primary objective of this the project was to develop awareness of alternative truck idle reduction technologies in Iowa and along the Interstate 35 trade corridor. Secondary objectives were the following:

1. Organize a one- to two-day conference carried out sometime during May or June 2004 that supports commercial adoption of idle reduction technologies in Iowa

2. Develop interim involvement of stakeholders with appropriate expertise, practical knowledge, contacts, and peripheral resources to support implementation of truck idle reduction in Iowa

3. Carry out activities defined by an initial focus group survey to develop relevant knowledge of truck idle reduction technologies and implementation issues

A longer term mission was to prepare demonstrations of truck idle reduction technologies for Iowa.

PROJECT METHODOLOGY

Preparation for the conference was the central objective of the activities covered under this phase of the project. Supporting the conference required developing a summary understanding of key issues relating to truck idle reduction, aligning with key contacts and expertise, and defining and promoting the issue to a constituency base and target audience.

An amendment to the work plan during the first year added the task of initiating a demonstration of commercial shorepower in Iowa. This activity resulted in the development of two proposals and a summary assessment of idle reduction as it relates to the state. This included assembling data on air quality, truck traffic, truck stops, and truck parking in the state; targeting key geographic areas; and involving relevant stakeholders. It should be noted that neither proposal resulted in funding, but did succeed in generating focused discussion among a range of potential stakeholders.

The first was a multi-state proposal to the National Association of State Energy Officials' State Technologies Advancement Collaborative (STAC) that was led by the New York State Energy Research and Development Authority and that would have coordinated shorepower deployment in Iowa with parallel activities in California, Maryland, Massachusetts, New York, and North Carolina.

The second proposal was prepared specifically for Iowa and submitted to the U.S. Environmental Protection Agency's (EPA) National Transportation Idle Free Corridors Program. This proposal brought about the involvement of the Bi-State Regional Commission, Shurepower LLC, MidAmerican Energy, the Iowa Energy Center, and the Iowa Department of Natural Resources.

A common theme to each proposal was to launch a shorepower site in Iowa that would then expand to other locations from commercial service revenue. This would require the demonstration facility to be situated in one or more highly visible locations, compatible with emerging truck technologies, and coordinated with the development of stationary facilities in other states. The STAC proposal took the approach that grant funding would be secured first, a study would then be conducted to identify the best place(s) in Iowa to demonstrate the facility, and a TSE provider would be supported with project funding at one or more truck stops, with little or no investment risk to the host site.

The approach taken for the EPA proposal, however, required us to identify and secure agreement of the host site in advance of submitting the proposal. The evaluation that resulted highlights key limitations in launching TSE in Iowa. These issues are discussed further in the following section.

KEY FINDINGS

Overview

The EPA estimates that long duration truck idling consumes 960 million gallons of diesel fuel throughout the United States and generates 11 million tons of Carbon Monoxide, 180,000 tons of Nitrogen Oxides, and 5,000 tons of particulate matter (PM) annually (U.S. EPA 2004). This is based on assumptions of eight-hours of idling per day; 300 days per year; 0.8 gallons of fuel used per hour; and a fleet estimate of 500,000 long-range, heavy-duty trucks (Lim 2002). On a per-truck basis, this equates to 2,400 hours and 1,920 gallons per year per truck. It should be noted that the eight-hour-per-day value was based on the FMSCA off-duty requirement in effect at the time; new rules adopted in 2004 increased this requirement to 10 hours.

A separate report issued by Argonne National Laboratory in June 2000 estimates that long haul truck idling consumes 838 million gallons each year. This value is derived from average idling of 1,830 hours per truck per year based on direct discussions with industry; fuel consumption of one gallon per hour of idling based on testing commissioned by the American Trucking Associations (ATA); and a U.S. fleet of 458,000 long-range heavy-duty trucks identified in the 1997 Vehicle Inventory and Use Survey (VIUS) (Gaines, Stodolsky, and Vyas 2000).

A third report, issued by the U.S. Department of Energy (DOE) (Slezak 2004), indicates average idling of about 2,000 hours per year per truck, or between 20%–39% of the time the engine is running. This value is based on direct participation by fleets, owner-operators, and manufacturers in two workshops carried out in 2003. This report indicates that about half of the fleets and 17% of owner-operators that participated in tracking idling, presumably using on-board computers.

Assessment

All three of these reports indicate that idling varies considerably by driver, location, and time of year. For example, most drivers require HVAC during the mid-summer and mid-winter months. However, while some frequently shut down when the weather is more moderate, others almost never shut down. Some indicate that leaving the windows open poses a safety risk in certain areas of the country; others idle out of habit. Further complicating the estimates are that different trucks, engine speeds, and load requirements inherently consume different rates of fuel while idling. The three reports generally agree, however, that idling by interstate trucks consumes roughly 2,000 gallons of fuel per truck each year.

Because data available at the time did not specifically identify the number of trucks equipped with sleeper berths in the United States, all three reports either directly or by reference based fleet size on the approximate number of long-range heavy-duty trucks identified in the 1997 VIUS. The VIUS defines long-range trucks as those with either no home base or those that operated more than 500 miles from a home base most frequently during the year surveyed (U.S. Department of Commerce 1999). This implies that vehicles are typically on the road either continuously or for at least 1,000 miles round trip. Using an average driving speed of 50 miles per hour implies driving times of more than 20 hours per trip, not including time spent loading, unloading, refueling, stopping for lunch, or for a number of other short-term reasons.

Because the implied trip time of long-range trucks clearly exceeded the daily driving time allowed by FMSCA, it was assumed that the majority of trucks listed in this category included sleeper berths and inherently represented the population that relies on long-duration idling to service overnight HVAC and hotel loads. Under this premise, however, it could also be assumed that a significant number of long-range medium trucks also met these criteria; these trucks are defined by an operating range of between 201 and 500 miles from a home base, implying between 8 and 20 hours round trip.

The 2002 VIUS (released December 2004) was the first to specifically identify the number of trucks in the U.S. fleet that are equipped with sleeper berths. Of the 677,600 indicated nationwide, 11,000 were based in Iowa; 98% were on single, double, or triple trailer combinations, 99% of which used diesel fuel (U.S. Department of Commerce 2004). The VIUS did not specifically correlate trucks with sleeper berths to range of operation. However, it should be noted that range of operation was only used to estimate fleet size; per-truck fuel consumption is based on direct discussion with fleets, and is not necessarily contingent upon range of operation.

As such, applying 2,000 gallons per truck to the 667,600 trucks in the U.S. fleet with sleeper berths implies that total fuel consumed throughout the United States may actually be as high as 1.35 billion gallons annually. This equates to approximately 3.7% of the 37.1 billion gallons of on-highway diesel fuel purchased throughout the United States in 2003 (U.S. Department of Energy 2003).

Estimates of Idling in Iowa

Applying the same 3.7% ratio to the 509 million gallons of fuel purchased in Iowa indicates that 18.6 million gallons of the fuel sold may have similarly been consumed by idling, both in-state and out-of-state at publicly accessible parking facilities and a number of other locations. Alternatively, applying an average fuel use of 2,000 gallons per truck to the 11,000 trucks based in Iowa totals 22 million gallons, which is 18% higher than the 18.6 million gallons estimated from fuel purchased. This could be interpreted as an indication that Iowa is a net provider of fleet services to other areas of the country, and that more Iowa-based trucks idle out of state than vice versa.

Idling At Commercial Truck Stops and Public Rest Areas

A 2002 report by the FHWA indicates there are approximately 315,850 available truck parking spaces at commercial truck stops and public rest areas nationwide, indicating a 10% surplus over the estimated daily peak-hour demand of 287,000 (FHWA 2002). Even though the report indicates a nationwide surplus, however, concentrated shortages exist in several high-traffic states, including Illinois, Indiana, Ohio, and some of the northeast and west coast states. In Iowa, peak-hour demand is estimated at 2,990 spaces, compared to approximately 6,013 available spaces, indicating a surplus of about twice what is needed (FHWA 2002a).

Peak-hour demand tends to occur at night, with spaces starting to fill in the late afternoon and early evening hours. The model used by the FHWA to calculate peak hour demand is based on Annual Average Daily Truck Traffic (AADTT), cumulative eight-day driving limits (which did not change with the new HOS rules), and other adjusting factors (FHWA 2002b). Deriving the values from AADTT implies that the model does not cause multiple off-duty parking demands to overlap in a single space during the course of a day; a space used for off-duty parking during the day implies a vacancy at night. Surplus spaces vacated during non-peak hours were considered sufficient to accommodate other short-term parking that occurs during the day.

As such, each calculated unit of parking demand is interpreted to represent the idling associated with a single truck. Multiplying annual fuel consumption of 2,000 gallons per truck (including short-term idling) by peak hour demand for 2,990 parking spaces implies that approximately 6.0 million gallons of fuel are consumed by idling each year at the commercial truck stops and public rest areas located in Iowa. This is only 32% of the 18.6 million gallons estimated from fuel sold in the state, implying that almost two-thirds of idling by trucks equipped with sleeper berths occurs at locations other than in-state commercial truck stops and public rest areas. A similar value at the national level indicates that 43% of the 1.35 billion gallons is consumed at commercial truck stops and public rest areas throughout the country.

Regional Summary

A more direct way to characterize long-haul truck idling in Iowa is to consider that 1.6% of the U.S. fleet (with sleeper berths) is based in Iowa, but only 1.4% of national on-highway diesel fuel sales, and 1.0% of national peak-hour parking is estimated to occur in the state. This implies that, relative to other parts of the country, proportionately more trucks are based in Iowa than operate in it, and more operate in it than park in it. In other words, Iowa could be considered a net provider of trucks that operate in other states, and in general, relatively more of a fuel depot than an overnight parking location.

Similarly, the percentages of trucks are higher than the percentages of fuel sales in each of Iowa's bordering states, implying that the region as a whole is generally a net provider of out-of-state fleet services. In Illinois to the east and Missouri to the south, however, the percentage of parking demand is higher than that of fuel sales, implying that each is relatively more of an origin/destination, and consequently better suited to advancing early-stage TSE infrastructure. Other states, including Iowa, could be considered more pass-through in nature. Table 1 illustrates these percentages for Iowa and its bordering states.

Table 1. Percent of U.S. trucking totals by state

	% of U.S. fleet	% of parking demand	% of highway diesel fuel	Parking / fuel sales
Missouri	3.5%	4.0%	2.6%	1.53
Illinois	8.8%	5.1%	3.8%	1.34
Kansas	1.8%	0.9%	1.1%	0.79
Minnesota	2.7%	1.3%	1.7%	0.76
Iowa	1.6%	1.0%	1.4%	0.76
South Dakota	1.1%	0.3%	0.5%	0.65
Wisconsin	2.6%	1.0%	1.8%	0.53
Nebraska	3.8%	0.4%	1.0%	0.37
U.S. totals	677,600	287,316	37 billion gallons	1.00

Figure 1 illustrates AADTT (and associated idling) locations and relative densities throughout the United States (FHWA 2004). In general, AADTT appears to be more heavily concentrated in states to the east of Iowa, with Interstate 80 projected to serve as a significant channel through Iowa.

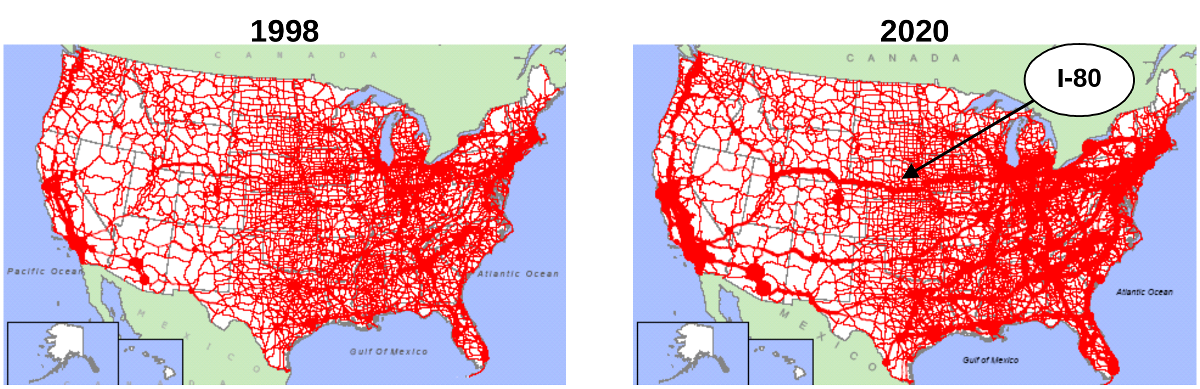

Figure 1. Projected AADTT for 1998 and 2020

Figure 2 associates the FHWA peak-hour parking demand estimate for Iowa to summary AADTT values surrounding key locations in the state (Iowa Department of Transportation 2002). It can be observed that the east-west traffic on I-80 is generally about three times that of the north-south traffic on either I-35 or I-29. It is also heavier to the east, and gradually disperses toward the west, except where I-80 meets I-35 in Des Moines. It could be inferred that part of the traffic that enters from the east generally starts to divert north after it enters Iowa. The Quad Cities and Des Moines appear to have the highest concentrations of truck activity. As will be discussed, the primary area of interest in regard to TSE narrows to the Quad Cities when air quality is taken into consideration.

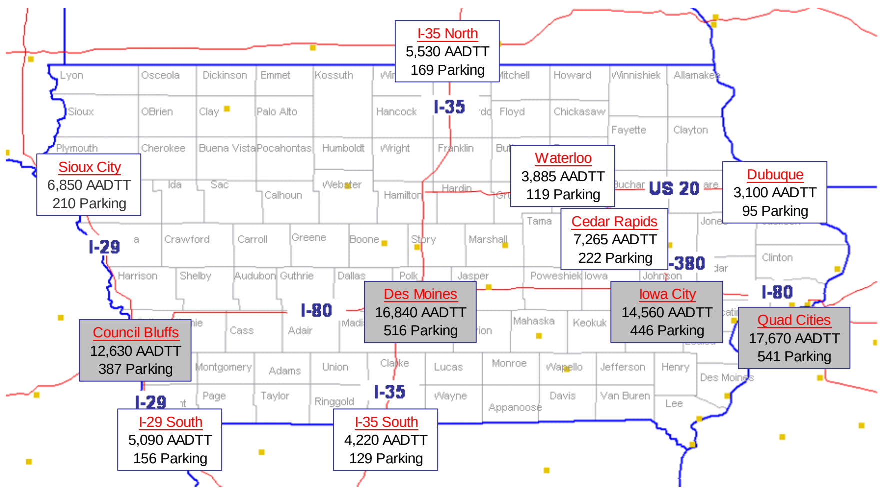

Figure 2. AADTT and parking estimates at key locations in Iowa

Air Quality

Actions to improve air quality in non-attainment areas are generally determined by the requirements of state implementation plans (SIPs), which define the strategies and control measures that local jurisdictions agree to implement to attain National Ambient Air Quality Standards (NAAQS). States or municipalities in non-attainment areas generally define the terms of the SIP, with requisite approval by the EPA (U.S. EPA 2004a). Such measures might include local ordinances, permit restrictions, or use of state and local funding to finance control technology that helps mitigate air quality problems. In cap and trade areas, industry-sponsored measures such as TSE, which permanently reduce emissions, may generate credits (with a cash value) that can be traded or used to offset future requirements to improve air quality.

Figure 3 (U.S. EPA 2004b) illustrates areas of the U.S. that are designated by the EPA to be in non-attainment of NAAQS for the pollutants of primary concern to truck idling; these are ground-level ozone and particulate matter < 2.5 micrometers in diameter (PM2.5). These areas offer relatively more immediate incentives to draw early-stage public and private investment in stationary TSE facilities. It can be observed that, compared to many other parts of the country, Iowa is located in an area with relatively few problems in this regard.

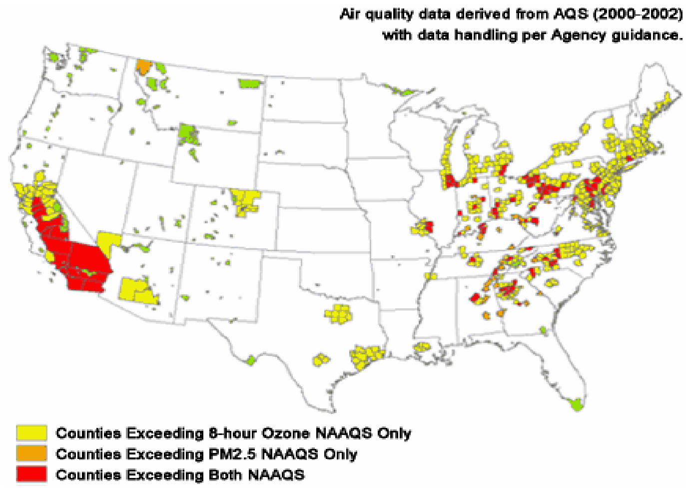

Figure 3. Areas of non-attainment in the U.S.

Figure 4 displays 2003 summary values from air quality monitors in Iowa, expressed as percentages of the NAAQS. Measured values that are within 90% of the respective NAAQS for either ozone or PM2.5 are highlighted in grey. The corresponding values, identified from an EPA website (U.S. EPA 2004c), are listed in Table 2. It can be observed that of the areas along an interstate corridor (with sufficient parking demand to support a TSE facility), the Quad Cities region is closest to exceeding NAAQS criteria.

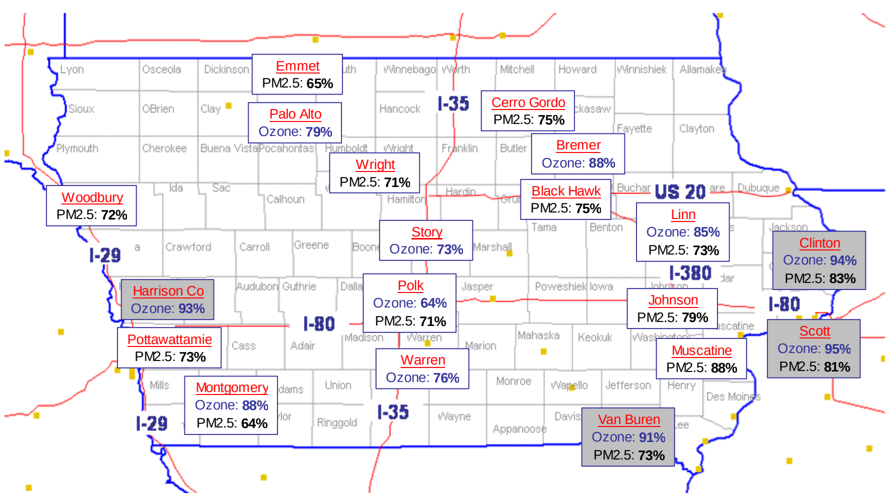

Figure 4. Iowa air quality values relative to NAAQS—2003

Table 2. Summary values in Iowa—ozone and PM 2.5

Location	Ozone 2003 summary value	% of NAAQS *0.08 ppm (8-hour average)*
Scott	0.076	95%
Clinton	0.075	94%
Harrison	0.074	93%
Van Buren	0.073	91%
Montgomery	0.070	88%
Bremer	0.070	88%
Linn	0.068	85%
Palo Alto	0.063	79%
Warren	0.061	76%
Story	0.058	73%
Polk	0.051	64%

Location	PM 2.5 2003 summary value	% of NAAQS *15.0 μg/m^3 (annual mean)*
Muscatine	13.2	88%
Clinton	12.4	83%
Scott	12.1	81%
Johnson	11.8	79%
Black Hawk	11.3	75%
Cerro Gordo	11.2	75%
Linn	11.0	73%
Pottawattamie	11.0	73%
Van Buren	10.9	73%
Woodbury	10.8	72%
Polk	10.7	71%
Wright	10.6	71%
Emmet	9.7	65%
Montgomery	9.6	64%

Cost of Alternatives

To date, air quality and energy efficiency have been the primary factors driving idle reduction. Air quality is generally a public policy (i.e., government) issue; energy efficiency is primarily a voluntary business concern justified primarily by operational fuel savings. With the exception of some of the less-than-fully-functioning options, most of the technologies available today are still viewed by truck owners as either not, or only marginally, worth the costs based on fuel savings alone. Current pricing has yet to motivate widespread adoption of fully functioning idle reduction technologies useful for ozone reduction.

Insights gathered from the industry workshops reported by the U.S. DOE (Slezak 2004) indicate that owners will require a two-year maximum payback and will likely base decisions solely on fuel costs, and that “the strongest interest is for combined heating/cooling/electrical systems with cooling or heating alone a distant second.” A fully installed cost of between $3,000 and $4,000 (not including operating costs) for a fully functioning system was indicated as an acceptable crossover value that would motivate truck owners to adopt mobile technologies in 2003. When the workshops were conducted, on-highway diesel prices averaged $1.51/gallon; as of December 2004, prices were $2.07/gallon, implying that the target value may now have risen by 41% (to between $4,100 and $5,500, with an average of $4,800).

Known technologies that are currently available to provide fully functioning HVAC and electricity for appliances are listed on an EPA website (U.S. EPA 2004d). These include auxiliary power units that average roughly $7,000/truck (varying between $6,000 and $8,000) and a 12 % federal excise tax of $840, which implies a total average of approximately $7,840. Full service TSE is known to cost approximately $18,000/parking space, and commercial shorepower is known to cost approximately

$6,000/parking space and require a $3,000 truck-mounted inverter/electric HVAC package to be fully functioning.

According to Argonne (Gaines, Stodolsky, and Vyas 2000), the energy needed to operate a typical APU is approximately 18% of the fuel consumed by idling, indicating a cost of about $0.37 per hour at $2.07/gallon. The two TSE options require a fee to truck owners of $1.25/hour for full service ($1.50 without a fleet discount) and $0.75/hour for shorepower. Table 3 compares approximate system costs and paybacks for each, based on 2,000 hours of use per year and disregarding qualitative issues relating to reliability or accessibility. Current APUs generally appear to have the shortest rate of payback when taking subsidized (infrastructure) costs into consideration. Because they also require a less-fixed capital requirement, it is likely that they will also evolve toward lower cost (technologically) at a faster rate.

Table 3. Approximate system costs for known alternatives

	Full service TSE	**Shorepower**	**APU**
System investments			
Parking space equipment	$18,000	$6,000	$0
Truck mounted equipment	$10	$3,000	$7,840
Total system investment:	*$18,010*	*$9,000*	*$7,840*
Operational savings			
Fuel savings	$4,140	$4,140	$4,140
Less: operating costs	($2,400)	($1,500)	($745)
Net annual savings:	*$1,740*	*$2,640*	*$3,395*
Simple payback (years):	***10.4***	***3.4***	***2.3***

Other Situational Factors Regarding TSE in Iowa

In regard to implementing a practical demonstration of shorepower TSE in Iowa, other situational factors were discovered that should also to be taken into consideration. These include the following:

1. Commercial parameters for known TSE platforms (IdleAire, Shurepower, and AirPower) are significantly dependent upon government funding to support installation; air quality issues are the primary factor driving government support. Iowa is in attainment of the truck idling air quality standards and, as such, is likely to be a low priority in national competitions for federal funding to support early-stage TSE deployments.

2. Federal highway funding for transportation projects that help non-attainment areas meet air quality standards is allocated to states through the Congestion Mitigation and Air Quality Improvement program. Because Iowa is in attainment, a base allocation is distributed through the Iowa Clean Air Attainment Program (ICAAP); this fund is relatively smaller and less restricted than in states with non-attainment areas. In FY 2005, ICAAP awarded $4.7 million, primarily to municipal road and transit projects, that reduce congestion-related idling across several modes, including trucks and passenger vehicles (Iowa Department of Transportation 2002a).

3. Restrictions on state-managed rest areas prohibit the collection of fees that would be essential to maintaining and advancing a commercial TSE platform. This in effect limits potential locations in Iowa to private truck stops; Figure 1 illustrates the locations of commercial truck stops operating in the state. The Quad Cities, Des Moines, and Council Bluffs areas (along I-80) are key locations in terms of truck traffic; the Quad Cities are of primary concern in regard to air quality concerns. Participation by major chains is viewed as essential in terms of coordinating standards, marketing, and positioning other facilities. The seven major chains highlighted in gray either indicated that they were not interested in participating in a shorepower demonstration or that they already have (corporate) right-of-first-refusal agreements in place or pending with IdleAire (Doty 2004).

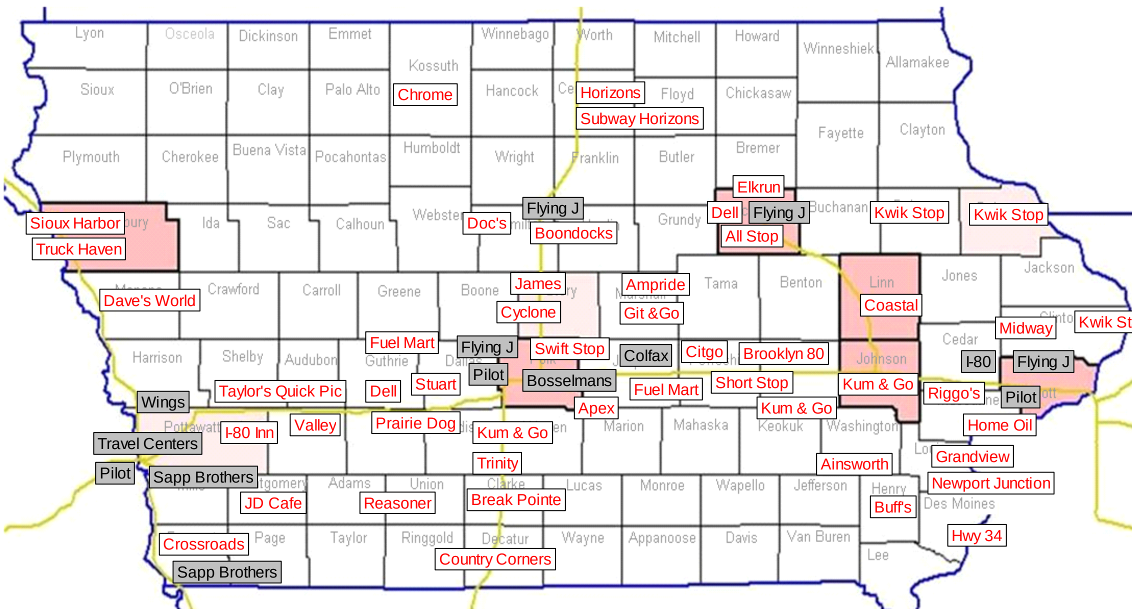

Figure 5. Commercial truck stops in Iowa

4. At $15,000–$18,000/parking space for a 50-space minimum facility, a full-service IdleAire site would cost between $750,000 and $900,000 to install. Less a 20% ICAAP cost share requirement (by IdleAire), this implies that the cost to Iowa would be between $600,000 and $720,000 per facility, or approximately 15% of the ICAAP fund. This would need to be justified against the peripheral infrastructure and traffic flow benefits that result from competing municipal road and transit projects.

5. Competitive national programs that target idle reduction include the STAC and Idle Free Corridors programs, both of which include air quality as an evaluation criterion; neither of our applications to these programs resulted in an Iowa project. A third is the Clean Cities program administered by the DOE; the state of Iowa is currently in the process of seeking designation to apply for project funding under this program.

CONCLUSIONS

An underlying purpose of this project was to establish a position for Iowa in advancing idle reduction technologies. Our initial focus anticipated the development of a market-responsive pilot demonstration of stationary shorepower within the state. After further review, however, we have concluded that Iowa's contribution to idle reduction could more cost effectively be served by refocusing toward facilitating the adoption of mobile technologies by Iowa-based fleets and owners, particularly those that operate in out-of-state non-attainment areas.

In evaluating the situation for a shorepower demonstration, we determined that Iowa is generally a net provider of trucks that operate in other states, and that the trucks that do stop in the state are more likely to engage in short-duration idling than in long-duration idling. We estimate that almost two-thirds of the idling by trucks equipped with sleeper berths that results from fuel sold in Iowa occurs at locations other than at commercial truck stops and public rest areas in the state.

At the same time, Iowa is currently in attainment of all air quality standards relevant to truck idling, and ranks just below average (30th) in terms of projected truck parking demand. From a national perspective, the focus of out-of-state public and private resources to support stationary TSE facilities is more likely to be targeted to other areas of the country that have higher traffic and more urgent air quality concerns. This implies that a stationary demonstration in Iowa would be largely dependent on limited state or local resources at the expense of other road and transit projects (that also generate idle reduction benefits).

At the same time, sites in Iowa that could support a practical stationary facility are currently limited to a relatively expensive full-service option. The developer of the system has expressed a strategy that will eventually transition it to less expensive shorepower as a larger number of compatible trucks enter the market. In this regard, a significant share of the installation costs of an early stage demonstration facility could end up funding equipment that will likely become obsolete before the end of its useful life. Delaying an Iowa-based demonstration until the future, however, could potentially offset the added costs of the full-service HVAC components altogether.

When taking subsidized costs into account, existing mobile technologies already appear to demonstrate the lowest combined investment, shortest payback, and highest function in terms of addressing short-term and other idling that occurs beyond off-duty periods at commercial truck stops. The mobility of these approaches would allow Iowa-based fleets to better meet compliance in existing non-attainment areas when necessary.

Mobile technologies also reduce the need to introduce third party operating margins into the solution, and provide a relatively more fluid approach for advancing early-stage technologies. A 50-space full-service TSE facility would lock in a significant share of limited state funding to a single method, with no regulatory requirement or reliable indication of fleet acceptance. The same level of funding could be used to evaluate a variety of alternatives that could be improved incrementally based on fleet-driven experimentation and recommendations.

Because Iowa is currently under minimal pressure to do so at this time, we recommended that the state wait and see whether unsubsidized TSE concepts prove successful in other states or become less expensive, or whether more mobile alternatives emerge to service idle reduction across broader geographic and functional contexts. Our goals during the interim are to facilitate and monitor pilot demonstrations of new or emerging technologies with emphasis on fleet mobility and gaining a better understanding of the situational requirements encountered by drivers.

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Iowa's High-Mast Lighting Towers: A Proactive Approach to a Problem

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ABSTRACT

The Iowa DOT owns 233 high-mast lighting towers, ranging from 100 to 180 feet tall. These structures are typically located at the intersection of major highways and interstates and provide broad illumination to the traveling public. In 2000, a statewide inspection was conducted of the 193 towers in the inventory at that time. A 140-foot tower located near Sioux City collapsed in 2003 due to the development of a large fatigue crack at the welded connection at the base plate. Subsequently, cracks were found in 20 other towers. In all cases, the cracked structures were taken out of service.

This paper will present forensic information about the collapsed and cracked towers and describe the instrumentation and long-term monitoring of Iowa towers. Preliminary findings of the monitoring will be included. The goal of the testing is to determine the types of vibration and stress ranges that the towers are experiencing. This testing will provide a better understanding of the ways the towers in Iowa will perform over time and will help develop policy for long-term inspection and maintenance of these structures.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: high-mast light towers—instrumentation and monitoring of light towers—long-term light pole performance

Regional Mobility Authorities in Texas

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ABSTRACT

A regional mobility authority (RMA) is an innovative tool that brings transportation decisions to the local levels and provides local governments the opportunity to accelerate needed projects. An RMA improves mobility, enhances economic development, and provides potential revenue sources through designing, constructing, operating and maintaining tolled or non-tolled roadways, passenger and freight rail, airports, and pedestrian and bicycle facilities.

Although the idea of an RMA is not a new way to finance transportation in the United States, Texas House Bill 3588 and its predecessor, Senate Bill 342, have opened the door to debt financing in Texas. With this additional innovative tool to finance transportation projects in Texas, it is important to understand the functions of RMAs and draw conclusions about the effects they might have in the state.

To accomplish this task, literature on the subject was reviewed and input from professionals involved in transportation and the RMA process were obtained. Several questions targeted at addressing several key objectives were created. A call list was developed, and from that list eight interviews were conducted with professionals from state agencies, local organizations, and research institutions. Financial options before the introduction of RMAs were compared to current practices in the state, and conclusions were drawn about their effectiveness. Upon completing the interviews and literature review, conclusions about RMAs were drawn on the following topics:

- Short-term benefits
- Short-term problems
- Long-term benefits
- Long-term problems
- Barriers to forming an RMA
- Agency cooperation

Key words: innovative financing—mobility—toll roads

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INTRODUCTION

In June 2003, Texas House Bill 3588 became law. House Bill 3588 amended the governing Regional Mobility Authority (RMA) legislation, Senate Bill 342. This gave the Texas Transportation Commission general oversight to create and dissolve an RMA as well as approve applications for federal funding (Russell and Saenz 2004). As RMAs are formed throughout Texas and become more common, the expected impact will influence the future of transportation financing in many of the following ways:

- Create revenue for transportation projects
- Give local governments more control in transportation planning
- Speed up project time lines, relieving congestion sooner
- Improve mobility and increase safety for motorists (Russell and Saenz 2004)

PROBLEM STATEMENT

Although the methodology of the RMA and its functional procedures of debt-financing transportation infrastructure is not a new way to implement projects in the United States, House Bill 3588 and its predecessor, Senate Bill 342, introduce debt financing into Texas as the solution to metropolitan congestion by expediting needed projects and increasing highway capacity. The addition of this innovative financing tool is a major departure from the pay-as-you-go philosophy of the past 88 years. Therefore, it is important to understand the way RMAs function and explore the expected effects on the transportation infrastructure in Texas.

RESEARCH OBJECTIVES

With Texas House Bill 3588 reaffirming its commitment to RMA formation, it is important to understand how RMAs are formed and how they will affect present and future transportation project financing. To better understand the affects of an RMA in Texas, the following objectives have been established:

- Determine the short-term effects of RMA formation and financing
- Determine the long-term effects of RMA formation and financing
- Investigate agency cooperation after the formation of an RMA
- Determine RMA's improvement over the past system
- Describe experiences with the formation of an RMA

BACKGROUND OF TRANSPORTATION FINANCING IN TEXAS

The state of Texas obtains monies for transportation-related projects in many ways. Among the most common are motor fuel taxes, motor vehicle registration fees, and federal reimbursements. The distribution of fiscal year 2003 can be seen in Figure 1 (TxDOT Finance Division 2004).

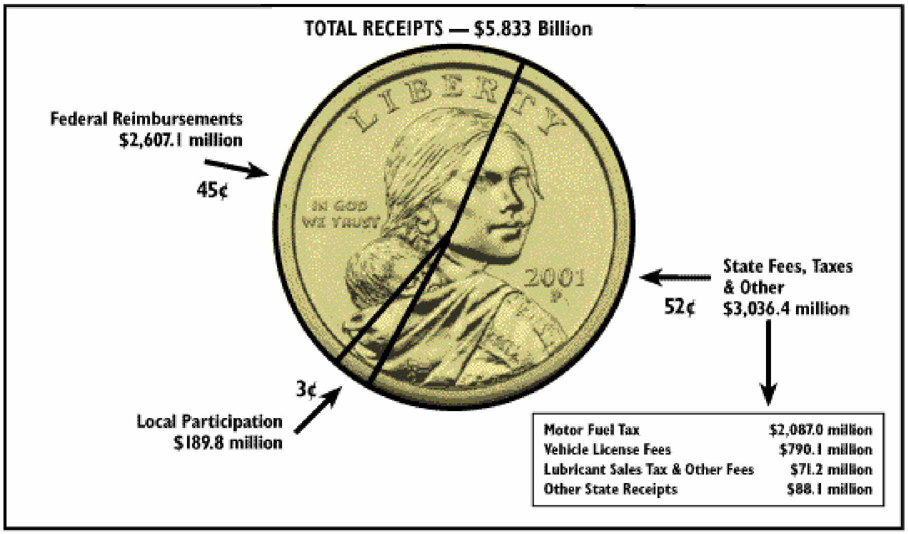

Figure 1. Distribution of total funding sources for fiscal year 2003

When reviewing current fuel tax returns, Texas is a donor state. A donor state is a state that receives less money back than the amount it sent to the Highway Trust Fund. Currently, Texas only receives approximately 88 cents for every dollar that is sent to the fund. In fiscal year 2003, Texas utilized 72% or 2.1 billion dollars of the fuel taxes collected for transportation. The distribution of gas tax usage is shown in Figure 2 (TxDOT Finance Division 2004). The Texas Department of Transportation (TxDOT) is seeking a federal act guaranteeing a 95 cent return per dollar vested as a Texas transportation priority and is currently pursuing maximum returns from fuel taxes (Johnson 2003).

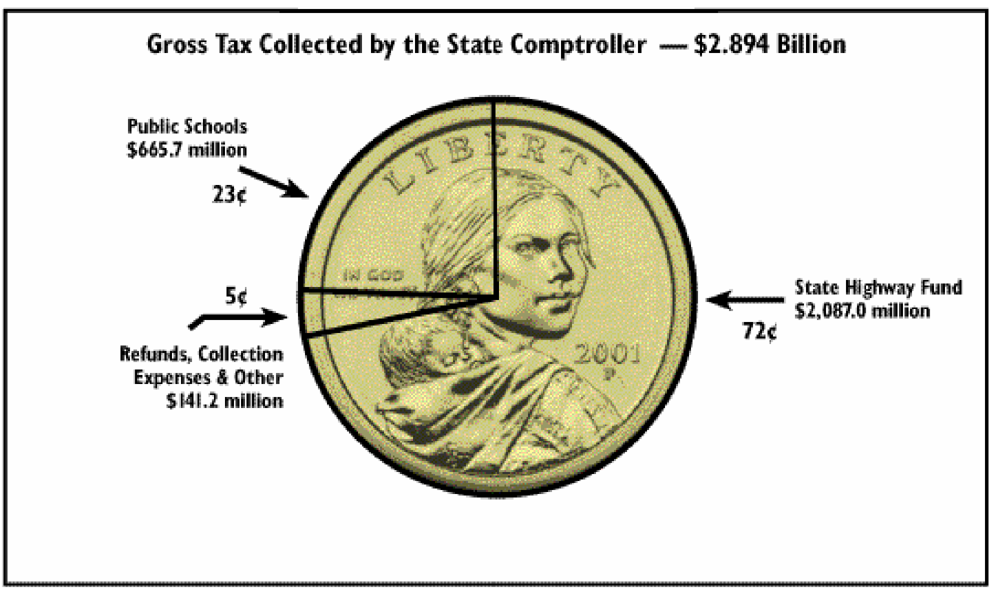

Figure 2. Distribution of Texas motor fuel taxes for fiscal year 2003

Vehicle registration fees collected in fiscal year 2003 amounted to over 1.2 billion dollars, establishing these fees as a significant source of money for transportation projects. Unlike fuel taxes, these monies go directly to the State of Texas for its use. According to Strayhorn (2004), “State law requires all vehicles, with few exceptions, that are operated on public roads to be titled and registered in Texas. TxDOT is the state agency responsible for this titling and registration, and state law designates county assessor-collectors as agents of the state.” The distribution of vehicle registration fees can be seen in Figure 3 (TxDOT Finance Division 2004).

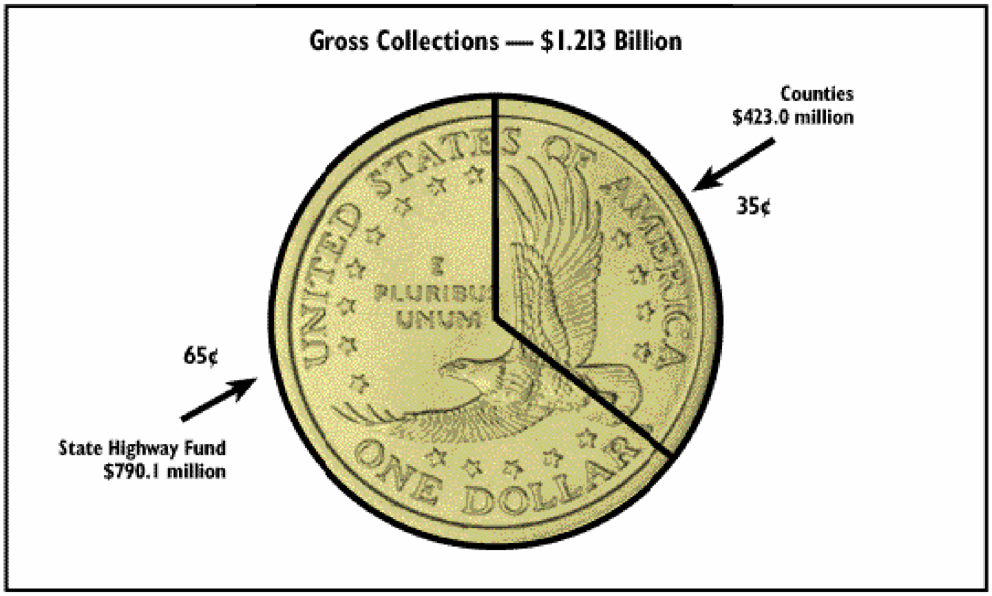

Figure 3. Distribution of Texas motor vehicle registration fees for fiscal year 2003

Another funding option exercised in Texas is Federal Highway Administration (FHWA) reimbursement. The FHWA reimburses 80% of the total cost of a project each month. There are five steps to the reimbursement process, which can be seen in Figure 4 (TxDOT 2004).

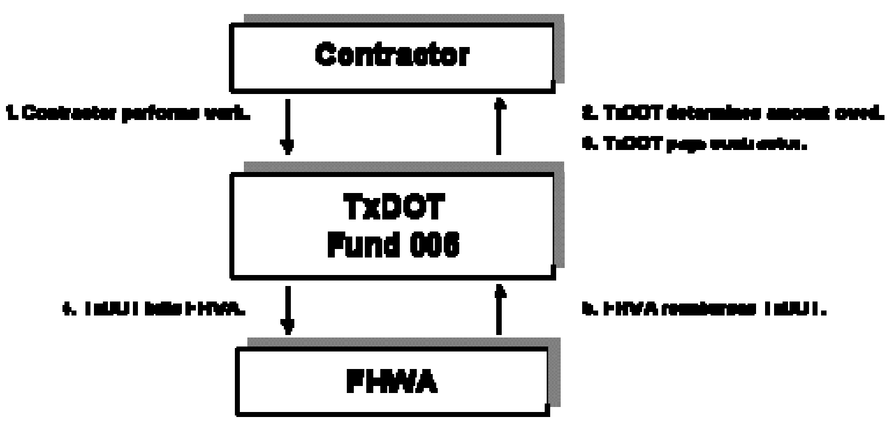

Figure 4. Process for FHWA reimbursement for TxDOT projects

State Infrastructure Bank

State infrastructure banks (SIB), authorized in 1995 by Public Law 104-59 as a part of the National Highway Designation Act, provides an innovative way to finance transportation projects. Part of a federal pilot program, SIBs provide loans at below-market interests rates for infrastructure projects (TxDOT 2004). States that have SIBs are allowed to transfer up to 10% of total federal dollars received, match that amount with state funds, and deposit everything into the SIB, thus creating a self sustaining, growing revolving loan fund (Bass 2001).The map of the United States in Figure 5 shows SIB participation levels (Innovative Finance 2003).

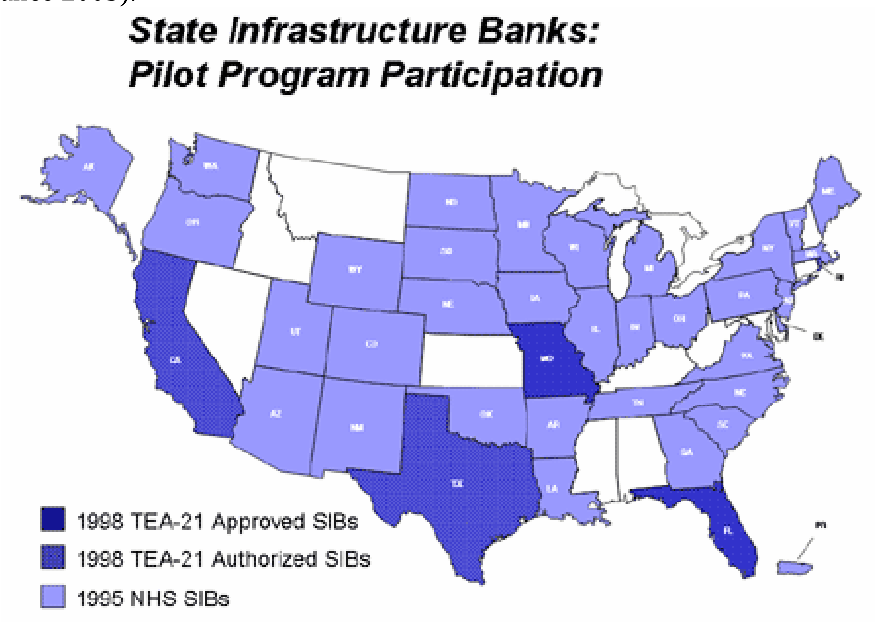

Figure 5. SIB participation

The Transportation Infrastructure Finance and Innovation Act (TIFIA) was developed as part of the Transportation Equity Act for the 21st Century (TEA-21). Under the establishment of this new federal program, the U.S. Department of Transportation (USDOT) provides credit assistance to states for major surface transportation projects of national or regional significance. During fiscal years 1999–2003, TEA-21 authorized up to $10.6 billion in revenue assistance. The TIFIA program's objective is to leverage limited federal resources. The strategic goal of TIFIA is to stimulate capital investments in transportation infrastructure by providing credit rather than grants to projects of importance (Sullivan and Callender 2003).

Toll roads are another way of financing transportation in Texas. In 1953, the Texas Turnpike Act became law, and in 1957 the first toll road in Texas was built, the Dallas-Fort Worth Turnpike (Turnbull 2003). The general timeline for toll authorities and related activities is as follows:

- 1953: Texas Turnpike Act became Law
- 1957: Dallas-Fort Worth Turnpike opened
- 1983: Harris County Toll Road Authority created
- 1997: North Texas Tollway Authority was created by Senate Bill 370
- 2001: Senate Bill 342 created RMAs
- 2003: House Bill 3588 "provided additional authority and created new opportunities for toll facilities" (Turnbull 2003)

Defining RMAs

According to the Texas Tollways website, an RMA is “a local transportation authority that can build, operate, and maintain toll roads.” Alamo RMA in Bexar County describes an RMA as able to “provide the San Antonio area with an opportunity to significantly accelerate needed transportation projects and have a local entity in place that will make its own mobility decisions for the community, while enhancing the economic vitality and quality of life in the San Antonio metropolitan area” (AlamoRMA 2004). Diana Vargas lists acceleration of projects, improvement of mobility, enhancement of economic development, and provision of potential revenue sources as reasons why RMAs continue to form in Texas. TxDOT’s RMA manual defines an RMA as “a political subdivision formed by one or more counties to finance, acquire, design, construct, operate, maintain, and expand transportation projects” (Russell and Saenz 2004).

An RMA is an innovative tool that brings transportation decisions to the local levels and provides local governments the opportunity to accelerate needed projects, improving mobility, enhancing economic development, and providing potential revenue sources through designing, constructing, operating, and maintaining tolled or non-tolled roadways, passenger and freight rail, airports, and pedestrian and bicycle facilities.

Once established, an RMA can generate revenue for additional transportation related projects. The primary purpose of an RMA is to give more control to local governments to accelerate needed projects that may take otherwise longer, in comparison with the previous process. With accelerated projects, congestion relief and increased mobility will be brought to the traveling public faster, increasing safety on Texas roadways (Russell and Saenz 2004).

The creation of an RMA begins at the request of one or more counties. Any county, including one that is part of an existing tollway authority, may form a RMA. The County Commissioners Court, the local governing body, must first authorize the creation of an RMA. The commissioners must recognize that the existence and purpose of the RMA will be for the planning, constructing, maintaining, and operating of transportation projects in their regional area of the state (Texas Statutes 2004).

Petitions for the creation of an RMA are then submitted to the chairman of the transportation commission and reviewed by TxDOT to ensure the application is complete. A detailed listing of the requirements can be found in the TxDOT publication, Regional Mobility Authorities Manual (Russell and Saenz 2004). This process is divided into three basic steps. For the purposes of this paper, a short description of each step is given in Table 1 (Russell and Saenz 2004).

Table 1. Three-step process for petition and approval of an RMA

Step 1: Submit petition to Texas Transportation Commission
• Resolution from commissioners court of each county • Identification of proposed transportation project • Description of impact on regional mobility • Appointment process of board members (i.e., involvement of city, county, or other local government entities in selecting board members
Step 2: Review petition and schedule public hearing
• TxDOT will review petition to ensure all requirements have been met before a public hearing date can be set • County will advertise the hearing in accordance with a public outreach plan developed with TxDOT • Legal notice will be posted in classified section of area newspapers • Hearing information and petition will be posted on county website • Other innovative outreach activities targeting the general public will occur
Step 3: Decision by Texas Transportation Commission
• Sufficient public support based on information for public outreach activities must be present • Benefit to traveling public must be evident • Project must be consistent with local and state transportation plans • Local and statewide mobility must be improved

The transportation commission may deny a petition if it determines the geographic representation and appointment process of the proposed RMA board will not fairly represent political subdivisions affected by its creation (Russell and Saenz 2004).

Each county that is a member of an RMA will appoint an equal number of members to serve on the board of directors, with a minimum of two members. The governor will then appoint an additional member, and this member shall be the presiding officer. Each board member will serve staggered six-year terms ending February 1 of odd numbered years. Members may be reappointed at the discretion of the appointing entity. The terms of no more than one-third of the board members may expire at once.

Other requirements that must be met in order to form an RMA are outlined in the Manual on Regional Mobility Authorities (Russell and Saenz 2004), and detailed in the Texas Transportation Code (Texas Statutes 2004).

Once the RMA is approved, the entity will take on many of the characteristics and authorities of its parent agency, TxDOT. The RMA has been described as an arm of TxDOT for the local area. Some of the authorities that exercised by the RMA are the following:

- Develop transportation projects
- Acquire or condemn property for transportation projects
- Enter into comprehensive development agreements
- Apply for loans from the SIB
- Establish tolls
- Use surplus revenue to finance other local transportation projects, either tolled or non-tolled

RMA Funding Sources

TIFIA and SIBs

In Texas, the Central Texas Regional Mobility Authority (CTRMA) used the TIFIA program to advance the US 183-A corridor, a large capital-intensive project in northern Travis County that otherwise might be delayed or not built at all. CTRMA received $66 million (one-third of the estimated project cost of $200 million) as a direct TIFIA loan (Innovative Finance 2003). Figure 6 indicates the states with current TIFIA and SIB loans for transportation infrastructure projects (Innovative Finance 2003).

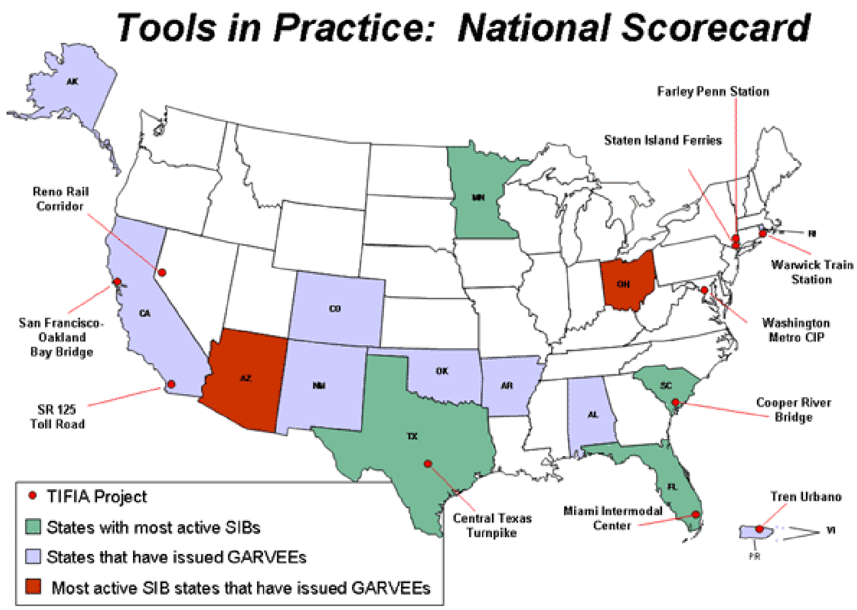

Figure 6. TIFIA loan activities in the United States

Federal Aid

Another source of funding for the RMA that can help finance transportation projects is federal aid provided by Section 129 of Title 23 U.S. Code. This amendment legislated by the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA), allows federal participation in a state loan for a toll project. Section 129 loans allow states flexibility to negotiate interest rates and other terms of the loan (Innovative Finance 2003).

Test and Evaluation Project 045 (TE-045)

Although the North Texas Turnpike Authority is not a RMA, it has taken advantage of another source of funding that could eventually be used to assist in financing future RMA projects. The FHWA uses Test and Evaluation Project 045 (TE-045) to expand transportation infrastructure investments. The program evaluates proposals from states for unique and innovative financing ideas that can be tested for implementation. The President George Bush Turnpike is the first project in the United States to benefit from TE-045 innovative finance provisions (Innovative Finance 2003).

Bonds and Other Monies

Other funding resources, including bonds and other monies, will continue to play an important role in financing future transportation infrastructure projects in Texas. Their main purpose is to get the highway projects built as soon as possible so they may begin to serve the traveling public. However, issuing revenue bonds and establishing toll rates on these projects is expected to be used more frequently. Project selection will be the most important aspect of the RMA. If RMAs are conservative in their selection of highways to construct, revenues and funding for future projects will be secured (Innovative Finance 2003).

Current Status of RMAs in Texas

According to Behren, "The state's regional mobility authorities are picking up steam with transportation projects planned for their areas. We are glad to see that happening and we want to support the efforts they put forth in bringing transportation solutions to the state" (quoted in Cross 2004). There are five currently established RMAs, and a future RMA in Webb County is being review by TxDOT and could be established by the end of the year. The existing RMAs are as follows:

- Travis/Williamson Counties, CTRMA
- Bexar County, Alamo Regional Mobility Authority (AlamoRMA)
- Grayson County
- Cameron County
- Smith/Gregg Counties, North East Texas Regional Mobility Authority (NETRMA)

The following section provides descriptions or lists of projects that were obtained from various RMAs.

Travis and Williamson Counties (CTRMA)

US 138A will be built by the new CTRMA and will bypass Cedar Park and Leander to the east, before rejoining US 183 near Seward Junction (Hurt 2004). Seventy miles of tolled lanes are planned in Central Texas. According to CTRMA (2004), "The approval means the Central Texas Mobility Authority can begin the task of investing more than $2.2 billion in mobility infrastructure and improvements for 13 projects in Williamson and Travis Counties. These improvements will help increase mobility, easing congestion and gridlock on our roads, and can be done more quickly than by using traditional methods."

Bexar County (AlamoRMA)

In planning are "a new interchange with direct connectors at US 281/Loop 1604, an upgrade of the interchange at Loop 1604/IH 10, two new lanes in each direction on loop 1604 from IH 10 to IH 35, and an expansion of US 281 from Loop 1604 to Stone Oak Parkway" (Rios 2004).

Grayson County

An extension of State Highway 289 in Pottsboro is planned. A pass-through toll will be used, which is a "fee the state pays on behalf of individual motorists to public or private entities that have taken on the burden of financing road improvements" (Vaughan 2004).

Cameron County

The West Loop project is planned for Brownsville. Approximately seven miles, it would extend from U.S. 77/U.S. 83 to Palm Boulevard (Rodriguez 2004).

Smith and Gregg Counties (NETRMA)

Loop 49 is planned to be placed around the city of Tyler.

STUDY APPROACH

With the creation of RMAs through Senate Bill 342 in 2001 and additional powers added in 2003 with House Bill 3588, it is important to explore their immediate effectiveness and the RMAs' involvement in future transportation decisions. Feedback from transportation professionals possessing informed opinions and knowledge concerning the RMAs' functional processes is critical to achieving the study objectives. To gain professional feedback on the current status of RMAs in Texas, phone interviews were conducted. The questions were designed to get input from transportation professionals concerning the short- and long-term benefits and problems associated with RMAs; barriers to forming RMAs; the relationship between TxDOT, metropolitan planning organizations (MPOs), and RMAs; and the improvement of current TxDOT practices.

KEY FINDINGS

Twenty-nine professionals were initially contacted for interviewing. Of those contacted, nine responded and eight interviews were conducted. The eight interviews included professionals from TxDOT, MPO offices, RMA boards, and transportation research. The survey responses from the interview questions were then assimilated with information from current literature. Observations were made about each survey question. The following observations are based on the authors' understanding of each survey question.

Short-Term Benefits

For the purposes of this report, the authors are in agreement that the definition of short-term would not exceed five years. This time frame commences whenever the Texas Transportation Commission approves the petition to form an RMA and its board of directors is selected. Many respondents stated that the immediate benefit realized from an RMA comes from the representation of local people on its board of directors. The feeling is that the RMA brings their region one step closer in the transportation decision-making process. TxDOT will continue to address the state's transportation needs within the jurisdictional area of the RMA. The short term-benefit of the RMA is that it can now begin to concentrate on transportation issues that will benefit and possibly stimulate economic growth in the area, rather that just sustain a minimum performance level of the system. Once they are formed, the RMAs' impact on moving projects closer to construction is evident, as shown by CTRMA and AlamoRMA. The CTRMA, with the help of TxDOT, has developed a unified transportation system plan for the region. Included are 10 projects in various stages of development. Currently under construction is US 183A, CTRMA's first entirely new project. Other projects currently under construction by TxDOT will add capacity to some of Austin's better-known highways. After construction is complete, the toll lanes will be managed by CTRMA. The short-term benefits of having numerous projects that will ultimately have the effect of relieving congestion in this region are not typical of the other four Texas RMAs. While the other RMAs in the state all have potential projects to develop, these projects are realistically one to four years away from beginning construction. If the RMAs continue the progress of project development, then the projects may qualify as short-term benefits by the definition set in this paper. These benefits are yet to be realized.

An unexpected short-term benefit has occurred between two competitive counties in northeastern Texas. Historically, political cooperation between Smith and Gregg Counties has been a rare occurrence. It is not unusual for counties to work independently to improve their economic futures and thus improve the quality of life of all concerned. Tyler and Marshall, two major cities that operate as the county seats, are approximately 30 miles apart and have been competing with each other since their establishment. The two local governments have joined together to form the North East Texas Regional Mobility Authority (NETRMA) in an effort to capitalize on the opportunity to control future transportation development in the region. These two rivals have found common ground in forming the RMA and have worked together under the guidance of TxDOT to developed a proposed toll project, Loop 49.

Short-Term Problems

The respondents have identified one major problem that presents a short-term obstacle for RMAs. Start-up funding, also know as seed money, has been mentioned as an obvious and substantial problem for RMAs in general, but the magnitude of the problem seems to vary between each RMA. Upon review of responses to this question, it is apparent that rural RMAs view start-up funding as a substantial problem for them in the short term. One respondent described in her response that there were no initial funding sources inside the county. This fact presents a problem for rural counties who want to form an RMA, because the seed money is almost nonexistent. A different example would be the CTRMA and the AlamoRMA. These RMAs are located in counties were there is a larger, more affluent population, potentially giving them the advantage in the amount of available start-up funds. The answer to this short-term problem comes from TxDOT. One TxDOT respondent made the statement that the Texas Transportation Commission is committed to the success of the RMAs to the extent that it will assist as much as possible in the formation of any viable RMA.

Long-Term Benefits

The respondents agree that the long-term benefits revolve around the generation of surplus revenue from toll projects. With this additional revenue, the RMAs will have a separate pot of money to finance other transportation projects specifically developed to address the transportation needs of the region. The long-term benefits begin to emerge as the RMA exercises its authority to develop a wide range of transportation projects. The revenue supplied by successful toll projects and the resulting flexibility to consider a broad range of projects provide local governments greater control in planning for the needs of the transportation system. The relationship between the RMA and TxDOT should resemble the relationship between TxDOT and the FHWA. The RMA, operating within the policies of TxDOT, will handle the daily operation of the highways under their authority. TxDOT will oversee responsibilities, but stay out of the daily operations. As all these elements work together, the mobility of the area improves congestion relief and increases motorist safety.

Long-Term Problems

If projects are not conservatively chosen and estimated with the highest degree of accuracy, they may not be successful and will result in lost revenues. As previously stated, revenues from toll projects are critical to the future of the RMA. If surplus revenues do not reach their estimated potential, then the RMA will lack the revenue to service the bond debt. Should this occur, then TxDOT will be forced to service the bond debt with funds that otherwise would be dedicated to different projects. Such an event will result in negative financial consequences for both transportation agencies and possibly jeopardize future projects.

Barriers to Forming an RMA

At the local level, the first and most important barrier to forming an RMA is local support. Since the RMA will be established for an indefinite time period to manage transportation, the region must be solidly in favor of its formation. There should be lengthy discussion that addresses all concerns before submitting the petition to the Transportation Commission for approval. With the resources that TxDOT can provide in encouraging and fostering any regions that can support a project, there are few or no barriers to forming an RMA. The process is simple and straightforward. With few or no barriers at the state level, this leaves initial funding or seed money as the only potential barrier to forming an RMA.

Agency Cooperation

MPOs are formed in cities with 50,000 or more in population by a federal mandate, ISTEA. The MPO's primary function is the planning of multimodal transportation within the city limits. MPOs directly control category 2 projects, where federal funds are involved, within their jurisdictional limits. Given this, RMAs will partner with MPOs, providing additional funding that will be used to advance MPO projects that are listed in their long-range plan. If long-range benefits are realized within the RMAs, then MPOs will benefit through the collaboration with RMAs and their funding sources, resulting in faster completion for some MPO projects. This is one of the reasons the state legislators gave the authority for the formation of RMAs. RMAs will be an arm of TxDOT focusing on regional transportation issues. MPOs will not have to plead for funding, but instead can work directly with local officials to solve their transportation issues. This process is the primary relationship between TxDOT and RMAs, relieving TxDOT of the burden of making decisions about regional transportation issues.

Circumventing the MPO Process with RMAs

According to TEA-21 legislation, if a project is built utilizing federal funds, it must be in the local MPO's long-term plan. But on a technicality, if an RMA can fund and successfully build a transportation facility without using federal funds, the MPO process could be ignored. This technicality has some MPO officials worried, leaving them scrambling to see that agreements be in place so that no such thing could occur. The majority of the respondents believe that excluding the MPO process would be detrimental to the success of an RMA. RMAs can benefit from the planning work already completed by the MPOs, which should foster a working relationship rather than an antagonistic one. Although there is potential for bypassing the MPO process with the establishment of surplus toll revenue, the projects that will be undertaken are of such magnitude that the MPO will most likely need to be involved in the process.

Redundancy of RMAs

Although at first one might think that toll authorities and RMAs are redundant, they are very different in two very important aspects: flexibility of projects and the use of surplus toll revenue. For example, the Harris County Toll Road Authority can only develop toll roads and spend excess toll revenues on the highway that generated that revenue. On the other hand, RMAs not only have the authority to construct and fund roadways, but can also authorize projects such as passenger and freight rail, airports, and pedestrian and bicycle facilities. The future of toll authorities, provided that they are managed correctly, will change little and no new authorities will form. It is quite obvious that RMAs will be more attractive to those thinking about forming a mobility agency due to their flexibility in transportation projects and use of generated toll revenues.

Improvement over TxDOT

The most significant improvement over the past system is that regional personnel are making decisions about regional transportation issues. As far as planning, designing, and constructing, there will be little change from the capabilities that TxDOT already possesses. It is also apparent that TxDOT, in the short term, will be providing much needed support to start RMA projects. For example, CTRMA's SH 183A was partially funded by TxDOT. If the current development trend continues with the formation of RMAs throughout Texas, the main purpose of an RMA's existence will be a managerial effort to obtain funding and relieve TxDOT of planning, designing, and maintaining locally desired projects. This should not only relieve TxDOT of the responsibility of addressing local transportation concerns, but give TxDOT more freedom to focus on the much needed infrastructure and maintenance problems facing the state. The new rising concern is the possibility of an RMA folding or no longer being able to sustain an undertaken project. What becomes of that roadway? One assumption is that the state will obtain the folded project at a bargain price. This, however, will return the responsibility to the state. The focus of RMAs should not be to improve TxDOT capabilities, but feel competent in meeting, developing, managing, and maintaining local transportation needs independent of state budgets and decision making.

The Success and Usefulness of RMAs in the Future

Several respondents felt that if RMAs are utilized wisely, they will be a useful tool in solving transportation problems in Texas. Others are not certain at this point. RMAs, theoretically, will allow local projects to be funded, developed, and built faster than possible in the current pay-as-you-go system. The measure of success will ultimately be the relief of TxDOT's responsibility of regional decision making. Only time will tell how effective RMAs will truly be.

CONCLUSIONS

An RMA has two primary functions: to plan, design, construct, and operate toll highways initially and to function as an extension of TxDOT within their respective region, funding and managing many transportation-related facilities. In order for an RMA to be a successful entity, projects must be chosen through careful thought and planning. Research is needed to accurately establish motorists' values of time and wiliness to pay in Texas, and currently feasibility tools are being developed to help with RMA planning, but are not yet proven. As to the future of transportation financing in Texas, it is still not certain the contribution RMAs will make.

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Rural Expressway Intersection Characteristics that Contribute to Reduced Safety Performance

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ABSTRACT

Expressways have been constructed in many states as a way to increase mobility without the expense of a full access-controlled or grade-separated facility. In most cases, it was assumed that these segments of highway would produce mobility and safety characteristics similar to other access-controlled facilities. However, recent research has found that there are problems with the safety performance of these systems associated with conventional at-grade, median-opening intersections. Although past research has been completed to examine the nature of crashes on these facilities, it is the purpose of this study to continue the research and analyze the common characteristics of the intersections. The intersections studied in this research were located throughout the state of Iowa. The objective of these analyses is to identify the major contributing factors that create problematic intersections in the state of Iowa.

From previous research, it is evident that factors in addition to roadway volume contribute to the safety performance of an at-grade, two-way, stop-controlled expressway intersection. This research identifies common characteristics that may increase or decrease the safety performance of a rural expressway intersection. The methodology used in this research includes the examination of 644 intersections throughout the state of Iowa. Through the use of a statewide database and crash information from 1996 to 2000, we were able to identify the 100 best and 100 worst performing intersections based on crash severity rate. For the 200 intersections, a statistical analysis was completed to determine the effects of intersection design and location has on safety performance. The safety performance of vertical/horizontal curve location, intersection skew, and land use were studied to determine the effects on rural expressway intersections.

Following the completion of the analysis of the 200 intersections, 30 intersections with the highest crash severity index rates were selected for more thorough site-specific analysis. As part of this analysis, we examined the impact of land use adjacent to the intersection and the impact of peaking in hourly traffic volumes. The research identifies attributes that impact crash severity both negatively and positively. Through the identification of these attributes, designers and planners can more adequately address safety concerns on rural expressway intersections.

Key words: intersection—rural expressway—safety performance

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

Rural expressways are typically four-lane, high-speed facilities. Rural expressway intersections are generally two-way, stop-controlled facilities. These intersections are often grade-separated or signalized near urban centers or at intersections with primary highways. Maze, Hawkins, and Burchett (2004) recently reported that there are problems with the safety performance of these rural expressway systems. The purpose of the present research is to compare and contrast the characteristics of rural expressway intersections that exhibit poor and good safety performance.

Through observations, Maze, Hawkins, and Burchett (2004) concluded that factors other than roadway volume contribute to the safety performance of a rural expressway intersection. Using a limited dataset (10 intersections) they speculated that design features at intersection approaches, including horizontal curves, vertical curves, intersection skew, and land use intensity, increase crash frequency and crash severity. These features will be referred to as intersection features of interest.

To further examine intersection features of interest, a database of expressway intersections was created. This database includes 644 at-grade intersections. From this set of intersections, the 100 intersections with the best safety performance and the 100 intersections with the poorest safety performance were identified for a comparative statistical analysis. After completing the analyses of these 200 intersections, the 30 worst performing intersections were examined in greater depth. Through the intersection analyses, common characteristics were identified that contribute to the safety performance of rural expressway intersections and the authors were able to make recommendations based on the findings.

This paper is organized into four sections. The first section is this introduction. The next section outlines a descriptive and statistical analysis of 200 intersections with an examination of common trends in crash rates and types. Both intersection alignment and land use are examined. The third section includes an analysis of the 30 intersections with the poorest safety performance. More thorough data collection was performed to provide more resolution to the analysis of these 30 intersections. The final section of this paper includes a summary of characteristics that significantly contribute to the reduction in safety performance of expressway intersections. This paper also includes recommendations for the improvement of expressway intersection design and suggestions for future expressway research.

RURAL EXPRESSWAY CRASH ANALYSIS

This section of the paper performs descriptive graphical analysis of the intersection crash database and uses the results of the graphical analysis to support the specification of a statistical model of intersection safety performance.

Database Development

A GIS-based rural expressway database was created to allow for easy access to crash information. Records from the following five databases were combined to create the expressway database:

- Iowa DOT Roadway Inventory Database
- Iowa video log imagery
- Iowa Department of Natural Resources color infrared imagery
- Iowa Department of Natural Resources land coverage imagery
- Iowa DOT crash record database (accident location and analysis system: ALAS)

For the analysis, the research team analyzed at-grade, two-way, stop-controlled expressway intersections. All of the analyzed intersections shared the following criteria:

- Located on a multi-lane, non–interstate-divided facility
- Partially access-controlled
- Two-way stop controlled

After the intersection database was completed, crash information was added through the crash record database using a buffer radius of 150 feet. Crash records are included from 1996 to 2000. This five-year period was selected to ensure consistency with other expressway intersection research completed in Iowa by Maze, Hawkins, and Burchett (2004).

Additional information was required to maintain the accuracy of the database. Additional visual inspection of aerial photography and the Iowa DOT's roadway inventory and personal observations of intersections were required to populate the database with additional data regarding features of the intersection, such as the skew between the expressway and the intersecting roadway, horizontal curve locations, vertical curve locations, and the land use adjacent to the intersection. This information was added to the over 100 other attributes already contained in the expressway database.

In total, the original database included 644 expressway intersections. Between 1996 and 2000, 327 of those intersections observed crashes. An initial query of the database allowed us to create an intersection severity index for each intersection. The simple severity index below was used:

- Fatal injury crash = 5
- Major injury crash = 4
- Minor injury crash = 3
- Possible injury crash = 2
- Property damage only crash = 1

Through the use of this severity index, a crash severity rate was created. From this crash severity rate, the 100 highest and 100 lowest severity intersections were selected from the 327 intersections that had experienced a crash during the study period. This set of 200 rural expressway intersections is the subject of all analyses completed in this section.

Descriptive Analysis of 200 Rural Expressway Intersections

Maze, Hawkins, and Burchett observed that crash rates on rural expressways increase with increasing mainline volumes (2004). The researchers also observed that crash severity increases with increasing minor roadway volume. To determine how the 200 selected intersections rank, an analysis of crash, severity, and fatality rates was completed. The rates were calculated using 1 million entering vehicles for crash rate and severity rate, while 100 million entering vehicles was used for the fatality rate.

Figure 1 compares the high- and low-severity intersections to the average Iowa expressway intersection rates. It is assumed that a comparison of the attributes of the good and poor performing intersections will allow the isolation of characteristics that result in good and bad safety performance. The overwhelming difference in crash rate also exhibits the need to better understand the hazards associated with rural expressway intersections to improve the safety of these intersections.

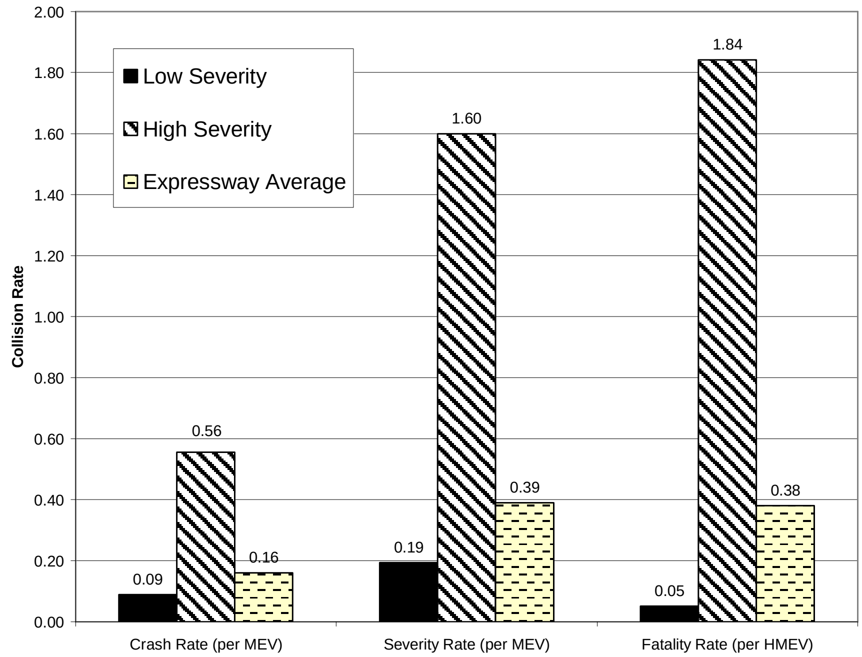

Figure 1. Crash, severity index, and fatality rate comparison

Geometric Features Descriptive Analysis

By using the database, the impacts of geometric features of interest on expressway intersection safety performance were examined. As discussed above, horizontal curve, vertical curve, and intersection skew were all added to the database. Due to limited resources, the information was introduced into the database through feature presence, and a dummy variable was added to indicate the presence of a feature: if the intersection was located on a curve or the intersection was not perpendicular to the expressway route, the dummy variable representing each feature was set equal to one, and if the feature was not present, the dummy variable was set to zero. This method allowed an analysis of the intersection safety performance with respect to a feature of interest.

The intersections were divided into four types: intersections located on a vertical curve, intersections located on a horizontal curve, intersections with non-perpendicular minor legs (skewed intersections), and intersections on a tangent. Some intersections included multiple geometric features of interest, so Figure 2 includes a total larger than the number of intersections in the dataset. Figure 2 describes the count of each intersection type observed at both high- and low- severity intersections. Notice that half of the low-severity intersections lie on a tangent, whereas a majority of the high-severity intersections have one or more geometric feature(s) of interest.

Figure 3 shows the crash rates observed for each geometric feature of interest at high-severity index locations. Notice that all of the intersections have similar crash rates, but that severity and fatality rates increase on vertical, horizontal, and skewed locations when compared to tangent intersections. Intersections on horizontal curves and non-perpendicular intersections have a higher fatality rate than intersections on vertical curves, and intersections on horizontal curves have a fatal crash rate 50% greater than intersections on tangent sections.

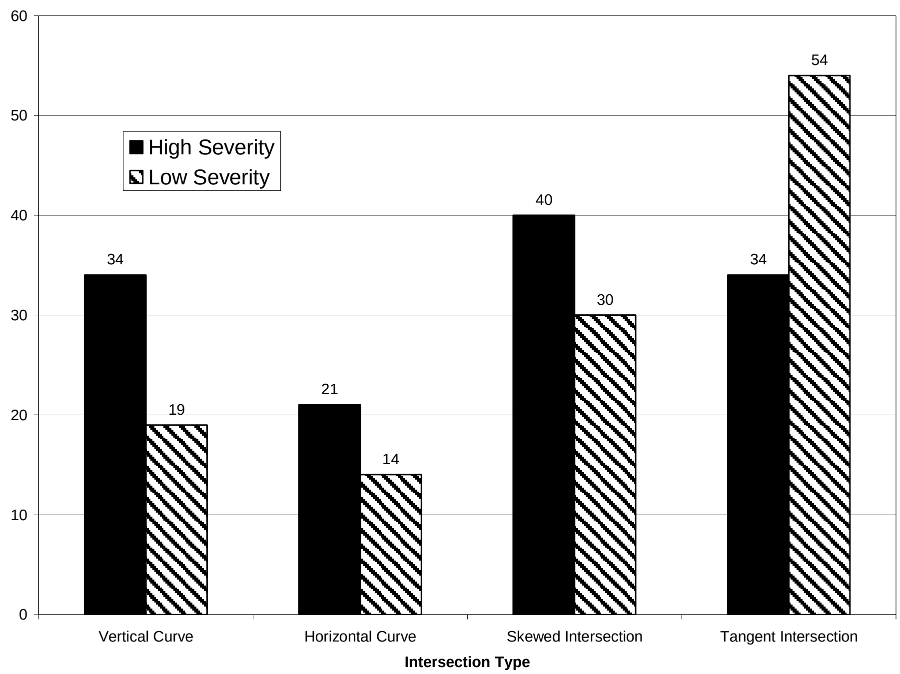

Figure 2. Intersection type distribution

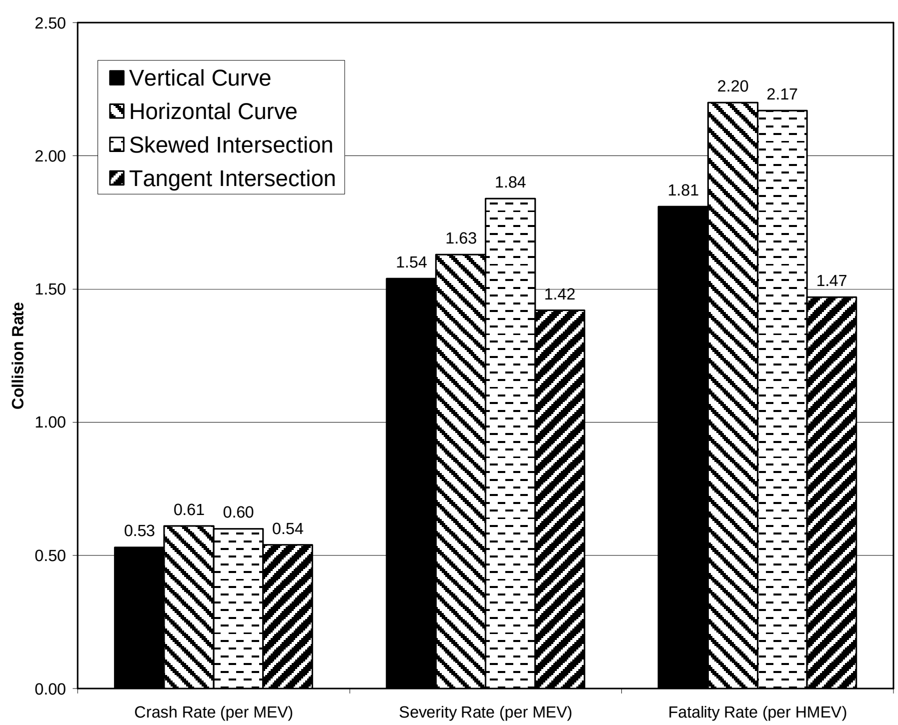

Figure 3. High-severity geometric location crash, severity, and fatality rates

The results of an identical analysis for the low-severity index locations are presented in Figure 4. Again, observe that skewed intersections tend to have more severe crashes than intersections with other features.

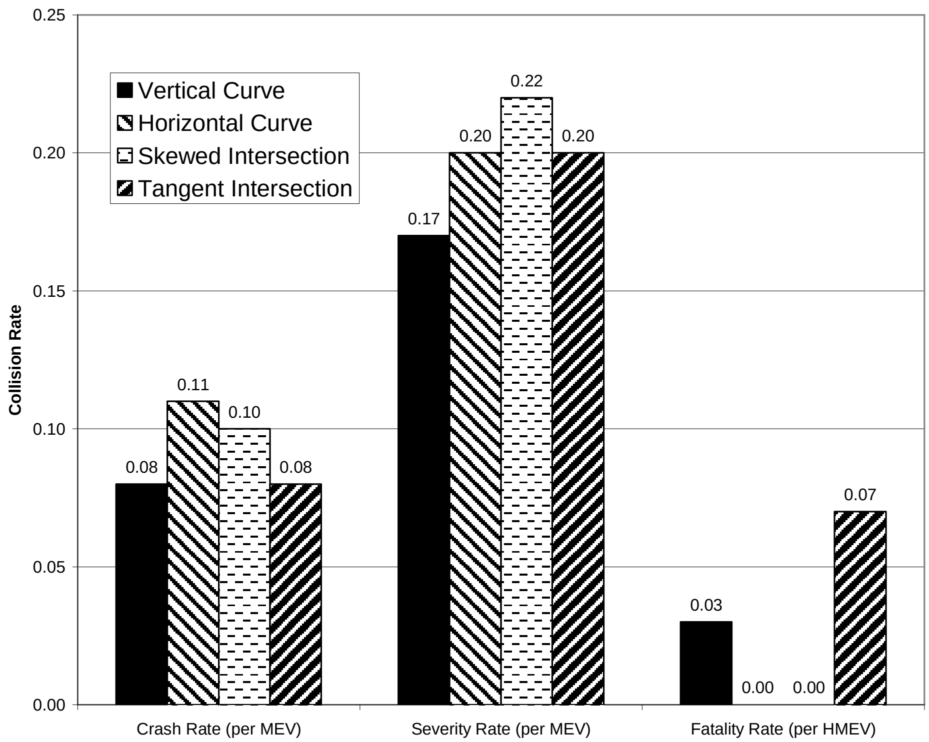

Figure 4. Low-severity geometric location crash, severity, and fatality rates

Additional analysis was completed to determine the effects that geometric features of interest have on intersection safety. Specifically, an analysis of crash type was completed to discover any trends that might relate to an increase in the severity or fatality rates shown in Figures 2 and 3. To remain consistent with previous research, the crash types were grouped into four categories: head-on, right-angle, rear-end, and sideswipe.

Figures 5 and 6 represent the crash distributions at high- and low-severity index intersections. Observe that almost 60% of the crashes occurring at both high- and low-severity index intersections are right-angle when a geometric feature of interest is present. These right-angled crashes are generally the most severe crashes and account for the increased severity rate observed in Figures 3 and 4. The tangent routes observe 25% fewer right-angle crashes than other geometric features of interest. Tangent intersections experience a higher percentage of rear-end or other crash types (single vehicle–fixed-object crashes).

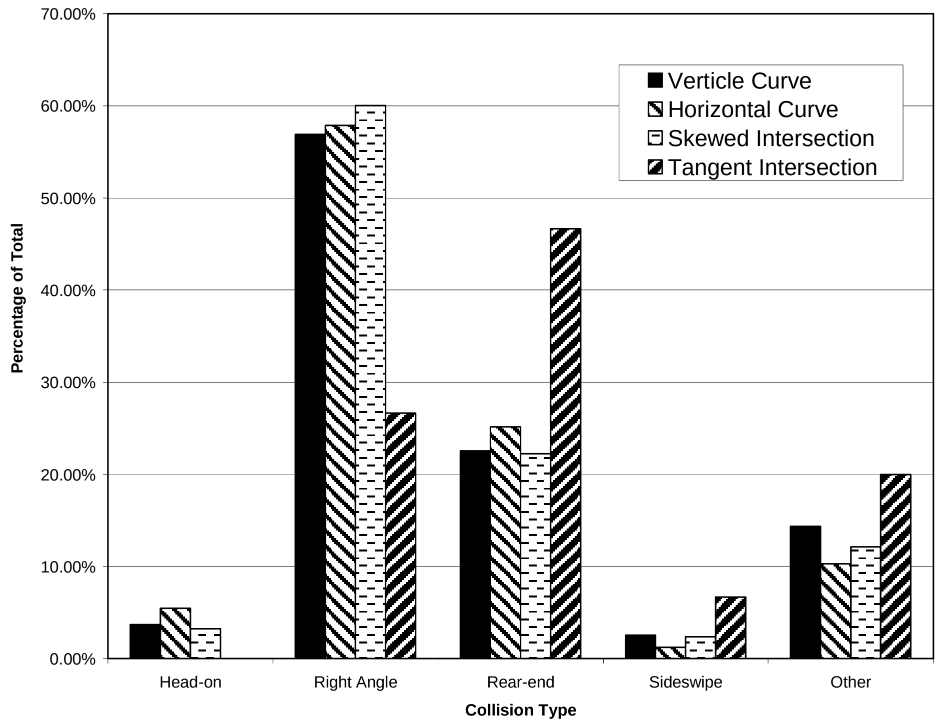

Figure 5. High-severity intersection crash type

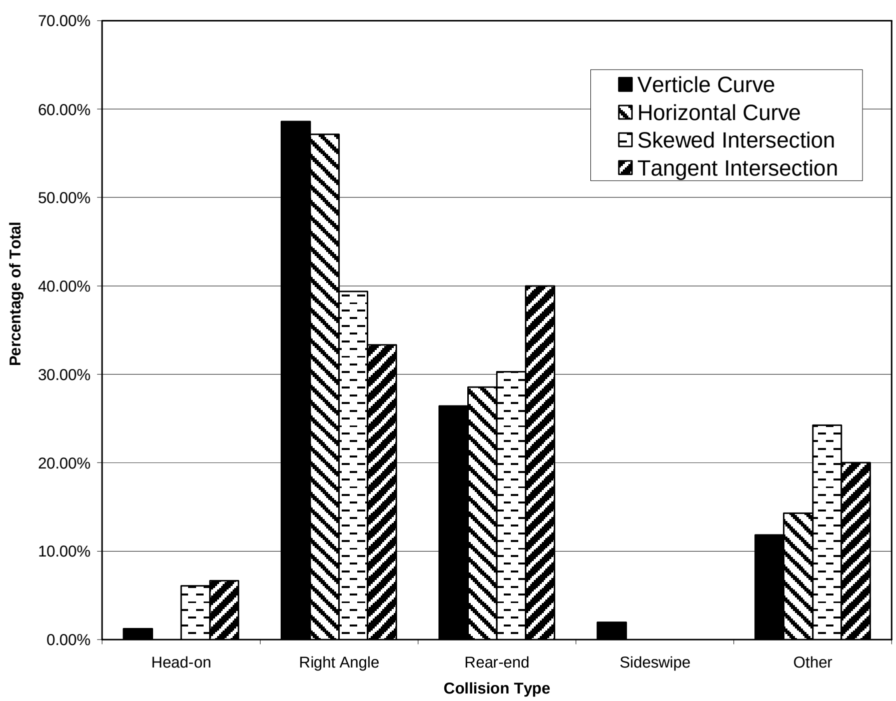

Figure 6. Low-severity intersection crash type

To complete the descriptive analysis of the geometric features, the type of injury accident associated with each type of intersection was examined. Figure 7 shows the proportion of crashes by severity type for each geometric feature of interest at high-severity intersections. A similar analysis for low-severity intersections was not done because the small number of crashes at these intersections does not provide meaningful information. At high-severity index locations, rather than at the tangent locations, more minor and major injury crashes occur where geometric features of interest are present.

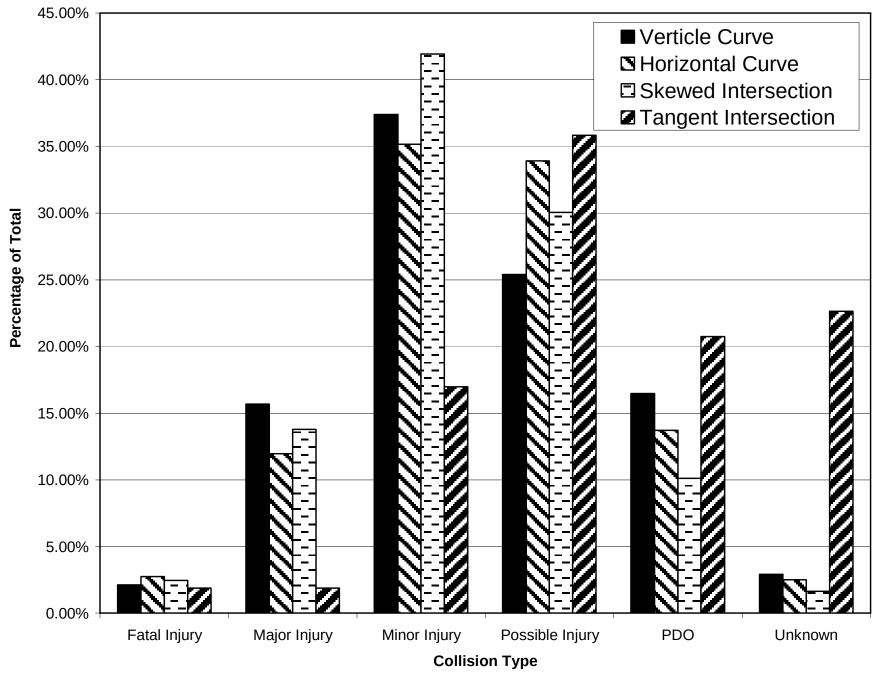

Figure 7. High-severity intersection injury distribution

Both the increased number of right-angle crashes and higher crash severity indicate the reduced safety performance created by geometric features. It would appear from this descriptive analysis that intersection skew and vertical and horizontal curvature reduce the intersection safety performance when compared to intersections tangent to the expressway.

Land Use Descriptive Analysis

In this section, the presence of land use type in relation to expressway intersection safety is examined. Specifically, three types of land use adjacent to the intersection are examined: agricultural, commercial, and residential. To determine the land use adjacent to intersections, aerial photographs of the intersection were used and the percentage of the land cover for each land use within one mile of the intersection was determined. The predominant land use was then indicated as the intersection land use type. Most intersections were surrounded by one type of land use, so discriminating among different land use types was generally not difficult.

Figure 8 shows the distribution of land use types among the intersections. This figure is a raw count. Notice that a majority of the low-severity intersections are bordered by agricultural land use, whereas the high-severity locations are bordered by residential or commercial land uses.

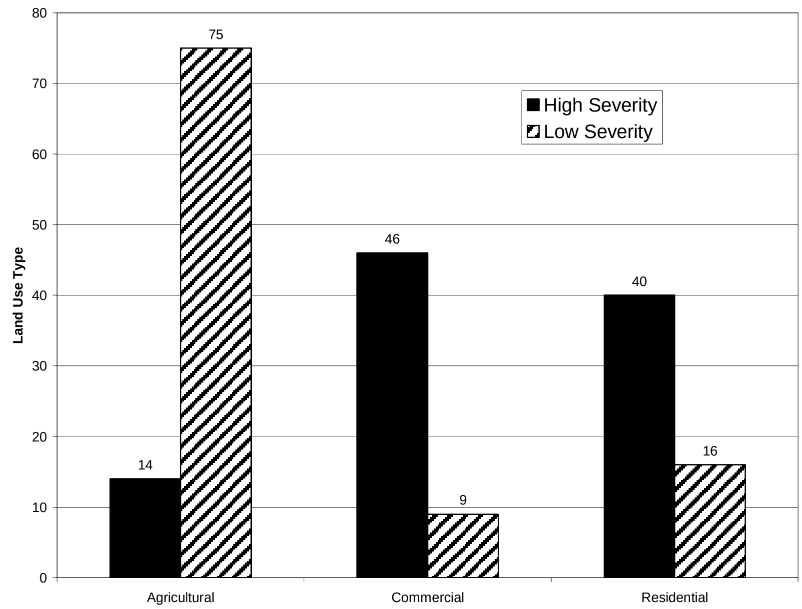

Figure 8. Land use type distribution

The crash rate, severity rate, and fatality rate for the high-severity index locations, with respect to land use, are plotted in Figure 9. Because of the low number of crashes, the same graph is not shown for the low severity intersections.

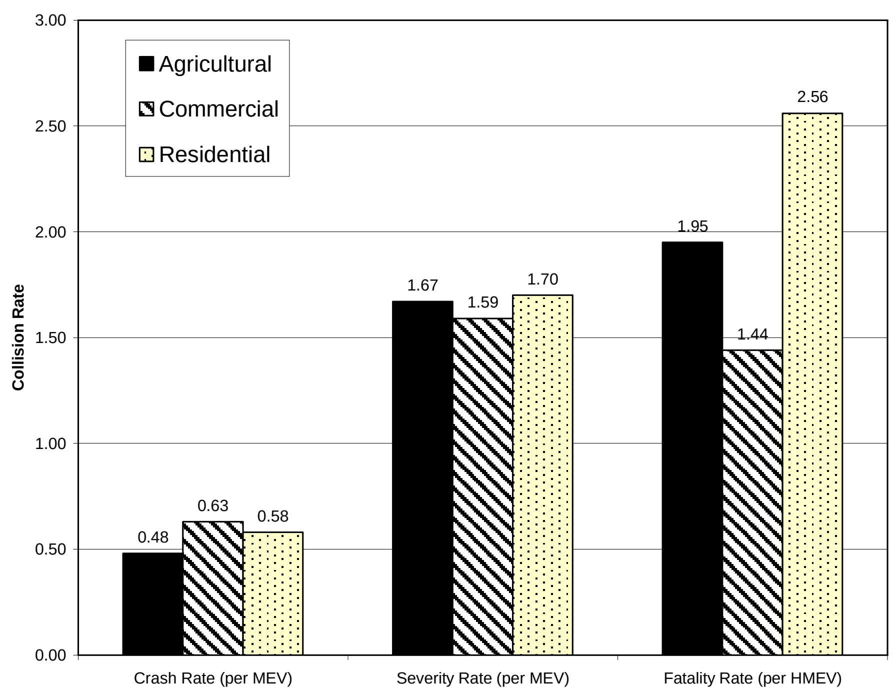

Figure 9. High-severity intersection land use rates

Intersections located adjacent to residential land use have a much higher fatality rate than those adjacent to agricultural land use. The fatality rates for commercial land use and residential land use are 50% and 75% greater than the fatality rates for agricultural land use. This is probably in part due to higher minor roadway volumes in developed areas, but it is speculated that the very high fatality crash rates from residential areas are partly due to the peaking into and out of residential developments as commuters travel to and from work.

Geometric Features and Land Use Statistical Analysis

The estimation of safety performance functions (SPF) was completed using the software package *LIMDEP* version 7.0. This program allowed the authors to run a negative binomial model that allows for over-dispersion within the dataset. Over-dispersion is generally evident in crash data. The crash severity index was tested to determine the best model. Each model represents the five-year total crash severity observed at each intersection. The SPF modeling statistically determines the interaction between intersection crash frequency and independent variables (i.e., approach volumes, land uses, and geometric features of interest).

As determined in previous research, both major and minor roadway volumes significantly affect crash frequency and severity. Therefore, the analysis includes both major and minor roadway volumes as independent variables of the SPF. A Rho-squared value was calculated to determine the goodness-of-fit value of the model. Similar to an R-squared value, the Rho-squared value ranges from 0.0 to 1.0 and measures the model's ability to account for the variance in the dependent variable. The closer the value is to 1.0, the better the model represents the dataset. Also included in each equation is the statistical significance of the parameter estimate. This is known as the p-value, which can be observed in parentheses below each variable.

Several models with different combinations of the land use and geometric features of interest were estimated, but the best model specification is shown in Equation 1. Horizontal curve and mainline volume were dropped as independent variables due to lack of statistical significance in their parameter estimates. The authors were surprised that horizontal curvature dropped out of the analysis. It is speculated that mainline volume tended to be collinear with intensity of land use and including the land use variable resulted in the mainline volume becoming statistically significant. Remaining in the model are skewed intersections, vertical curvature, and commercial and residential land use. Each of these variables' parameter estimates are very statistically significant. Also, the high Rho-squared value of 0.558 demonstrates that this model is extremely high for a model of this type.

$$\text{Crash Sev} = e^{(1.683 + (0.00016*M2) + (0.59910*S) + (0.5988*V) + (0.7762*C) + (0.5896*R))} \qquad (1)$$

$$(0.00001)(0.6217) \quad (0.00001) \quad (0.0001) \quad (0.0011) \quad (0.0001) \quad (0.0001)$$

Rho-Squared Value = 0.558

where

Minor roadway volume = M2
Intersection skew = S
Vertical curve = V
Commercial land use = C
Residential land use = R

RURAL EXPRESSWAY ANALYSIS OF 30 WORST PERFORMING INTERSECTIONS

Through observations made in the descriptive and statistical analysis, it was evident that additional research was needed to increase the resolution of this examination of the effects of both geometric features of interest and land use variables on expressway intersection safety performance. A sample set of the 30 intersections with the poorest safety performance was selected and additional information on crash, volume, and intersection geometric features of interest was collected. This involved inspecting each of the 30 intersections to collect specific information on land use and geometric features of interest. These data were then used in a crash analysis to determine impact of these variables on safety performance.

Hourly Volume Analysis

Although residential development serves as proxy for peak volumes, more detailed analysis of hourly volumes was needed to determine the actual impact of peaking on safety performance. With the assistance of the Iowa DOT, hourly volumes were obtained for all 30 intersections. An hourly count was obtained for each of the 30 expressway intersections with a count taken for 24 hours on a Tuesday, Wednesday, or Thursday. These volumes were then used to determine the morning and evening peak hours. On average, the morning peak occurred between 6 a.m. and 9 a.m., while the evening peak volumes occur between 4 p.m. and 7 p.m. Once the peak hours were determined for each intersection, crash information was extracted from the expressway intersection database. The crashes at each intersection were then calculated for a peak hour crash percentage versus off-peak hour crash percentage. It was determined that 51.75% of the accidents at the sample set intersections occurred during the peak hours. These peak hour volumes averaged 45.20% of the total daily traffic volume, which indicates that average peak morning and afternoon hours experienced more than 20% of the daily traffic. Typical hourly peaking is in the range of 8% to 12% of daily traffic (Texas Transportation Institute). Therefore, the highest crash rate intersections experience extremely high hourly peaking. Although the sample size is limited, this does seem to indicate that peaking is an important variable.

Near-side vs. Far-side Crashes

Through the INTERSECTION MAGIC version 6.60 software package, each intersection in the 30-intersection set was analyzed to determine whether each crash occurred on the near lanes of the expressway to the minor approach or the far lanes of the expressway. Crashes were grouped into three categories: near-side, far-side, and other crashes. The “other” category is limited to single-vehicle, rear-end, or fixed-object crashes. Due to the limitation of the available data, only information from 1996 to 1999 was used; however, four years of data should be sufficient to minimize the impact of random spikes in crash activity.

Figure 10 shows the distribution of near- and far-side crashes for all 30 intersection. Note that almost 50% of the crashes that occurred at the 30 intersections were far-side crashes. The ratio of near-side to far-side crashes is 62% to 38%. A far-side crash indicates the crossing driver misjudged the gap in the far-side lanes.

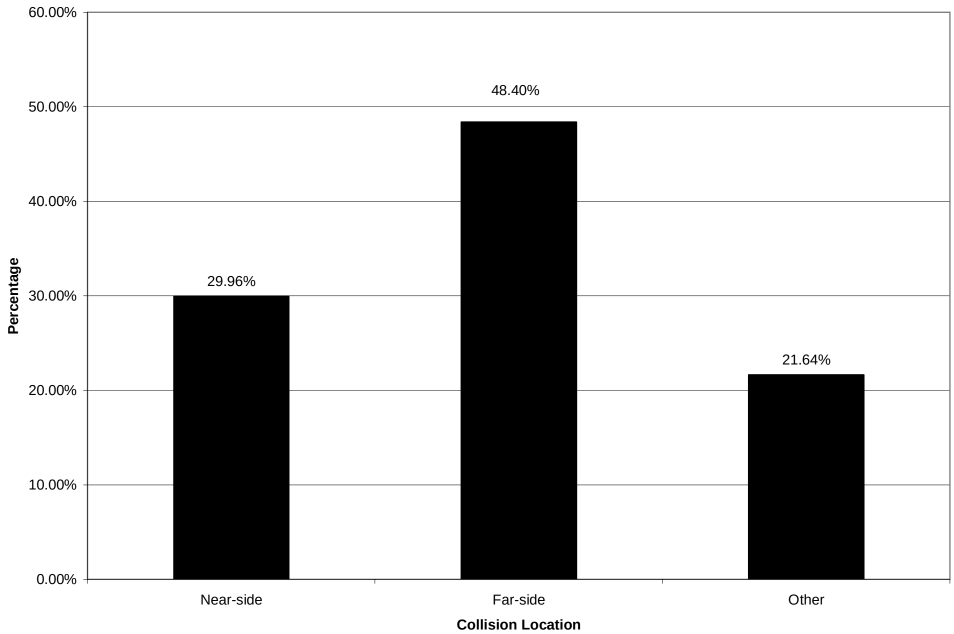

Figure 10. Sample set intersection near-side vs. far-side crash distribution

As discussed earlier in the paper, intersections on horizontal curves seem to perform differently than intersections on vertical curves or skewed intersections, and an analysis of these intersections was completed to determine possible differences within the data. Of the 30 intersections researched, 7 were located on a horizontal curve. All seven intersections were not located near major commercial or residential development, but were typically part of a bypass. These horizontal curves were determined to have 3% of curvature or more per 100 feet of expressway. In each case, the intersections were four-legged, but observed a higher volume on one of the minor legs than on the other. An analysis of these seven intersections was completed to view the possible difference in the intersections. Figure 11 shows that horizontal curves do not observe a roughly equal distribution of far-side or near-side crashes. Also note that when horizontal curve locations are removed from the remaining intersections, the far-side crash percentage increases. This is interpreted to mean that drivers crossing the expressway have difficulty even judging gaps in the near-side lanes. Pulling out in front of vehicles in the near-side lanes may in part explain the elevated fatal crash rate seen in Figure 3.

Further examination of the horizontal curve locations found that over 60% of the crashes occurred nearest to the minor leg intersection approach with the highest volume. The lower volume minor roadway approach observed 25% of the volume typically observed on the remaining minor leg. Each of the seven horizontal curve locations was similar in design and surrounding land use. From these observations, it is clear that horizontal curves create a unique hazard for drivers. These curves are located throughout the state and are typically found on bypasses around cites. Although this is a small percentage of the total intersections in the state, it is clear that they are unique in that they do not follow the trends of other intersection geometric features.

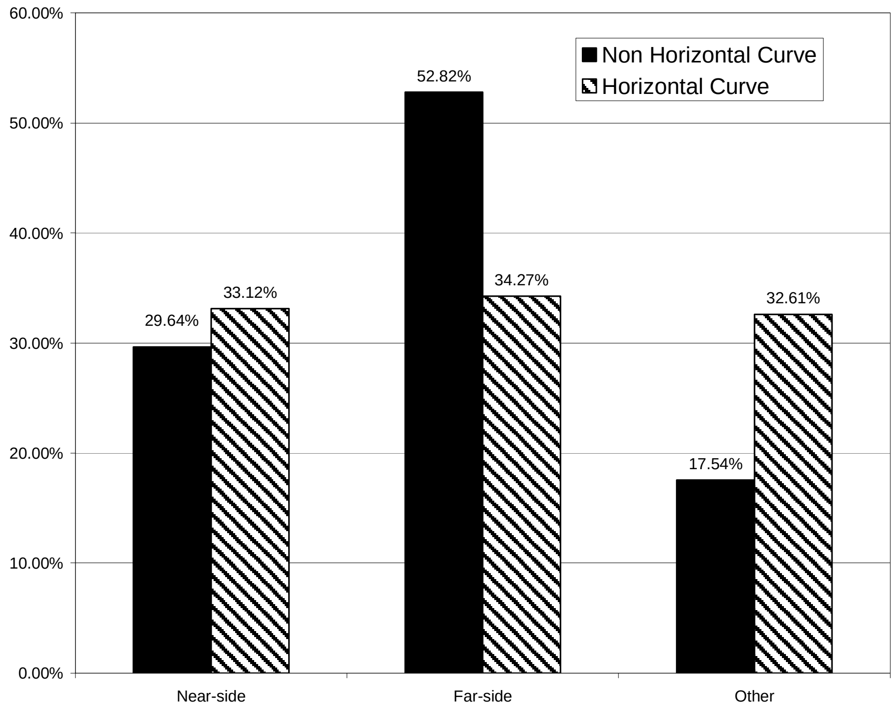

Figure 11. Far-side vs. near-side comparison of horizontal curve intersections

This sample analysis demonstrates that added influences of geometric features and land use on safety performance of expressway intersections. Although a small sample, this analysis demonstrates a need for further research and determination of key influencing factors of expressway crashes. The clear trends shown in right-angle and far-side crashes demonstrate the predictions calculated through the statistical models. Crash severity at expressway intersections is clearly related to intersection design and surrounding land use.

CONCLUSIONS AND RECOMMENDATIONS

Through this research it was observed that safety performance of expressway intersection varies greatly. Much of the variation in safety performance is explained by minor roadway volumes, but some of the variation can be attributed to the expressway curvature at the intersection, skew of the intersection, and land use surrounding the intersection. These features impact both the crash rate and crash severity. Although only a small set of intersections was examined, it appears that judging gaps in the far lanes is most problematic for drivers, except on horizontal curves, where drivers have equal difficulty judging gaps in both the far-side and near-side lanes.

It is recommended that research to address these issues cover detailed analysis of more intersections at locations in the United States other than Iowa. Although we believe that some intersection attributes elevate crash rates, more research is needed to quantify these relationships. With a better understanding of the most problematic design features of expressway intersections, these conditions can be avoided during corridor planning. For existing expressways, safety engineers can begin to address proactively the intersections with features that tend to make them problematic.

ACKNOWLEDGMENTS

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Alternative Crash Severity Ranking Measures and the Implications on Crash Severity Ranking Procedures

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ABSTRACT

Most states currently use ranking procedures to identify intersections with safety concerns. One objective of a state intersection ranking procedure is to provide a reliable start to the prioritization of safety improvements. For efficiency purposes, it is crucial to have a ranking system that identifies intersections with the most severe safety concerns. Otherwise, potentially hazardous intersections could be overlooked.

Crash severity rankings are a part of many state safety management plans and are often used in combination with crash frequency and/or crash rate rankings. While the procedures for calculating crash frequency and crash rate at intersections are straightforward, there is no standard procedure for calculating crash severity. In fact, the methods used to define crash severity vary throughout the nation. Consequently, different approaches can produce different results, despite their common objective.

The research presented in this paper had multiple objectives. The first objective was to introduce and describe a variety of used-in-practice crash severity ranking methodologies utilized by state DOTs. The second objective was to examine three years (1998 to 2000) of Wisconsin intersection crash data to identify or more closely define crash severity at intersections by exploring its relationship with crash type and vehicle damage. The third objective was to develop alternative crash severity ranking methodologies based on crash type and vehicle damage. The new crash severity ranking procedures developed from this work are then presented and compared to the results of four example state DOT ranking procedures.

Key words: crash frequency—crash rate—crash severity ranking—safety management

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

PROBLEM STATEMENT

Crash severity rankings are a part of many state safety management plans and are often used in combination with crash frequency and/or crash rate rankings. While the procedures for calculating crash frequency and crash rate at intersections are well known, there is not a customary procedure for calculating crash severity. In fact, throughout the nation, a variety of methodologies are used to calculate crash severity. Consequently, these approaches can produce considerably different results, despite their common objective.

Another cause for concern with crash severity rankings is the weights typically applied to fatal crashes. Crash severity rankings based on the comprehensive costs of different injury levels (i.e., value loss methods) typically weigh fatal crashes much more heavily than lesser injury crashes. As a result, intersections that observed a fatality typically rank very high on the list. Fatal crashes, however, are a rare event at individual intersections and, nationwide, fatalities only occur in about 0.6% of all traffic crashes (National Highway Traffic Safety Administration 2005). Therefore, in crash severity ranking procedures that heavily weigh fatalities, potentially hazardous intersections that did not observe a fatality in the time period analyzed may be overlooked.

Also, since most crash severity ranking procedures are solely based on the injury outcomes in crashes, they are also subject to the variability associated with injuries caused by a variety of factors. These factors include seatbelt usage, number of occupants, age of occupants, mode of transportation, crash type, speed of impact, extent of vehicle damage, etc. Consequently, the results of a crash severity ranking may be skewed by factors that affect the injury outcomes in a crash, even factors that are not related to the geometry or operations of a roadway.

OBJECTIVES

This research had multiple objectives. The first objective was to introduce and describe a variety of crash severity ranking procedures used in practice by state DOTs.

The next objective was to identify or more closely define crash severity at intersections by exploring the impacts that crash type and vehicle damage have on the injuries sustained by occupants. These two factors were chosen because the United States General Accounting Office (GAO) once concluded that crash type and speed of impact, which has been shown to relate to vehicle deformation (Mackay, Hill, Parkin, and Munns 1993), were the two most important factors related to the injury risk of occupants (GAO 1995). Also, both of these factors are related to the geometry and operations of roadways.

Another objective was to develop crash severity ranking procedures based on crash type and vehicle damage. It was hypothesized that rankings based on these two factors, which are related to the injury risk of occupants and the geometry and operations of roadways, may provide more appropriate crash severity ranking results. The new ranking procedures utilized an approach based on the average injury cost per vehicle.

The last objective was to evaluate and compare the results of the developed alternative ranking procedures and the results of example used-in-practice ranking procedures using a subset of Wisconsin intersection crash data.

RESEARCH METHODOLOGY

The crash severity ranking procedures used by state DOTs were determined through a review of previous research and by contacting state DOTs via email and/or telephone. The ranking methodologies described and evaluated in this study were from states located in the Midwestern United States.

Crash type, vehicle damage, and their impact on the injury severity of vehicle occupants in intersection crashes were evaluated. The database used for the exploration of these two factors consisted of data from reported intersection crashes in the state of Wisconsin from 1998 to 2000. The database contained information for 141,161 crashes, which involved 274,285 vehicles, 407,870 vehicle occupants, and 2,348 pedestrians. Crashes with incomplete descriptive data were not analyzed. The number and types of injuries observed by occupants in vehicles in nine different crash types and in vehicles sustaining five different vehicle damage levels were explored. Crash type was defined as the most harmful event of each vehicle.

Two alternative crash severity ranking methodologies were developed by calculating average injury costs per vehicle for vehicles in nine crash types and for vehicles sustaining five different levels of vehicle damage. The average injury cost per vehicle in each category was calculated as the sum of the 2003 National Safety Council comprehensive injury costs in that category divided by the number of vehicles in that category (see Equation 1) (National Safety Council 2003). The results were rounded to the nearest thousandth dollar.

$$AC = \frac{N \times \$1{,}900 + C \times \$20{,}200 + B \times \$42{,}500 + A \times \$165{,}000 + K \times \$3{,}340{,}000}{Number\ of\ Vehicles} \qquad (1)$$

where AC = Average cost of injuries per vehicle
N = Number of occupants reporting no injuries
C = Number of occupants reporting possible injuries
B = Number of occupants reporting non-incapacitating injuries
A = Number of occupants reporting incapacitating injuries
K = Number of occupants killed

The two alternative crash severity ranking methodologies and four state DOT crash severity ranking methodologies were applied to a subset of Wisconsin intersection crash data containing 1,369 intersections and 25,633 crashes. This sample contained 73 fatal crashes, 10,754 injury crashes and 14,806 property damage only (PDO) crashes. The top 100 intersections resulting from each of the six ranking methodologies were evaluated and compared based on the following measures:

- Similarities between ranking results
- Annual stability
- Identification of severe events

KEY FINDINGS

Used-In-Practice Crash Severity Rankings

Table 1 shows the four used-in-practice crash severity ranking procedures evaluated in this study. These ranking procedures follow two basic methodologies: equivalent property damage only (EPDO) and value loss (i.e., an estimate of the economic loss associated with crashes).

Table 1. Used-in-practice crash severity ranking methods, with applied weights

Ranking Method	No injury	Possible injury	Non-incapacitating injury	Incapacitating injury	Fatality
North Dakota DOT[a] (EPDO, simple)	1	3	3	3	12
Illinois DOT[a] (EPDO, complex)	1	2	5	9	10
Iowa DOT[b] (value loss)	$2,500	$2,500	$10,000	$150,000	$1,000,000
Minnesota DOT[ac] (value loss, reduced fatal)	$4,200	$29,000	$58,000	$270,000	$540,000

[a] Weights or costs applied to each crash based on the maximum injury observed
[b] Weights or costs applied to each occupant based on the injury observed, plus the actual cost of the property damage, or $2,500 if unknown
[c] If three or more fatal crashes occurred at a specific location, the cost applied to the third or any additional fatal crashes is $3,400,000

The EPDO ranking method is a common crash severity ranking approach. In this approach, injury crashes and fatal crashes are assigned weights that are intended to represent their equivalent in PDO crashes. The North Dakota DOT and the Illinois DOT both utilize EPDO ranking approaches; however, their methods are slightly different. The Illinois DOT method (i.e., EPDO, complex) requires more data than the North Dakota DOT method (i.e., EPDO, simple) because it uses five injury categories (Hallmark and Basavaraju 2002), while the North Dakota DOT method uses three (North Dakota DOT 2000).

Another common approach for identifying and ranking intersections is to calculate the comprehensive injury costs. These methods are often referred to as value loss ranking procedures. The Iowa DOT and the Minnesota DOT both utilize value loss methods. One recognized characteristic of value loss methods is that rankings tend to be biased towards locations that have observed a fatality (Hallmark and Basavaraju 2002). This potential bias is a result of the high costs typically applied to fatal crashes. To reduce this bias, the Minnesota DOT applies a cost of $540,000 to fatal crashes (Minnesota DOT 2004). Although this cost is twice that of an incapacitating injury crash, it is far less than the $3,400,000 value the Minnesota DOT has recognized as the actual comprehensive cost of a fatal crash. The Iowa DOT crash severity ranking method does not include a fatal crash cost reduction (Pawlovich 2002). However, the Iowa DOT evaluates five years of crash data and includes an estimate of the actual property damage, both of which may help alleviate the bias towards locations that observed fatal crashes. The Minnesota DOT, North Dakota DOT, and the Illinois DOT all evaluate three years of crash data.

Crash Type

The quantity and types of injuries reported by occupants in more than 274,000 vehicles were explored. These vehicles were categorized into nine different intersection crash types.

Crash Type Frequency

Figure 1 shows the percentages of vehicles in the sampled intersection data by crash type. More than half of the vehicles were involved in angle crashes and approximately one-quarter were in rear-end collisions. Seven percent of the sampled vehicles were in sideswipe/same crashes and 6% collided with a fixed object. The remaining five crash types (i.e., head-on, overturn, non-fixed object, no collision, and sideswipe/opposite) made up the remaining 7% of the vehicle database.

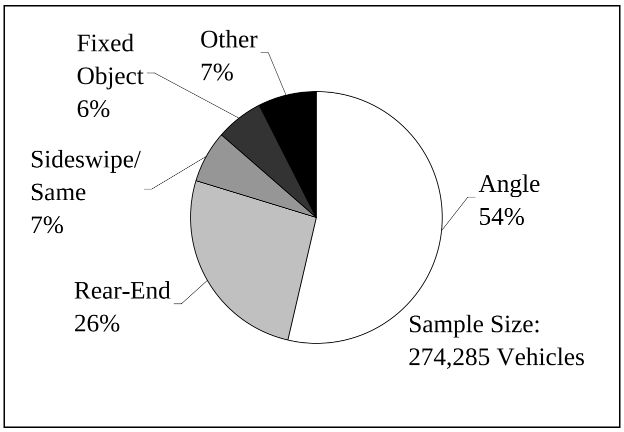

Figure 1. Crash types

Crash Type Injury Outcomes

The percent of occupants by injury and crash type are shown in Figure 2. Overturns and head-on collisions appeared to be the most hazardous crash types. In overturn crashes, nearly 55% of the occupants reported some level of injury and more than 10% of occupants were incapacitated or killed. In head-on collisions, more than 5% of occupants were incapacitated or killed and about 35% of occupants reported an injury of some type. In the remaining seven crash types, more than three-quarters of the occupants did not report an injury and only a very small percentage of these occupants were incapacitated or killed.

More than 75% of vehicle occupants in the two most common intersection crash types, angles and rear-end collisions, did not report an injury. Occupants in rear-end collisions were more susceptible to possible injuries than occupants of angle crashes. However, vehicle occupants in angle crashes were about twice as likely to observe non-incapacitating and incapacitating injuries than occupants of vehicles in rear-end crashes. Occupants were nine times more likely to be killed in an angle crash than in a rear-end crash.

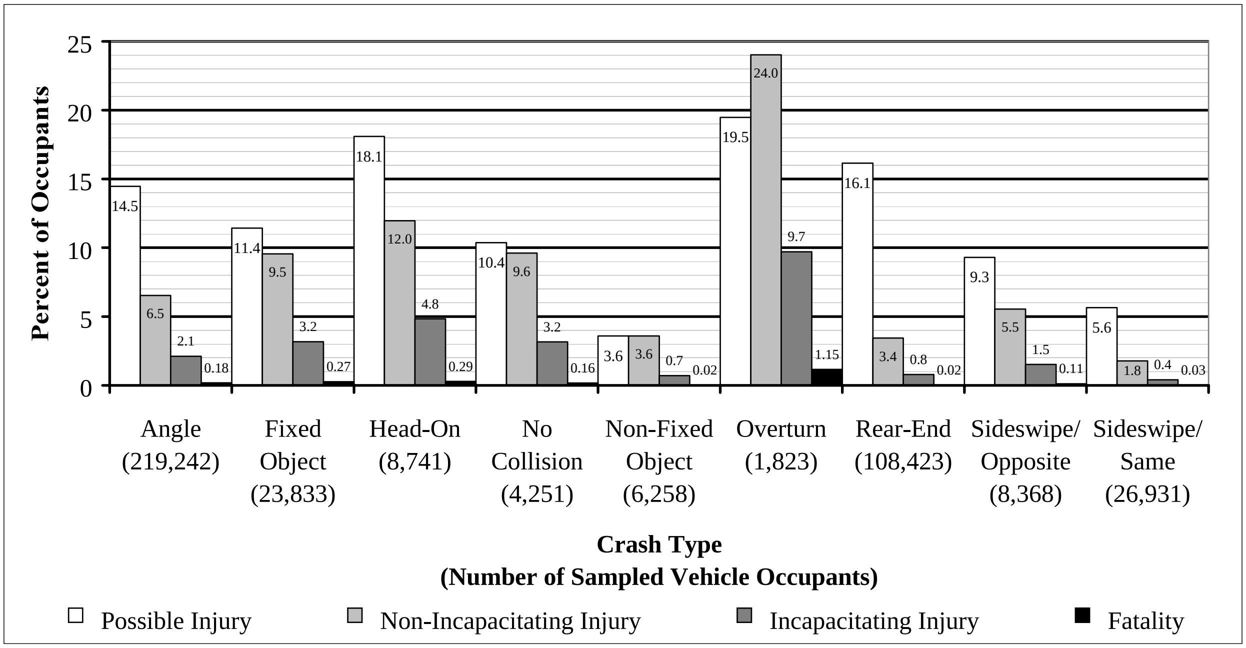

Figure 2. Occupant injury outcomes by crash type

Crash Type Ranking Methodology

The average injury cost per vehicle was determined for vehicles in each of the nine explored crash types. These costs are shown in Table 2 and are the basis for the alternative crash type ranking methodology evaluated in this paper. Vehicles in overturn collisions had the highest average cost at $101,000, which was more than double that of vehicles in any other crash type. Vehicles in the remaining eight crash types had average costs ranging from $8,000 to $41,000.

Table 2. Crash type ranking

Crash type	**Average cost per vehicle**
Overturn	$101,000
Head-on	$41,000
Fixed object	$31,000
No collision	$26,000
Angle	$25,000
Sideswipe/opposite	$18,000
Rear-end	$13,000
Non-fixed object	$8,000
Sideswipe/same	$8,000

It seemed unusual that the average cost was higher for vehicles in non-collision crashes (e.g., fire/explosion, immersion, jackknife, or other) than for vehicles in crashes that involved objects or other vehicles. The individual accident records showed that the severe injuries reported in many non-collision crashes were actually the result of occupants falling out or off of a vehicle. Alcohol played a role in several of those non-collision crashes.

Vehicle Damage

The numbers and types of injuries reported by 407,870 sampled occupants in vehicles sustaining certain damage levels in intersection crashes were explored. Vehicle damage is an input on Wisconsin's accident report form. The descriptions for each vehicle damage level are shown in Table 3 (Wisconsin Division of Motor Vehicles 1998).

Table 3. Vehicle damage descriptions

Vehicle damage	Description
None	No apparent damage to vehicle.
Minor	Damage of cosmetic nature, or vehicle is dented but repairable. Examples: paint scratches, tire scuff marks, bumper rub marks, blown tire(s), broken windshield or window, missing trim pieces, small dents but no creased metal parts.
Moderate	Vehicle quarterpanels are dented or creased. Broken or missing parts can be either replaced or repaired. Vehicle frame or unibody are not damaged. Includes engine compartment fires.
Severe	Vehicle not drivable but may be salvaged.
Very severe	Vehicle is not salvageable. Examples: extensive vehicle damage due to impact of collision, vehicle fire, and vehicle rollover damaging all areas of the vehicle.

Vehicle Damage Injury Outcomes

Figure 3 shows the percent of vehicles by extent of vehicle damage. More than three-quarters of the vehicles sampled were damaged moderately or less. Approximately 17% of the vehicles were damaged severely (i.e., not drivable, but salvageable) and 5% were damaged very severely (i.e., not salvageable).

The percent of occupants by injury outcomes and damage levels are shown in Figure 4. As the level of vehicle damage increased, so did the percent of occupants that reported an injury. Very severely damaged vehicles caused the most harm to occupants. More than 65% of occupants in very severely damaged vehicles reported injuries or were killed. In particular, 2.1% of these occupants were killed and 14.6% reported incapacitating injuries.

Occupants of severely damaged vehicles were less susceptible to injury than occupants of very severely damaged vehicles, but more susceptible to injury than in vehicles damaged moderately or less. Nearly 4% of occupants in severely damaged vehicles reported incapacitating injuries and approximately 0.1% were killed.

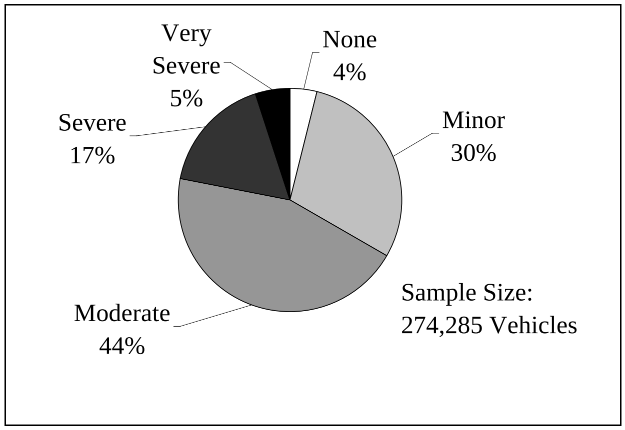

Figure 3. Vehicle damage levels

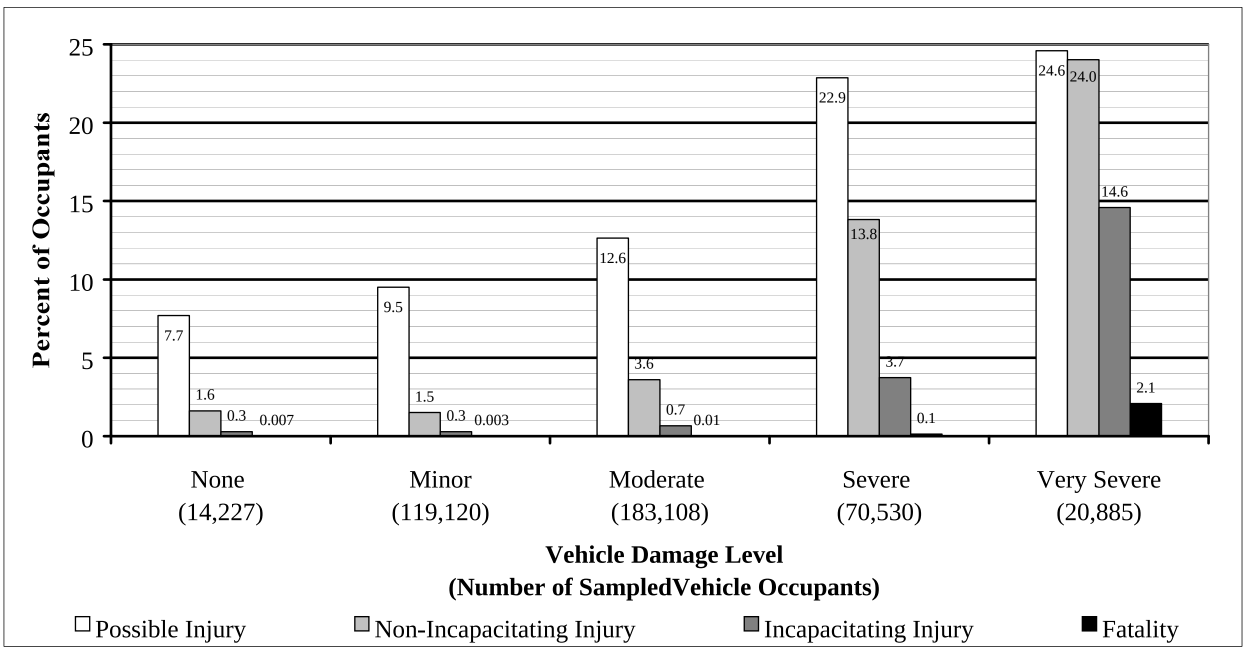

Figure 4. Occupant injury outcomes by vehicle damage

More than 210,000 vehicles were damaged moderately or less, encompassing 78% of the vehicle database. These vehicles generally caused little or no harm to occupants. Less than 0.7% of the incapacitating injuries and less than 0.01% of the fatalities occurred in these vehicles.

Vehicle Damage Ranking Methodology

The average injury cost per vehicle was determined for vehicles sustaining each of the five damage levels. These costs are shown in Table 4 and were the basis for the vehicle damage alternative ranking methodology evaluated in this paper. Very severely damaged vehicles had the highest average cost at $170,000. This cost is more than five times that of severely damaged vehicles, and more than 15 times that of moderately damaged vehicles. Vehicles with minor damage or vehicles not damaged had comparatively low average costs of $7,000 and $6,000, respectively.

Table 4. Vehicle damage ranking

Vehicle damage level	Average cost per vehicle
Very severe	$170,000
Severe	$33,000
Moderate	$11,000
Minor	$7,000
None	$6,000

Pedestrians

Crash type and vehicle damage are not reported for pedestrians involved in crashes. For example, the most harmful event for a vehicle that collided with a pedestrian would likely be coded as a collision with a pedestrian (i.e., non-fixed object). However, there is no crash type input for the pedestrian that was hit by the vehicle. Therefore, an average injury cost per pedestrian was calculated for the 2,348 pedestrians in this sample. The result, $109,000, was higher than the average cost for vehicles in all of the nine evaluated crash types and four of the five vehicle damage levels. The high average cost is representative of pedestrians' elevated susceptibility to injury or fatality when impacted by a motor vehicle. This average cost per pedestrian is to be used in both the crash type and vehicle damage rankings. In the evaluated used-in-practice rankings, all injuries, including pedestrian injuries, are taken into consideration.

RANKING EVALUATIONS

Three tests were conducted to evaluate and compare the results of the two newly developed alternative crash severity rankings (see Tables 2 and 4) and the four used-in-practice crash severity rankings (see Table 1). The tests focused on the top 100 intersections identified by each ranking method using a sample set of Wisconsin intersection data containing 25,633 crashes at 1,369 intersections from 1998 to 2000.

Top 100 List Similarities

The number of intersections that appeared in two top-100 lists produced by two different ranking methodologies were counted. The purpose of this test is to gauge the amount of similarity between the results of two separate crash severity rankings, which in theory have the same objective (i.e., to identify the most hazardous intersections). The results of this test are shown in Table 5.

The two top-100 intersection lists that showed the most similarities were the lists produced by the North Dakota DOT and the Illinois DOT ranking methods. Ninety-one of the same intersections appeared in the top-100 lists produced by each of these EPDO methods. More than 80 of the same intersections appeared in the crash type ranking's top-100 list and also appeared in the North Dakota DOT or the Illinois DOT top-100 lists. Approximately 60 of the same intersections appeared in the vehicle damage ranking's top-100 list and in the other rankings' top-100 lists, except for in the Iowa DOT list, in which only 36 of the same intersections appeared.

Table 5. Top-100 list similarities (intersections similar among the top-100 lists)

Ranking method	Vehicle damage	North Dakota DOT (EPDO, simple)	Illinois DOT (EPDO, complex)	Iowa DOT (value loss)	Minnesota DOT (value loss, reduced fatal)
Crash type	59	83	81	26	56
Vehicle damage		60	63	36	60
North Dakota DOT (EPDO, simple)			91	28	58
Illinois DOT (EPDO, complex)				33	70
Iowa DOT (value loss)					56

The Iowa DOT top-100 list showed the least amount of similarity to other methods' ranking results. Only 26 to 36 intersections appeared in both the Iowa DOT's top-100 list and in another ranking method's top-100 list, except for the Minnesota DOT top-100 list, in which 56 of the same intersections appeared.

Top-100 Annual Stability

The purpose of this test was to evaluate a measure of ranking stability by ranking the intersections using only one-year periods of crash data and then determine whether a ranking method produced consistent top-100 intersection lists. The sampled intersection data was ranked by each of the six tested ranking methodologies three times, once for each year of crash data. Each ranking method's top 100 intersections in 1998, 1999, and 2000 were determined. The intersections that appeared in the 1998, 1999, and 2000's top-100 list were counted. The results of this annual stability test are shown in Figure 5.

The crash type ranking appeared to be the most stable ranking methodology on an annual basis. In this ranking, 53 intersections maintained a position in the 1998, 1999, and 2000 top-100 lists. The North Dakota DOT and the Illinois DOT ranking results were less stable than for the crash type ranking, but more stable than the vehicle damage and value loss ranking methods. In the Iowa DOT and the Minnesota DOT ranking results, only four and seven intersections appeared in each of the methods' three annual top-100 lists, respectively.

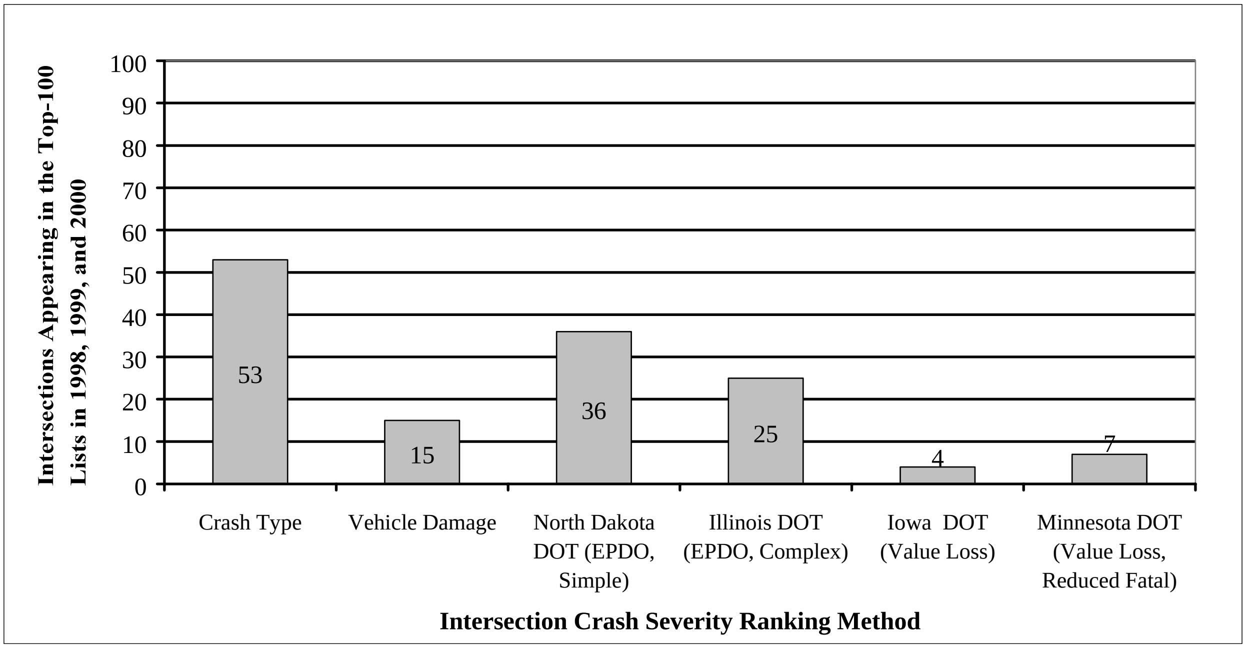

Figure 5. Results of the top-100 annual stability test

Top-100 Severity Statistics

The total number of severe events that occurred in the top-100 intersections for each ranking methodology over the three-year period was determined. The purpose of this test was to evaluate the ability of a ranking to identify intersections that observed crash events that caused considerable harm to occupants. The three severe events evaluated in this test were injury crashes, fatal crashes, and very severely damaged vehicles. Table 6 shows the number of these events that occurred in the top-100 ranking intersections identified by each ranking methodology.

In five of the six ranking methods, more than 2,000 injury crashes were identified. However, the Iowa DOT ranking method, which heavily weighs fatalities, identified only 1,250 injury crashes.

Although the Iowa DOT ranking method was the least effective method for identifying intersections with high numbers of injury crashes, it identified 70 of the 73 fatal crashes that occurred in the subset of intersection data. This was considerably more fatal crashes than was identified by any other ranking method. The Minnesota DOT ranking method identified 31 fatal crashes, and the remaining methods only identified 8 to 13 fatal crashes.

Very severely damaged vehicles, which were shown earlier to correlate with considerable harm to occupants, were also evaluated in this test. As expected, the vehicle damage ranking method was the most effective method for identifying very severely damaged vehicles. More than 1,200 vehicles were very severely damaged at the top-100 intersections determined by this alternative ranking methodology. The Minnesota DOT ranking method identified 968 very severely damaged vehicles and the Iowa DOT's method identified 871. The crash type ranking's top-100 intersections observed the fewest number of very severely damaged vehicles, with 775 vehicles sustaining this damage level.

Table 6. Top-100 list severity statistics

Ranking method	Injury crashes	Fatal crashes	Very severely damaged vehicles
Crash type	2,268	8	775
Vehicle damage	2,063	13	1,232
North Dakota DOT (EPDO, simple)	2,394	9	787
Illinois DOT (EPDO, complex)	2,381	10	840
Iowa DOT (value loss)	1,250	70	871
Minnesota DOT (value loss, reduced fatal)	2,004	31	968

CONCLUSIONS

Nationwide, significant variability exists between the types of crash severity ranking methods used as components of safety management systems. Consequently, these approaches can produce considerably different results, despite their common objective.

In this paper, four different used-in-practice crash severity ranking methodologies were discussed. It was discovered that the North Dakota DOT and the Illinois DOT both utilize EPDO ranking approaches that apply relatively low weights to injury and fatal crashes. The major difference between these two methods is that the North Dakota DOT uses data for three basic severity levels (PDO, injury, and fatal) while the Illinois DOT uses data for five severity levels (PDO, possibly injury, non-incapacitating injury, incapacitating injury, and fatal). The Minnesota DOT and the Iowa DOT use value loss rankings based on the comprehensive injury costs of crashes. These methods apply proportionally higher weights to injury and fatal crashes than the EPDO methods. The major difference between these two value loss ranking methods is that the Minnesota DOT uses a reduced fatal crash cost. The reduced cost is equivalent to twice the cost of an incapacitating injury instead of using their actual comprehensive fatal crash cost estimate of $3,400,000.

Also in this paper, crash severity was more closely defined by evaluating crash factors related to the injury risk of occupants. This was accomplished by evaluating the impacts crash type and vehicle damage had on occupant injury severity using a three-year (1998 to 2000) sample of Wisconsin intersection crash data containing information for more than 400,000 occupants. While the quantity and types of injuries observed varied between crash types, more significant differences in occupant injury risk were observed between vehicles with different vehicle damage levels. For example, in vehicles damaged very severely (i.e., not salvageable), approximately one out of every 50 occupants was killed, whereas in vehicles damaged moderately only one out of approximately 10,000 occupants was killed.

The crash type and vehicle damage relationships to injury severity were then used to develop two alternative crash severity ranking methodologies. The crash type and the vehicle damage alternative ranking methodologies were determined using an average injury cost per vehicle methodology. Average costs were calculated for nine crash types and five vehicle damage levels. It was hypothesized that one or both of the newly developed rankings may be more appropriate than the used-in-practice rankings for identifying potentially hazardous intersections.

The two newly developed alternative crash severity ranking methods and four used-in-practice crash severity ranking methods were applied to subset of Wisconsin intersection data containing 25,633 crashes occurring at 1,369 intersections from 1998 to 2000. Three tests were conducted to evaluate and compare the results of the rankings. The tests focused on the top 100 intersections identified by each ranking method. The similarities between the results of two methods were discussed, as well as each method's annual ranking stability, and the number of severe events (i.e., injury crashes, fatal crashes, and very severely damaged vehicles) that were observed in the top 100 intersections identified by each method. The results of the ranking evaluations are summarized below:

- The crash type ranking produced results similar to the North Dakota DOT and the Illinois DOT ranking methods. The results of this ranking showed more annual stability than all of the other evaluated rankings. The crash type ranking was also effective at identifying injury crashes, but not as effective at identifying fatal crashes or very severely damaged vehicles.
- The vehicle damage ranking produced results most similar to the Illinois DOT ranking method. The results showed more annual stability than the value loss ranking results. Compared to the other rankings, the vehicle damage ranking was effective at identifying injury crashes and even more effective at identifying very severely damaged vehicles. This method was not as effective at identifying fatal crashes.
- The two EPDO rankings (i.e., North Dakota DOT and Illinois DOT) defined crash severity slightly differently but still produced relatively similar ranking results. The North Dakota DOT method was more stable than the Illinois DOT method and both methods were more stable than the vehicle damage ranking and the two value loss rankings. Both EPDO methods were effective at identifying intersections that observed high numbers of injury crashes, but were less effective at identifying fatal crashes and very severely damaged vehicles.
- The results of the two value loss rankings (e.g., Iowa DOT and Minnesota DOT) showed some similarities and some differences between the results of the two rankings. These methods showed the least annual stability of any of the rankings evaluated. The Minnesota DOT method was effective at identifying all three severe events evaluated. The Iowa DOT ranking method, however, was the most effective method for identifying fatal crashes, but identified far less injury crashes than the other evaluated rankings. The Iowa DOT method was effective at identifying very severely damaged vehicles.

In conclusion, crash severity is a subjective measure that is inconsistently defined throughout the nation. As a result, there was not a test and/or comparison that could have decisively determined the best ranking method for finding the most hazardous intersections. The evaluation and comparisons presented in this paper identified some of the similarities and differences between the six ranking methodologies tested.

It is recommended that decision makers use the information presented in this paper to help them identify a crash severity ranking methodology that meets the needs of their jurisdiction. The two alternative ranking methods developed in this research both have characteristics that may be advantageous over current used-in-practice methods. The crash type ranking produced more consistent results than any other evaluated ranking and successfully identified intersections that observed a high number of injury crashes. The vehicle damage ranking procedure was the most effective methodology for identifying very severely damaged vehicles and provided more consistent results than the two evaluated value loss rankings. Neither of the two alternative ranking methods biased results toward fatal crash sites, and both focused on factors related to the geometry and operations of roadways. The vehicle damage ranking appeared to be very effective at identifying severe crash events that were likely to cause severe injuries and fatalities.

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Geometric Categories as Intersection Safety Evaluation Tools

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ABSTRACT

A number of operational and geometric factors impact the safety of at-grade intersections. Examples include the type of traffic control, the existence of left-turn lanes, and adequate sight distance. Determining whether an intersection of particular geometric design has unusual crash patterns could help identify those locations that need additional safety evaluation.

During the last two years, the Wisconsin Department of Transportation and the University of Wisconsin-Madison created an intersection crash database segmented by area type (i.e., rural and urban), traffic control, traffic volumes, and general geometric categories. The overall database included crash report and entering volume information for more than 1,700 locations and 34,500 crashes. In addition, a series of 18 intersection categories were defined with respect to the number of intersection approach legs, the number of major road lanes, and the existence of a left-turn lane and/or a roadway median. More than 1,200 of the intersections were geometrically categorized, and the crash patterns at these summarized and evaluated.

A portion of the results from the activities above is presented in this paper. More specifically, annual average crash frequencies and average crash rates are presented for intersections with varying operational and geometric characteristics. This integration of crash data and geometrics is not typically available for the general statewide safety management of intersections. These safety evaluation measures will be used to identify intersection locations of interest in Wisconsin, and are expected to be an invaluable resource for improving roadway safety in the state and possibly the region.

Key words: crash rate —geometry—intersection—safety

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PROBLEM STATEMENT

Intersection crashes throughout the United States are a costly problem, both economically and in terms of the injuries and fatalities they produce. Limited public funds, however, require that the application of intersection safety improvements be efficient and effective. To reduce the number of intersection crashes, a systematic understanding of the intersection safety problem within a jurisdiction is needed. An understanding of the typical or expected intersection crash patterns can assist transportation professionals with identifying intersection safety problems.

OBJECTIVE

The objective of this paper is to provide intersection crash frequency and crash rate statistics for intersections with varying operational and geometric characteristics. These statistics were calculated in the draft version of the Wisconsin intersection safety report (Knapp and Campbell 2004). The Wisconsin Department of Transportation (WisDOT) report is the first such safety summary for Wisconsin intersections and includes even more detailed intersection statistics than what is presented in this paper. It should assist WisDOT staff and other transportation professionals in their safety decision making process. The information should also help identify intersection locations of concern (e.g., locations with greater than average crash experience) and assist in the investigation of what might be the problem.

RESEARCH METHODOLOGY

The basis for the intersection statistics presented in this paper is three years (1998 to 2000) of intersection and intersection-related crash data from the WisDOT database. Only locations along the WisDOT state or connecting highway system (i.e., those sections of the state or United States highway system maintained by a local jurisdiction) were considered. In addition, only those rural intersection locations with at least three crashes in any one year from 1998 to 2000 were summarized. Urban intersections had to have at least five crashes in any one year. Urban intersections were located in incorporated areas, while rural intersections were located in unincorporated areas. The crash patterns at the intersection locations that met these minimum crash requirements are summarized in this paper.

The intersection safety measures presented in this paper are from the draft version of the WisDOT intersection safety report in which crash data from the years 1998 to 2000 was analyzed. The final version of the report, which is scheduled for completion in July 2006, will be updated for the years 2001 to 2003.

KEY FINDINGS

Several intersection safety measures are presented in this paper. State highway intersection and intersection-related crash statistics (e.g., average annual crash frequency and average crash rate) were calculated and summarized by area type (i.e., rural and urban), traffic control, and volume. Crash statistics were also calculated for intersections with different geometric characteristics, such as the number of approach legs, the existence of a median, the number of travel lanes on the major approaches, and the existence of left-turn lanes on the major approaches.

Intersection Traffic Control and Volume

Table 1 shows the average annual crash frequencies and average crash rates for the intersections in the database designated as rural and urban. The frequencies and rates presented are also grouped by intersection traffic control and by entering volumes, measured in vehicles per day (VPD). Crash rates were calculated per million entering vehicles (MEV). Comparisons of these crash statistics, however, need to account for the different minimum crash requirements used to identify the rural and urban intersections in the database.

A few interesting data patterns and/or trends are shown in Table 1. Not surprisingly, the average annual crash frequencies at the urban intersections were always larger than those that occurred at the rural intersections. In addition, the trends in the crash frequencies for different traffic control and annual average daily entering volumes are similar for both rural and urban locations. The through-stop controlled intersections (i.e., minor-roadway stop-controlled) always had the lowest frequency of crashes and the signalized intersections the highest. Of course, the crash frequencies also increased with volume in both rural and urban areas.

Table 1. Traffic control and entering volume

	Sample size	Average annual crash frequency		Average crash rate per MEV	
	Rural/Urban	Rural	Urban	Rural	Urban
All intersections	592/1,147	3.49	8.26	0.95	0.98
Traffic control					
Signal	99/731	6.61	10.18	0.96	1.05
Through-stop	469/413	2.84	4.87	0.95	0.84
Four-way stop	24/3	3.28	8.00	0.88	1.29
Entering volume (VPD)					
< 15,000	466/226	2.83	4.85	1.01	1.25
15,000 to 25,000	91/441	4.82	6.53	0.72	0.92
> 25,000	35/440	8.80	12.06	0.71	0.88

The patterns shown in Table 1 for average crash rates are less consistent than those described for the crash frequencies. These differences are due to the introduction of entering volumes into the calculation, and the wide range of crash-volume combinations that can and do occur in the database. As expected, urban crash rates are almost always higher than their comparable rural crash rates. However, the urban crash rate calculated for through-stop-controlled intersections was smaller than that calculated for similar rural intersections. This outcome appears to be a result of the database containing data for a number of urban through-stop-controlled intersections with very large entering volumes (e.g., more than 55,000 VPD) but relatively few crashes (e.g., 10 or fewer in three years). In some cases a crash rate produced for this type of situation may not be a true measure of its safety. The high volumes may simply be restricting its use by minor roadway vehicles and subsequently reducing the potential for crashes. The four-way stop-controlled crash rate in Table 1 should also be used with caution due to the small sample used in its calculation (only three intersections with these characteristics are in the database). Not surprisingly, the crash rates at both rural and urban intersections also decrease with volume (i.e., the volume using the intersections generally increases more quickly than the number of crashes at the intersections).

Intersection Geometries

In the WisDOT study, a total of 481 rural and 918 urban intersections were assigned one of 18 geometric categories. These totals represent about 80% of the locations in the intersection crash database. Four primary geometrics were used to differentiate the 18 geometric categories. These geometrics included the number of intersection approach legs, number of major roadway lanes, whether the major roadway had a median, and the existence of left-turn lanes on the major roadway. The intersections were grouped by these four geometrics for both rural and urban intersections. The crash frequency and rate summary statistics for these groups were then calculated and are summarized in the following paragraphs.

Number of Approach Legs

The average annual crash frequencies and average crash rates calculated for four-legged intersections were higher than those calculated for three-legged intersections for both rural and urban intersections (see Table 2). This result is not unexpected, because the number of potential vehicle conflicts at four-legged intersections is more than three times that of three-legged intersections. At three-legged intersections, rural intersections observed a higher average crash rate than urban intersections. This outcome was believed to be the result of a series of very high volume urban intersections with very few crashes. At four-legged intersections, both rural and urban intersections observed similar average crash rates.

Table 2. Approach legs

Intersection approach legs	Sample size Rural/Urban	Average annual crash frequency		Average crash rate per MEV	
		Rural	**Urban**	**Rural**	**Urban**
Three-Leg	105/103	2.63	5.82	0.88	0.72
Four-Leg	349/615	3.89	8.61	0.98	1.00

Number of Lanes

Table 3 shows that the average annual crash frequencies calculated for intersections with a four-lane major roadway was higher than that calculated for intersections with a two-lane major roadway in both rural and urban locations. Since four-lane roadways typically have higher volumes than two-lane roadways, this result was not unexpected. Although intersections with four-lane major roadways generally observed more crashes than intersections with two-lane major roadways, these four-lane facilities averaged lower crash rates in both rural and urban locations. The lowest average crash rate was observed in rural intersections with a four-lane major roadway.

Table 3. Number of lanes on the major roadway

Intersection major roadway	Sample size Rural/Urban	Average annual crash frequency		Average crash rate per MEV	
		Rural	**Urban**	**Rural**	**Urban**
Two-lane	319/178	3.12	5.51	1.03	1.07
Four-lane	143/570	4.72	9.04	0.79	0.99

Median Existence

The average annual crash frequencies and average crash rates calculated for intersections with or without a median, in both rural and urban locations, are shown in Table 4. Since medians separate opposing traffic and sometimes enable the construction of exclusive left-turn lane storage bays, it was anticipated that intersections on divided roadways would have lower crash rates than intersections on undivided roadways. As expected, intersections on divided roadways had lower crash rates than intersections on undivided roadways. It appeared that rural intersections benefited most from the divided geometry and observed a crash rate of 0.79 crashes per MEV compared to the undivided rural intersections, which observed a crash rate of 1.01 crashes per MEV.

Table 4. Existence of a median on the major roadway

Intersection major roadway	Sample size Rural/Urban	Average annual crash frequency		Average crash rate per MEV	
		Rural	**Urban**	**Rural**	**Urban**
Undivided	350/338	3.22	6.16	1.01	1.06
Divided	112/410	4.85	9.88	0.79	0.96

Left-Turn Lanes

The addition of left-turn lanes to an intersection is a typical geometric improvement used to add capacity and improve safety. The average crash frequencies and rates for intersections with and without left-turn lanes on the major roadway are shown in Table 5 for both rural and urban locations. At rural intersections, the average crash rate for intersections without left-turn lanes was higher than for intersections with left-turn lanes. However, the difference in crash rates was relatively small. In urban locations, intersections with or without left-turn lanes observed nearly equivalent crash rates. While the results appear to indicate that the safety impact of a left-turn lane may be small or insignificant, it must be recognized that left-turn lanes are generally added based on capacity needs. Intersections with left-turn lanes may have been as crash prone as intersections without left-turn lanes simply because they had higher left-turn volumes, resulting in more potential vehicle conflicts.

Table 5. Left-turn lanes on the major roadway

Intersection major roadway	Sample size Rural/Urban	Average annual crash frequency		Average crash rate per MEV	
		Rural	**Urban**	**Rural**	**Urban**
Left-turn lanes	261/514	4.17	9.21	0.92	1.00
No left-turn lane	201/234	2.90	5.98	0.99	1.01

CONCLUSIONS

Limited public resources require the efficient and effective application of intersection safety improvements. Intersection locations that may need more detailed safety analysis and/or potential improvements must be identified. An understanding of the typical or expected intersection crash patterns within a jurisdiction can assist transportation professionals with this identification. The intersection safety measures presented in this paper are from the draft version of the WisDOT intersection safety report, in which crash data from the years 1998 to 2000 were analyzed. The final version of the report, scheduled for completion in July 2006, will be updated for the years 2001 to 2003 and will contain numerous typical and/or expected safety measures at intersections based on area type (i.e. rural or urban), traffic control (e.g. signal, through-stop, and four-way stop), traffic volume, and 18 different intersection geometric categories.

In this paper, a portion of the intersection crash statistics calculated in the draft version of the WisDOT intersection safety report was presented. The crash statistics were summarized based on area type, traffic control, traffic volumes, and four primary geometrics: number of approach lanes, number of through lanes on the major roadway, existence of a median on the major roadway, and existence of left-turn lanes on the major roadway. All of these geometric characteristics are typically added as volumes increase. The following conclusions are based on the results of the intersection crash statistics calculations presented in this paper.

The crash database included three years (1998 to 2000) of crash information from those intersections that met a predefined minimum crash requirement. The application of this type of filter is typical, and is normally applied to limit the scope of a safety evaluation to those facilities expected to be of interest. The database summarized in this report considered urban intersections if they had five or more crashes in any one year. Rural intersections were included in the database if they had three or more crashes in any one year. All the locations in the database were along the state highway or connecting highway system. The database included information about more than 34,000 crashes at more than 1,700 locations.

The crash statistics for the intersections were calculated for rural and urban intersections. The average annual crash frequencies for the rural and urban intersections, respectively, were 3.49 and 8.26 crashes per year. The average rural and urban intersection crash rates, however, were determined to be 0.95 and 0.98 crashes per MEV, respectively. For more detailed safety evaluations, similar statistics were also provided for rural and urban intersections with different traffic control and annual average daily entering volumes.

The patterns and trends found for the crash frequencies and crash rates at rural and urban intersections were generally as expected. Not surprisingly, the crash frequencies increased and the crash rates decreased with volume at both rural and urban locations. The average annual crash frequency at signalized intersections was also greater than this measure at four-way stop-controlled intersections. Through-stop-controlled (i.e., minor-roadway stop-controlled) intersections had the smallest average annual crash frequency.

The crash rate patterns found for rural and urban intersections with different traffic control varied more than the previously described crash frequency patterns. Rural signalized and through-stop-controlled intersections in the database had similar average crash rates, and the rural four-way stop-controlled intersections had the lowest crash rate of the three. At the urban intersections, through-stop-controlled locations exhibited the lowest average crash rate, but this outcome was believed to be the result of a series of very high volume intersections with very few crashes. In fact, the average crash rate calculated for urban through-stop-controlled intersections was unexpectedly smaller than the same measure for rural intersections. This characteristic of the database is believed to be one of its recognized weaknesses. The crash rate calculated for signalized and four-way stop-controlled urban intersections was greater than those for the rural intersections, and the four-way stop-controlled rate was the largest average urban crash rate calculated. Unfortunately, this four-way stop-controlled crash rate was also only based on data from three intersections, and it should be used with caution. The lack of data for four-way stop-controlled intersections is the second recognized weakness in the database.

The average crash rate for four-legged intersections was higher than at three-legged intersections. The difference was more evident at urban intersections than at rural intersections.

The average crash rate for intersections with a four-lane major roadway was lower than at intersections with a two-lane major roadway. The difference was more evident at rural intersections than at urban intersections.

The average crash rate for intersections without a median on the major roadway was higher than at intersections with a median on the major roadway. The difference was more evident at rural intersections than at urban intersections.

At rural intersections, the average crash rate for intersections without left-turn lanes on the major roadway was higher than at intersections with left-turn lanes on the major roadway. At urban intersections, nearly equivalent crash rates were observed regardless of the existence of left-turn lanes. The results appear to indicate that the safety impact of a left-turn lane may be small or insignificant. However, it must be recognized that left-turn lanes are generally added when warranted by capacity demands. Intersections with left-turn lanes may have been nearly as crash prone as intersections without left-turn lanes simply because they had more potential vehicle conflicts resulting from higher left-turn volumes.

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Safety, Security, and Efficiency Benefits of Technology in Highway Hazardous Materials Transportation Applications

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ABSTRACT

The Federal Motor Carrier Safety Administration recently undertook an ambitious project entitled the Hazardous Materials Safety and Security Technology Field Operational Test. The objective was to use existing technologies to demonstrate an approach to enhancing the safety and security of hazardous materials transportation by highway, with the goal of speeding up deployment by the trucking industry.

Various technologies and groups of technologies were tested in four segments of the hazardous materials transportation industry and evaluated for benefits to safety, security, and efficiency, as well as benefits to pubic sector law enforcement and emergency response agencies. Wireless communications systems with GPS positioning provided the primary efficiency gains to motor carriers. Positive benefit-cost ratios were identified in all segments of the industry included in the test. In addition, the wireless communication system provided a baseline vulnerability reduction in the security assessment. Additional technologies used in conjunction with the base system provided incremental reductions in vulnerability levels.

The technologies were also successful in reducing notification times to public sector agencies and providing them with increased accuracy of information. These improvements identified in the notification and emergency response system may also provide safety benefits in the form of accident and incident mitigation and security interdiction scenarios.

Key words: HAZMAT—hazardous materials transportation—transportation safety

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PROBLEM STATEMENT

Following the September 11, 2001 terrorist attacks on the United States, the U.S. Department of Transportation was asked to identify areas within the transportation system that were vulnerable to a terrorist attack. Hazardous materials (HAZMAT) transportation was identified as a major area of concern. There are over 800,000 shipments of hazardous materials in the United States each day. These shipments range from high-hazard shipments, such as explosives and toxic-by-inhalation shipments, to small packages of corrosives or flammables. Nearly 300,000 of the daily shipments in the United States are petroleum products by truck, amounting to nearly 3 million tons (U.S. DOT 1998).

When investigating ways to improve the security of hazardous materials transportation, the Federal Motor Carrier Safety Administration (FMCSA) within the U.S Department of Transportation identified the need to have security during all phases of the transportation cycle (pick up, en route transportation, and delivery) and in each element of the shipment (driver, vehicle, and cargo). For each of these elements, different measures may be put in place to reduce vulnerability. These include a regulatory framework, outreach and educational activities, operational and procedural changes, and the increased use of technology, which is the focus of this project.

PROJECT OBJECTIVE

The objective of the Hazardous Materials Safety and Security Technology Operational Test (FOT) was to use existing technological solutions to demonstrate an approach to enhance the safety and security of hazardous materials transportation by highway, with the goal of increasing deployment of effective technologies. The evaluation methodology presented in this paper quantified the benefits and costs of implementing these technologies in the hazardous materials transportation industry.

RESEARCH METHODOLOGY

The research methodology described below was designed for two proposes. The first was to design a system that met the deployment objectives for the project, demonstrating the functionality of the technologies tested. Second, the methodology was designed to facilitate the independent evaluation, which included the benefit-cost analysis for the use of the technologies. This section will provide an overview of the methodology used to meet these objectives.

Risk/Threat Assessment of Hazardous Materials Transportation

Twenty-five functional requirements were identified for the test. These requirements were organized around the pick-up, en route, and delivery phases of a HAZMAT shipment. However, before developing the final approach to test these requirements, an assessment of the risks and threats of various HAZMAT operations and HAZMAT supply chains was conducted. The purpose of this effort was to ensure that the areas of greatest concern (operations with the highest risks and/or vulnerabilities) in HAZMAT transportation were being addressed by the functional requirements and technologies.

The assessment began with a broad look at HAZMAT transportation, including such factors as type of material, quantity of shipment, shipment frequency, type of operation, and routing. These factors were then considered from two different perspectives: intentional (i.e., terrorist) vs. unintentional (i.e., accidental) releases. Then, reference components were established for intentional releases. The primary purpose for defining reference components was to organize, identify, and represent typical vulnerabilities. Each reference component was defined not to represent industry best practices related to security but to reflect the combination of vulnerabilities that can be readily found throughout industry.

Based on this analysis, a number of vulnerabilities were identified for different types and classes of hazardous materials and categorized into four groups: physical, operational, informational, and environmental. Further analysis was conducted to consider HAZMAT groupings from a threat-based perspective as opposed to department of transportation regulatory hazard classes and to consider different types of attack profiles on these hazard groupings. Finally, a consequence analysis was conducted to identify and rank the final set of threat and HAZMAT groupings of highest concern. This ranking was then used as a basis to select the hazardous materials and operational scenarios (based on attack profiles) to be tested.

Technology Selection

This FOT was not a technology development activity, but was rather an integration of existing technologies that could address the specific functional requirements. Listed below are the major technological components tested.

- Tracking technologies
    - Wireless satellite or terrestrial communications (with GPS)
    - Geo-fence mapping software
    - Tethered trailer tracking
    - Untethered trailer tracking
- Panic buttons (in-dash and wireless)
- Driver and cargo authentication
    - Global login
    - Biometric identification
    - Electronic supply chain manifest (ESCM)
    - Electronic seals
- Intelligent onboard computers (OBC)
    - Vehicle disabling (remote, local, and loss of signal)
    - Remote locking and unlocking
- Public sector reporting center concept

Technology Tiers

It was recognized early in the FOT that the unique operational characteristics of many of the hazardous materials carriers around the country would not lend themselves to a full-scale deployment of all the technologies described above on every vehicle. To represent these concerns of the market, the FOT team separated the various technology components into six technology tiers, ranging from a low-end cost of approximately $250 per vehicle to a high-end cost of approximately $3,500 per vehicle. See Table 1.

Table 1. Technology tiers

Tier (cost)	Description
1 ($250)	Includes a digital cellular phone with pickup and delivery software with on-phone/on-board directions/mapping; on-site vehicle disabling with the wireless panic remote
2 ($800)	Includes terrestrial communications with in-dash panic button
3 ($2,000)	Includes satellite communications with an in-dash panic button and global login
4 ($2,500)	Includes all of what is in Tier 3 but adds the additional OBC; the other variant includes satellite communications with an in-dash and wireless panic button with biometric authorization, and e-manifest
5 ($3,000)	Includes satellite communications with an in-dash and wireless panic button with biometric authorization, e-manifest, and an additional OBC; the other variant is swapping the OBC for an untethered trailer-tracking device
6 ($3,500)	Includes satellite communications with an in-dash and wireless panic button with biometric authorization, e-manifest and e-Seals

The price estimates by tier reflect only the hardware installed on the truck in commercial quantities for the vendors involved in the test. It does not reflect the price of servers and dispatch systems, since this can vary widely depending on customer preferences. In addition, the price estimates reflect the cost of an initial installation (assuming no technology previously installed on the truck). A more comprehensive discussion of the costs is included later in the discussion of the benefit-cost analysis.

Scenario Development

The final step in developing the concept of operations for the FOT was to match up each technology component with a testing scenario. The scenarios were developed to address the functional requirements, threats, and vulnerabilities identified in the threat/risk assessment and the desire of the team to test a wide range of technologies across a range of business types. A summary is shown in Table 2.

Data Collection

Both quantitative and qualitative data were collected to support the technology-based and system-based evaluations. Qualitative data was derived from on-site observations and personal interviews during the FOT. Information was gathered on such topics as the operational effectiveness of the technology, customer satisfaction, and institutional challenges. For example, drivers were asked about the ease of use of the various technologies and how adding the technology impacted their daily operations. Quantitative data was collected through system-generated archived reports, which provided ongoing data collection of use and performance of technology applications throughout the FOT.

Table 2. Technology components by scenario

Scenario	Description	Technology components
1	Bulk fuel delivery	• Wireless satellite or terrestrial communication • Global login • In-dash and wireless panic button • On-board computer
2	LTL high-hazard	• Wireless satellite or terrestrial communication • Global login • In-dash and wireless panic button • Wireless panic button
3	Bulk other	• Wireless satellite communications • Biometric authentication • In-dash and wireless panic button • Electronic supply chain manifest
4	Truckload explosives	• Wireless satellite communication • Biometric authentication • In-dash and wireless panic button • Electronic supply chain manifest • On-board computer • Wireless electronic cargo seal • Geo-fencing • Untethered trailer tracking

KEY FINDINGS

The following section describes the three impact areas analyzed: safety, security, and efficiency. The safety impact area was combined with work relating to the pubic sector, as the goals of both areas are the same: prevention and mitigation of incidents. An overview of the unique methodology for each impact area is also provided.

Security Benefits Assessment

Methodology

The security benefits were measured primarily in terms of the measured reduction of vulnerability, using the following basic vulnerability assessment formula:

$$\text{Threat} \times \text{Vulnerability} \times \text{Consequence} = \text{Cost}$$

Threat is primarily a function of terrorist aims and operating procedures. This factor was based on the work completed during the initial threat/vulnerability assessment and held constant in this analysis. Measuring vulnerabilities was accomplished with the assistance of an expert panel that included representatives of industry trade associations, government experts, and security and counterterrorism professionals. The expert panel assisted in recruiting a wider group of experts to serve on a Delphi panel. Two rounds of Delphi questionnaires were undertaken, one to look at the vulnerabilities and threats before the application of the technology solutions, and one to look at the same vulnerabilities and threats after the application of the technologies.

A technique designed to arrive at a consensus regarding an issue under investigation, the Delphi method consists of a series of repeated interrogations, usually by means of questionnaires, of a group of individuals whose opinions or judgments are of interest. After the initial interrogation of each individual, each subsequent interrogation is accompanied by information regarding the preceding round of replies, usually presented anonymously. The participant is encouraged to reconsider, and if appropriate, to change a previous reply in light of the replies of other members of the group. After two or three rounds, the group position is determined by averaging. The Delphi method was originally developed at the RAND Corporation by Olaf Helmer and Norman Dalkey (Linestone and Tuiroff 1975).

Security Assessment Results

The security assessment considered the vulnerability reduction in the three main attack profiles identified in the vulnerability analysis: theft, diversion, and interception by load type. In all cases, the technologies provide some vulnerability reduction, starting with the core element of the wireless communications systems. The additional technologies, such as panic button and vehicle disabling, then provided incremental gains in vulnerability reduction. Table 3 provides a sampling of the vulnerability reduction achieved through the use of selected technologies. Vulnerability reductions from 0% to 10% are considered nil; from 11% to 25% are considered low; from 26% to 50% are considered medium; and greater than 50% are considered high.

Table 3. Select reductions in overall vulnerability

Technology scenario	Bulk fuel	LTL-high hazard	Bulk chemicals	Truckload explosives
Wireless communications + GPS position (base)	17%	16%	16%	12%
Driver ID + base	25%	25%	23%	18%
Panic alert + base	27%	25%	25%	21%
Panic alert + remote vehicle disabling + base	32%	32%	31%	25%

Table 4 provides information regarding the vulnerability numbers focusing on theft. The vulnerability reductions in Tables 3 and 4 make it apparent that the technologies are more useful for reducing the vulnerabilities relating to theft, which included hijacking, than for other potential scenarios. Table 4 provides a view of the different methodologies employed while studying the effects of the technologies on vulnerability reduction. First, the vulnerability reduction was calculated, and then multiplied by potential consequences to develop the overall benefits. Then, benefit-cost ratios were calculated on an annual basis. Finally, realizing that threat can be unpredictable and vary over time, breakeven numbers of successful attacks that needed to be prevented via the technologies to equal the costs of deploying the technologies was calculated.

The breakeven number of attacks is presented as a decision tool: if one believes that the probability of an attack (threat) is greater than the breakeven for a technology combination for a load type, and then for society, the investment in the technology combination can be considered sound. For example, preventing one attack over the three-year period would easily surpass the breakeven point in the bulk fuel scenario shown below.

Table 4. Select benefits for theft of bulk fuel scenario

Technology	Vulnerability reduction	Benefits (000,000)	Benefit-cost ratio	Breakeven point
Wireless communication with GPS (base)	23%	$622	1.5	.108
Base + driver ID	40%	$933	2.1	.117
Base + panic button	42%	$955	2.3	.114
Base + panic button + vehicle disable	52%	$1,207	2.6	.123

While the technology combinations tested show promise for reducing the vulnerabilities of truck-based hazardous materials shipments, the Delphi panelists and the test participants provided a clear message that not all solutions are foolproof. Their responses also indicated that not all solutions perform the same in a dynamic real world environment in which human and technology failures can occur and where the adversary is looking for new ways to subvert security efforts. The implementation of technology is only one part of a thorough security strategy, and the results show that technologies can have an impact on vulnerability reduction and therefore improve security.

Efficiency Benefits Assessment

Efficiency Benefits Assessment Methodology

The operational efficiency assessment examined return on investment (ROI) by using the formula

ROI = Total Benefits Achieved/Total Investment Costs

The motor carriers' primary viewpoint for operational efficiency is focused mainly on the ability to communicate efficiently with drivers, know where vehicles are located and to be able to manage these assets, track driver and vehicle operating performance, and plan loads more efficiently. Two different methodologies were employed. For the bulk fuel and LTL pick-up and deliver scenario, a driver productivity model was found to be most appropriate. Analysis of the other scenarios focused on an ROI model measuring the following direct benefits to motor carriers:

- Reduced telecommunications costs
- Increased driver-dispatcher ratios
- Reduced on-the-road downtime, which translates into potential load increases or trips
- Reduced fuel consumption and engine wear
- Reduced maintenance costs and increased revenue through decreased repair downtime
- Reduced miles

Efficiency Assessment Results

The operational efficiency assessment shows a positive benefit-cost ratio in all fleet segments that were part of the FOT, with a high-end ratio of 7.2:1 for a truckload explosives carrier, with payback in only three months. Past surveys have indicated that the market penetration rate reaches nearly 60% if the payback period is between one and two years (ATRI 2002). The best case scenario payback in all four segments of the industry was in less than six months. The costs shown below included purchase and installation costs amortized over three years, plus ongoing messaging with hourly positioning and maintenance costs. Table 5 is a summary of the efficiency assessment.

Table 5. Efficiency assessment summary

Operation type	Factors	Annual cost/truck	Annual benefit/truck	Benefit-cost ratio	Payback (months)
LTL (pick-up and delivery)	Driver productivity	$1,188 (terrestrial GPS)	$1,920	1.6:1	13
Bulk fuel			$5,832	4.9:1	3
Bulk chemical	• Reduced call stops • Improved maintenance scheduling • Reduced out-of-route miles • Improved utilization	$1,524 (satellite GPS)	$1,560–$7,116	1.0:1–4.7:1	5–34
LTL-high hazard			$2,352–$9,840	1.5:1–6.5:1	3–17
Truckload explosives			$1,824–$11,004	1.2:1–7.2:1	3–25

An example of the typical operational efficiency gains measured and assessed by the evaluation team under this effort is a bulk fuel motor carrier. Based on data from 19 drivers over an 11-week period, the weekly driver productivity reports demonstrated an overall increase in driver productivity of 11%, bringing the aggregate level to approximately 90% of the target the carrier had set. Based on this data, the evaluation team calculated an average savings of $5,800 per year per truck for bulk fuel carriers (versus a case of no technology).

The ROI model developed was based on actual operational efficiency data provided by three of the carrier participants. In addition to the overall efficiency assessment, to explore the low-end efficiency benefits the project drew on previous work that indicated that not all carriers were able to gain benefits in all areas. For those areas where this situation pertained, a minimum benefit was calculated, with benefits shown as a range. For example, all carriers may not be able to generate additional revenue by hauling an additional load or by adding another truck to their fleet. The annual costs defined in Table 5 include the costs of hardware and installation, amortized over three years (1/3 hardware costs) plus service fees for a year.

Safety and Public Sector Benefits

Overview

Although the primary focus of this project was to look at the technology from the perspective of the private sector user of the technology, some effort was also spent conducting testing with public sector agencies. Three functional requirements involved public sector agencies:

1. Hazardous materials driver identification and verification by roadside safety enforcement officers
2. Hazardous materials cargo route adherence by the dispatcher and roadside safety enforcement officers
3. Real time emergency alert message notification by the dispatcher

In addition to using the technologies that were already part of the test to study the interaction between the public sector and the on-board technologies (specifically, satellite communications, global login, biometric login, electronic supply chain manifests, geo-fencing and panic buttons), a new technology concept, the "public sector reporting center," was developed by integrating the data into a single database. This allowed users of the data to specify when they would like alert messages and how they would like to receive them. For example, an agency could specify that it would like alerts when an unauthorized driver alert is generated for a vehicle carrying hazardous materials. This message was then delivered via phone, e-mail, fax, or to a wireless handheld device as specified by the user. In addition to the public sector users, it was also shown that the individual trucking companies had a need and desire for this capability.

Evaluation Methodology

The public sector evaluation effort focused on two hypotheses:

1. The response times for emergency and enforcement personnel to respond to a hazardous materials safety or security incident can be improved through the implementation of the technologies and the public sector reporting center operational concept.
2. The quality of information provided to first responders will improve through the implementation of these technologies and the reporting center operational concept.

The qualitative interviews with the public sector participants were used to collect information concerning the quality and timeliness of information provided by the technologies tested. Additional information was gathered to determine user perceptions of effectiveness and appropriateness to the enforcement operational environment. Tailored testing and staged events were used to assess whether the systems met the functional requirements and quantify and qualify the improvements in alert notification timeliness.

Results: Response Time Improvements

The first hypothesis is accepted based on the data generated from field testing at the four on-site locations and comments from law enforcement and emergency response personnel. The following is a summary of the results, organized according to the functional requirements tested with the public sector.

<u>Driver identification</u>

Typically, without the use of technology, driver identification takes from 30 minutes to 2 hours and may require a trip to the local police station. Biometrics and global login allow on-site verification in minutes.

<u>Route adherence</u>

Motor carrier estimates for locating an off-route vehicle range between four and eight hours, while electronic geofencing identified the situation in approximately one hour based on standard positioning rates, which could be increased if necessary based on carrier needs and the sensitivity of the load.

<u>Panic alerts</u>

According to the Center for Technology Commercialization (CTC), the best estimate for notification time (including exact location information) to state police agencies in the event of an incident is 27 minutes. The CTC serves as NASA's Northeast Regional Technology Transfer Center, covering the six New England states plus New York and New Jersey. The CTC acts as a gateway for the transfer of NASA and other federal technology to private industry. The CTC's Public Safety Technology Center is an informational clearinghouse focused on the development and uses of advanced technologies that can help reduce violent crime, promote officer safety, and impact public safety's ability to effectively combat crime and respond to terrorist threats. Other sources, including the COMCARE alliance and Operation Respond, indicate an elapsed time of 20 minutes. The FOT technologies performed this task at a maximum time of two minutes, showing a potential savings of 18 minutes for alert notification and location information. In some cases, material type was included with the alert as well, further enhancing response capabilities.

Results: Hazardous Materials Information Improvements

The second hypothesis was also accepted based on the data discussed above. The following is a summary of the results organized according to the functional requirements tested with the public sector.

Driver identification

The biometric and global login provides accurate, truthful information about a driver during roadside inspection activities. With laptop access in remote locations, law enforcement can verify driver identity and, with ESCM capabilities, ensure that the correct driver is associated with the correct vehicle/cargo.

Route adherence

Wireless communications and geofencing provide direct, timely, and accurate data on the location of a vehicle. The PSRC approach gave exception-based alerts to law enforcement by whatever means they chose, and it functioned quickly and reliably. The alerts included all available data, including GPS position and manifest information.

Panic alerts

Panic buttons provide an effective way to transmit emergency event information directly to law enforcement through the PSRC and provide the exact location of the subject vehicle utilizing the GPS position.

CONCLUSIONS

The core enabling technology, wireless communication systems with GPS positioning, is the technology that showed the primary operational efficiency gains. Productivity gains in terms of increased personnel and asset utilization are found to outweigh the costs of deploying the technology with a payback on investment in less than one year in many cases. With the proven reliability of the technology in the market place and the appropriateness of application to a wide range of fleets, significant industry net benefits could be realized through full deployment. Even with attractive ROI and low payback periods, however, full deployment is likely to be a long-term scenario.

Moreover, this core enabling technology also provides significant security benefits, through the reduction of identified vulnerabilities. The implication is that the wireless communication system with GPS has the capability of more than covering its costs to motor carriers while providing a significant security benefit to society. The remaining technologies, while providing considerable potential security benefits (societal benefits), do not provide direct operational efficiency benefits to the motor carriers.

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The views expressed herein are those of the authors and do not reflect the opinions of the U.S. Department of Transportation, the FMCSA, or the other organizations involved. Status updates and reports can be found at the FMCSA (website www.fmcsa.dot.gov) and at the project team website (www.safehazmat.com).

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Addressing Non-Intersection Crashes Statewide: Identification and Ranking of Highway Segments in Need of Safety Upgrades Using a Floating Highway Segment Tool Integrated with GIS

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ABSTRACT

Identification of highway segments with an unusually high occurrence of non-intersection crashes has been a particularly challenging problem for the Wisconsin Department of Transportation, because these crashes were scattered over approximately 9,500 miles of highway.

The PRÈCIS floating highway segment algorithm was developed to identify crash rates and crash densities at any given point along the entire length of any undivided state trunk highway (STH). Once the entire set of STH mileage is processed through the algorithm, crash rates and crash densities can be processed to identify locations where these metrics exceed a predetermined value. Locations can be rank-ordered for treatment across the entire STH system. Crash rate information is then integrated on GIS maps and a graph providing visual feedback about higher non-intersection crash concentrations.

For this research, crash information was integrated with roadway feature information to produce tabular listings organized by increasing mile point, providing a rich information environment for safety engineers addressing safety issues on any given highway section. Additional information can be extracted from a photolog database, eliminating to a large degree unnecessary field visits. In addition, overall crash rates, crash densities and other statistics were developed for each number-of-lanes/population density (urban or rural) cohort. Statewide statistics were developed as well. Specific injury and fatal, wet and snow, darkness, horizontal or vertical curve, and fixed object crash statistics were produced.

The methodology developed to address statewide non-intersection crashes systematically provides a powerful tool to prioritize statewide safety improvements, apply for federal safety improvement program funding, and address litigation cases.

Key words: decision support tool—highway segment safety—non-intersection crash—statewide safety—two-lane highway

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

PROBLEM STATEMENT

During the last six years, the Wisconsin Department of Transportation (WisDOT) has been working to address specific Wisconsin transportation safety priorities. These priorities, inspired by the 1997 AASHTO Strategic Highway Safety Plan and modified to address areas where fatalities could be reduced the most for Wisconsin, include keeping vehicles on the roadway and minimizing crash consequences of vehicles leaving the roadway. WisDOT also identified improving data and decision support systems as issues of paramount importance in setting safety priorities, allocating safety improvement funding, and evaluating countermeasure effectiveness.

The effort presented herein addressed the run-off-road (ROR) crash problem on undivided rural highways and produced a decision support system designed for statewide application. The extent of the ROR problem in Wisconsin was known in general terms at the outset of this effort; crash frequencies, crash types, and numbers of objects hit were available and could be summarized for the entire state, a county, or a highway. Crashes could be plotted on a base map and crash rates could be produced within the available GIS database. However, the GIS database was created mainly to support a statewide highway project database, rather than produce crash rates. Thus, GIS-based crash rates were calculated for sequential highway segments of uneven lengths, resulting in some drawbacks described later in this paper.

Identification of highway segments with an unusually high occurrence of non-intersection crashes was a particularly challenging problem for WisDOT, because these crashes were infrequent and scattered over approximately 9,500 miles of highway. It was necessary to find a method to calculate crash densities (crashes per mile) and crash rates (crashes per 100 million vehicle-miles of travel, or 100MVMT) and identify highway segments where these two metrics were unusually high. Although some results are presented in the "Findings" section below, the focus of this paper is on the method, rather than the results.

OBJECTIVES

It was desired to develop a methodology with statewide applicability, suitable for the identification of crash concentrations located at long distances from each other. The methodology needed to be sensitive enough to work with very low crash densities (crashes per mile) and use a uniform criterion for all highway segments. For the methodology to be useful to decision makers, results presentation would have to be appropriate from the statewide level down to the route and the individual project level, using an intuitive problem segment identification. A practical methodology also needed to be inexpensive in terms of database and work-hour requirements; reliance on existing databases and automation were therefore very desirable.

METHODOLOGY

Emphasis was placed on developing safety measures of effectiveness appropriate for highway segments: crash density (crashes per mile) and crash rate (crashes per 100MVMT). Use of sequential highway segments was undesirable, because of the possibility of missing higher crash concentrations when crashes occurred on either side of the common border between two sequential segments. Also undesirable was the use of uneven-length and especially short segments, because crash rate fluctuations in response to small changes in crash frequencies become more pronounced as segment length becomes shorter.

The PRÈCIS Algorithm

A floating highway segment algorithm, PRÈCIS, was developed to address these concerns. Each highway was divided into 1/100th-mile segments, the smallest mile point increment used for the WisDOT highway inventory. Each 1/100th-mile segment was represented by a database record that included the number of crashes at this mile point, the average daily traffic (ADT), and highway information, such as number of lanes and urban or rural designation. Once the length of the floating highway segment was specified (a length of one mile will be used in this example), a crash rate was calculated for each database record. This crash rate was based on the crash experience and travel present in the1/2-mile segment preceding and 1/2-mile following the record. The floating highway segment would then move 1/100th of a mile downstream (one "step") and a crash rate would be calculated for the next record, and so on until the entire highway was covered. The same process would be repeated for each state highway until a crash rate was calculated for each 1/100th of a mile on each state highway.

PRÈCIS Results Use

When the entire network of state highways is processed through PRÈCIS, a rich crash rate database will be available for analysis. Highway segments with the highest crash rates could then be identified statewide and targeted for inclusion in a safety improvement program. The selection process could be based on a rate quality control method, whereby only segments with crash rates exceeding the average statewide crash rate by a certain multiple of standard deviations would be selected for treatment. The total mileage selected for treatment could be fine-tuned to fit the available highway safety upgrade budget by adjusting the standard deviation multiples (a higher number would produce a smaller set of segments, and vice versa).

PRÈCIS Graphics

The results presentation capabilities of PRÈCIS, which was developed in a GIS environment, make it a useful tool for decision makers, from the statewide to the project level. The rich crash rate database can be visually displayed with graphics similar to those shown in Figure 1, where color and spatial extent indicate the relative severity and exact location of safety problems along a given state highway. Figure 1 presents the entire alignment of a state highway (approximately 200 miles long). A similar results presentation is possible for a county or for the entire state. The small state map insert indicates the location of the highway in the state.

Two color-coded lines of different thicknesses (see top-right insert) following the highway alignment are used to present crash rates for ROR (thick line) and all non-intersection crashes (thin line). Crash rate ranges represented by each color are explained in the legend: red indicates higher crash rates and green indicates lower rates. Problem segments can be readily identified, and their exact locations can be determined by cross-street name or by geographic features displayed on the map.

The line chart at the bottom of Figure 1 consists of dots, each representing the exact crash rate calculated for each 1/100th-mile segment. The y-axis represents the number of crashes per 100MVMT, while the x-axis represents the cumulative mile point along the highway. The blue line represents non-intersection crash rates and the green line represents ROR crash rates. Locations with the highest crash rates can be pinpointed and locations where ROR crashes are the majority of non-intersection crashes (i.e., blue and green lines are close together) can be identified for treatment.

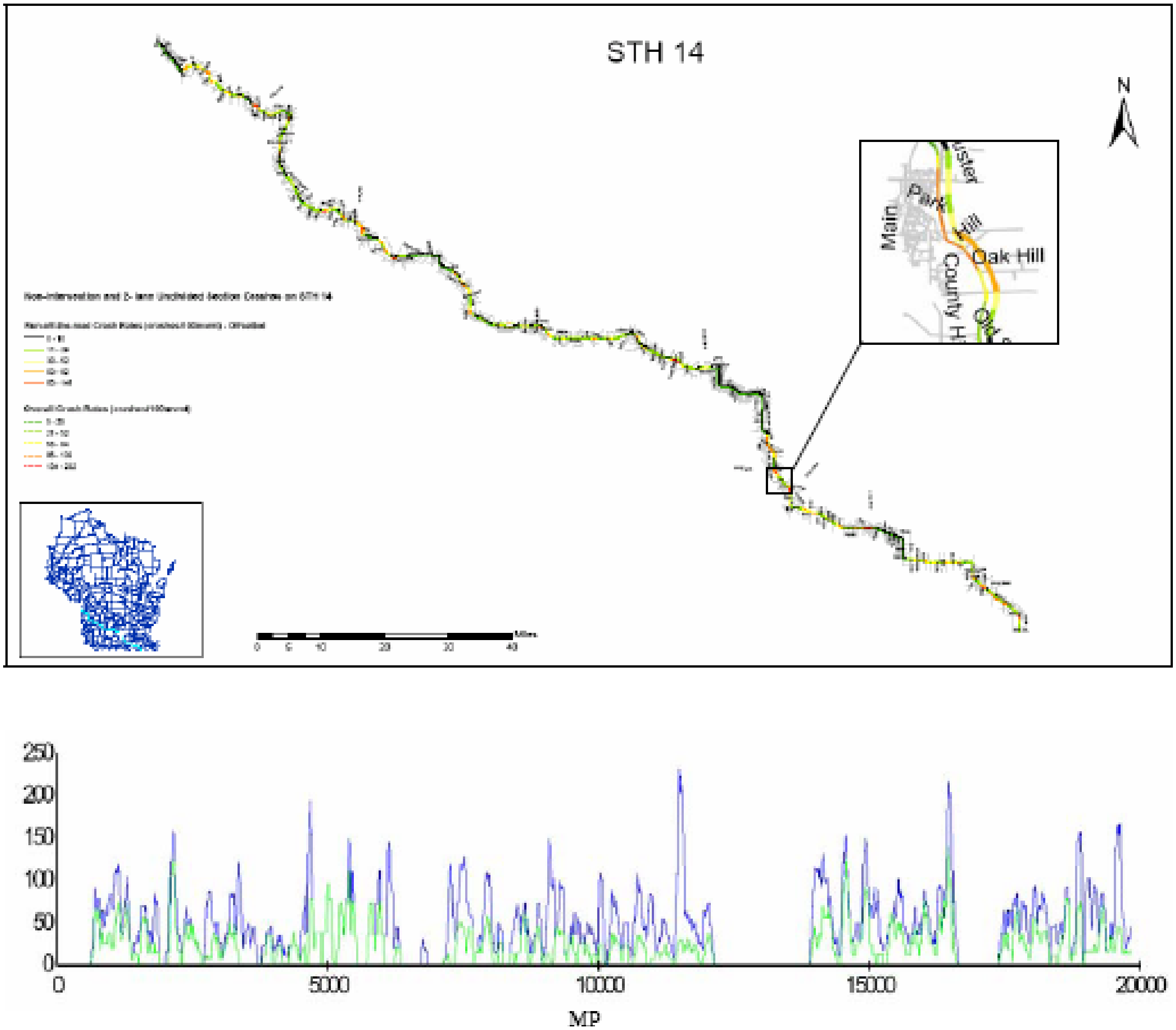

Figure 1. PRÈCIS crash rates for non-intersection (thin line top, blue line bottom) and run-off-road crashes (thick line top, green line bottom)

The safety specialist can readily select segments of interest from the entire length of a highway and zoom in for a closer examination, overlaying the PRÈCIS graphics on an aerial photograph or other appropriate means for a smaller scale analysis, and consulting crash record information, pictures from the photolog database, available plan and profile drawings, or other sources of information.

Tabular Results Presentation

A special tabular presentation was developed to accompany PRÈCIS graphics (see Table 1). Roadway feature descriptions were interleaved with crash database information and presented in a cumulative mile point sequence. This presentation allows the safety specialist to mentally walk through a highway

Table 1. Interleaved highway and crash information, sorted by cumulative mile point

Crash / State Trunk Highway Log Interleaf ROR Crashes on STH 14. Page F

COUNTY	HWY	REFPT	CUM_MP	FEATURE	AADT	Relation to Roadway	HR	DATE	Rd Cond	Lgt Cond	Sev	Most Harmful Even
LACR	014E	001	.00	>> C OF LA CROSSE	.	.	.		.	.	.	.
LACR	014E	004D	.82	B-32-0300 BRIDGE	.	.	.		.	.	.	.
LACR	014E	005	1.34	USH 14 WB	.	.	.		.	.	.	.
LACR	014E	007	1.83	STH 33 EB	.	.	.		.	.	.	.
LACR	014E	008	2.05		.	.	.		.	.	.	.
LACR	014E	010	2.88	STH 35 NB	.	.	.		.	.	.	.
LACR	014E	011A	3.60	EAST AVE	.	.	.		.	.	.	.
LACR	014E	011T	4.31	LOSEY BLVD	.	.	.		.	.	.	.
LACR	014E	014B	6.10	USH 61 SB	.	.	.		.	.	.	.
LACR	014E	014G	6.19	USH 61 SB	.	.	.		.	.	.	.
LACR	014E	014K	6.20	USH 61 NB	.	.	.		.	.	.	.
			6.86		6980	On Roadway	22	10/23/99	DRY	DARK	Inj	GUARDRAIL FACE
LACR	014E	016	7.22	CTH MM	.	.	.		.	.	.	.
			7.32		6980	Outside Should. Left	6	01/06/99	SNOW/SLUSH	DAWN	PDO	GUARDRAIL FACE
			7.42		6980	Outside Should. Right	12	01/06/99	SNOW/SLUSH	DAYLIGHT	PDO	GUARDRAIL FACE
			7.47		6980	Shoulder	19	03/26/99	DRY	DARK	PDO	GUARDRAIL FACE
LACR	014E	017M	7.51	JUSTIN RD	.	.	.		.	.	.	.
			7.52		6980	Shoulder	16	06/10/98	DRY	DAYLIGHT	PDO	GUARDRAIL END
			8.05		6980	Outside Should. Right	8	01/21/98	ICE	DAYLIGHT	PDO	DITCH
			8.60		6980	On Roadway	14	06/08/99	WET	DAYLIGHT	PDO	GUARDRAIL FACE
LACR	014E	018	8.60	HELKE RD	.	.	.		.	.	.	.
			8.60		6980	Outside Should. Left	4	05/10/2000	DRY	DARK	PDO	DITCH
			8.60		6980	Outside Should. Right	13	01/05/99	SNOW/SLUSH	DAYLIGHT	PDO	GUARDRAIL FACE
			9.58		6980	On Roadway	11	10/18/98	DRY	DAYLIGHT	PDO	BRIDGE RAIL
			9.58		6980	On Roadway	3	06/27/99	DRY	DARK	Inj	GUARDRAIL FACE
LACR	014E	022	9.69	BREIDEL COULEE RD	.	.	.		.	.	.	.
			9.99		6980	On Roadway	19	07/26/99	DRY	DAYLIGHT	Inj	64
LACR	014E	025	10.32	>> T OF SHELBY	.	.	.		.	.	.	.
			10.62		6980	Outside Should. Right	16	06/01/2000	WET	DAYLIGHT	PDO	EMBANKMENT
LACR	014E	026	11.05	CTH M	.	.	.		.	.	.	.
			11.05		6120	Outside Should. Right	20	11/20/2000	0	DARK LIGHTED	Inj	GUARDRAIL END
			11.15		6120	Outside Should. Left	2	11/24/2000	ICE	DARK	PDO	OVERTURN
			11.35		6120	Outside Should. Left	16	02/17/2000	SNOW/SLUSH	DAYLIGHT	Inj	DITCH
			11.55		6120	Shoulder	7	12/08/99	ICE	DAWN	Inj	OVERTURN
			11.75		6120	On Roadway	15	04/29/2000	DRY	DAYLIGHT	PDO	OTHER NON-FIXED OBJECT
LACR	014E	027	12.17	CTH MM	.	.	.		.	.	.	.
			12.58		6120	Outside Should. Right	12	12/09/99	DRY	DAYLIGHT	PDO	OVERTURN
			12.67		6120	Outside Should. Right	17	04/01/98	DRY	DAYLIGHT	PDO	GUARDRAIL FACE
			12.88		6120	On Roadway	11	02/20/2000	DRY	DAYLIGHT	PDO	OTHER NON-FIXED OBJECT
LACR	014E	029	13.28	BARTSCH RD	.	.	.		.	.	.	.
			13.28		6120	Outside Should. Right	4	01/30/99	DRY	DARK	PDO	DITCH
			13.38		6120	Outside Should. Right	14	02/12/2000	SNOW/SLUSH	DAYLIGHT	Inj	EMBANKMENT
VERN	014E	032	14.43	CTH N	.	.	.		.	.	.	.
			14.43		6440	Outside Should. Right	14	12/16/2000	SNOW/SLUSH	DAYLIGHT	PDO	TRAFFIC SIGN POST
VERN	014E	034	15.30	HOHLFELD RD	.	.	.		.	.	.	.
			15.75		6440	Outside Should. Left	8	12/07/99	ICE	DAYLIGHT	PDO	GUARDRAIL FACE
			16.05		6440	Outside Should. Left	21	11/26/99	ICE	DARK	Inj	GUARDRAIL END
			16.15		6440	Outside Should. Left	18	12/28/2000	SNOW/SLUSH	DARK	Inj	EMBANKMENT
			16.30		6440	Outside Should. Right	0	12/25/98	DRY	DARK	PDO	EMBANKMENT
VERN	014E	035	16.45	DAHLEN LA	.	.	.		.	.	.	.

segment and place individual crashes at the exact points where they occurred. The table includes locations of intersections, driveways, bridges and political jurisdiction boundaries (city or county lines). Any crash features can be selected for inclusion in the table, depending on the focus of the safety analysis.

Table 1 is focused on ROR crashes on US Highway 14 near the Minnesota-Wisconsin border. Only major landmarks are presented from the highway log (driveway locations were omitted); ADT, crash location in relation to the roadway, date and time, light and pavement condition, crash severity, and the most harmful event were selected from the crash record. Highway and crash information are presented sequentially, sorted by cumulative mile point. The table can be used in conjunction with the color-coded highway alignment and crash rate line charts in Figure 1.

FINDINGS

Crash rates and crash densities were generated at the individual highway level. Aggregate statistics were also produced for two-, three-, and four-lane undivided highways, and separately for urban and rural highways.

Table 2 presents aggregate statistics for several undivided highway categories. Most of the undivided highway mileage (93%) is two-lane rural highways, which account for 86% of all ROR crashes. For two-lane highways, urban crash densities are almost twice as high as their rural counterparts; crash rates are lower than those for rural highways. One ROR crash occurs every 2.25 miles per year, on average.

Table 2. Statewide run-off-road crash statistics

Crash type	No. of lanes	Miles	Crashes	Crashes per mile	Crash rate*
Rural	2	8819.81	11629	0.44	34.19
	3	15.76	45	0.95	35.12
	4	65.28	122	0.62	16.74
Urban	2	402.72	1036	0.86	25.14
	3	12.53	60	1.60	30.76
	4	154.69	569	1.23	21.27

*Crash rate in crashes per 100MVMT.

About two-thirds of ROR crashes on two-lane rural highways involve collisions with fixed objects, which occur once every 3.7 miles, on average; half occur under dark conditions, about 4.5 miles apart, on average (see Table 3). Serious outcome crashes (injury and fatal) represent 44% of all ROR crashes and occur once every 5.3 miles, on average.

Table 3. Statewide statistics for run-off-road crashes on two-lane rural highways

Crash type	Crashes	Crashes per mile	Crash rate*
Fixed Object	7,195	0.27	21.15
Dark	5,839	0.22	17.17
Injury + Fatal	5,117	0.19	15.04
Wet + Snow	4,997	0.19	14.69
Overturn	3,060	0.12	9.00
Ditch	1,593	0.06	4.68
Hz Vt curve	1,571	0.06	4.62
Tree	1,133	0.04	3.33
Guardrail	802	0.03	2.36
Utility Pole	661	0.02	1.94
Embankment	613	0.02	1.80
All crashes	11,629	0.44	34.19

*Crash rate in crashes per 100MVMT

CONCLUSION

PRÈCIS uses a fixed-length highway segment; thus, crash rate fluctuations due to segment length variations are completely eliminated. The floating segment length can be adjusted, depending on the crash type being analyzed, and made sufficiently long to avoid sudden crash rate increases (crash rate spikes) where isolated crashes are present. The problem of missing crash concentrations when crashes are located on either side of the border between fixed sequential highway segments is eliminated; all crash concentrations are identified.

The rich crash rate database created using the PRÈCIS floating segment algorithm allows a detailed examination of the entire state and the identification of all segments with higher-than-usual crash rates. The results presentation capabilities of PRÈCIS in a GIS environment make it a useful tool for decision makers, because it does not require special training or familiarity with the algorithm or the underlying database. The availability of highway/crash interleaf tables is a useful complement to the graphic crash rate presentation.

A safety specialist can use the generated graphs to create lists of highway segments for inclusion in statewide safety upgrade programs. The total length of segments chosen for treatment can be adjusted by varying the minimum highway segment crash rate used as a criterion for inclusion in the treatment list. PRÈCIS is flexible and can be adopted to analyze any type of widely scattered crashes and/or contrast the occurrence of two crash types side-by-side, as demonstrated in Figure 1.

PRÈCIS is automated and applies a uniform criterion statewide, generating consistent results across highways. Annual applications of PRÈCIS will also generate consistent results from year to year. The algorithm’s robustness can provide the backbone for a statewide safety upgrade priority list that should withstand court challenges and will be useful when applying for federal highway improvement funds.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the help of the following WisDOT employees: Dick Lange, John Corbin, Brad Javenkoski, and Carrie Cooper. We also acknowledge the help of Pete Rusch of the FHWA, as well as the many other individuals who built and maintained the high-quality databases we relied upon for this project.

Influence of Compaction Energy on Soil Engineering Properties

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ABSTRACT

Strength and deformation parameters of a compacted soil are known to be related to soil type and moisture. However, little attention has been directed towards understanding the influence of compaction energy on soil type and moisture. This paper describes a laboratory study conducted to evaluate the relationship between soil type, soil moisture content, and compaction energy on five cohesive soil types.

Specimens were compacted with impact energy at levels of 355, 592 (standard Proctor), 987, 1643, and 2693 kJ/m^3 (modified Proctor) over a wide range of moisture contents to determine dry unit weight, unconfined compressive strength, and the secant (50% strain) stiffness. In total, 125 Proctor tests and 95 unconfined compression tests were performed. At each energy level, a soil specimen was tested at four to five moisture contents with respect to its standard Proctor moisture range. In addition, 48 consolidated undrained triaxial tests were performed at the five energy levels and four moisture content levels for a silt to evaluate changes in effective stress shear strength parameters.

This paper summarizes the results of statistical analyses performed on all tests conducted. The models that best explain variability in dry unit weight, strength, and stiffness are presented. Models are presented for each individual soil type and for all soils grouped together. Independent variables used in the modeling include compaction energy, moisture content, Atterberg limits, material passing the No. 200 sieve, and clay fraction. Results show that compaction energy is a key factor in determining soil strength and stiffness parameters and should be considered during the planning phase of any earthwork construction operation.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: compaction energy—soil moisture content—soil stiffness—soil strength

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Expanding the Use of Integral Abutments in Iowa

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ABSTRACT

In the mid-1960s Iowa began experimenting with jointless bridges constructed with integral abutments. Later, in the 1980s, researchers at Iowa State University developed a methodology for analysis of the piles in integral abutments. The methodology determines the depth at which a pile can be considered to have a fixed support. With an established location of fixity, the pile can be checked for ductility to ensure that it can flex without damage and can be checked as a column to ensure that it can support the abutment, end span, and traffic. With the methodology, it is possible to set general policy limits on the use of integral abutments, address industry trends, and consider unusual site conditions. Based on a parameter study, Iowa recently increased the bridge length limits for integral abutments to 575 feet for concrete superstructures and to 400 feet for steel superstructures, with reductions for skew. Use of compact, Grade 50 H-piles, or deeper prebored holes permits longer end spans to 150 feet or more for steel superstructures. On a case-by-case basis, the methodology permits consideration of piles with downdrag, retaining walls near abutments, unsymmetrical site conditions, and sites with bedrock near the surface.

Use of jointless bridges reduces initial costs by eliminating bearings and expansion joints and reduces maintenance costs because there can be no damage from leaking joints. Expanding the use of jointless bridges with integral abutments has improved the overall life-cycle cost of Iowa bridges.

Key words: integral abutments—jointless bridges

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

Background

A covered timber bridge had sidewalls and a roof to protect the timber superstructure from weather and decay. In a similar way, a jointless bridge deck protects the superstructure and substructure from deterioration. Integral abutments are jointless bridge components that eliminate the need for expansion joints in the deck and prevent runoff from rapidly damaging beam or girder ends, bearings, and abutments.

Iowa has minimized the use of expansion joints in bridge decks for many years. In 1939, V7 three-span steel bridge standard plans made use of continuous beams with only one deck expansion joint. In 1960, H12, H13, and H14 prestressed beam standard plans had no deck expansion joints, and in about 1965 Iowa began experimenting with integral abutments. Early experiments involved supporting abutments on timber piles with heads wrapped in carpet padding. In 1967 the bridge office issued the first integral abutment bridge length guidelines, and the office updated the guidelines in 1980. During the 1980s, Iowa State University (ISU) conducted research on steel H-pile behavior and analysis of integral abutment bridges (Greimann et al. 1987). After the research, in 1988 the Office of Bridges and Structures at the Iowa DOT issued guidelines that were in effect until 2002. The 1988 guidelines limited concrete or steel bridges to a length of 150 feet with skews to 45 degrees, to 300 feet with skews to 30 degrees, and to 500 feet with special investigation.

Typical Integral Abutment

A typical Iowa integral abutment for a continuous welded plate girder bridge is illustrated in Figure 1. The construction process begins as the contractor prebores holes and drives a single line of steel H-piles aligned for weak axis bending (although the piles may be skewed up to 30 degrees to align with the abutment). The contractor then cuts off the piles so that they will project into the abutment two feet, thereby ensuring a fixed pile head. To prevent loss of soil below the abutment, the contractor fills the prebored holes with bentonite slurry. Next, the contactor places formwork and reinforcement and casts the abutment footing. After the abutment footing cures, the contractor erects girders on three-inch deep S-shapes or steel bars. Finally, when additional formwork and reinforcement are in place, the contractor casts the end diaphragm with the deck. Casting the end diaphragm and deck makes the pile-to-beam or pile-to-girder connection rigid, causing the piles and superstructure to behave as a continuous rigid frame.

Research

At the time Iowa began experimenting with integral abutments, at least one engineer in the bridge office performed moment distribution analyses of the continuous pile-beam frame. Performing the analyses undoubtedly raised questions about the behavior of the piles in the frame, and, to answer the questions, Iowa initiated several research projects in the 1980s that were completed by Iowa State University civil engineering faculty.

ISU researchers first conducted field tests of steel H-piles and laboratory tests of model piles that they correlated with finite element analyses. The finite element analyses modeled a pile with beam elements and vertical and horizontal soil springs. The researchers addressed the questions of pile ductility and column capacity and eventually developed hand computation examples for checking a skewed integral abutment pile under simplified assumptions for superstructure behavior. Because the AASHTO standard specifications (AASHTO 1983) did not include information for pile ductility and weak axis bending, the researchers developed their methodology from additional information in the engineering literature and the American Institute of Steel Construction, Inc. allowable stress design specifications (AISC 1978).

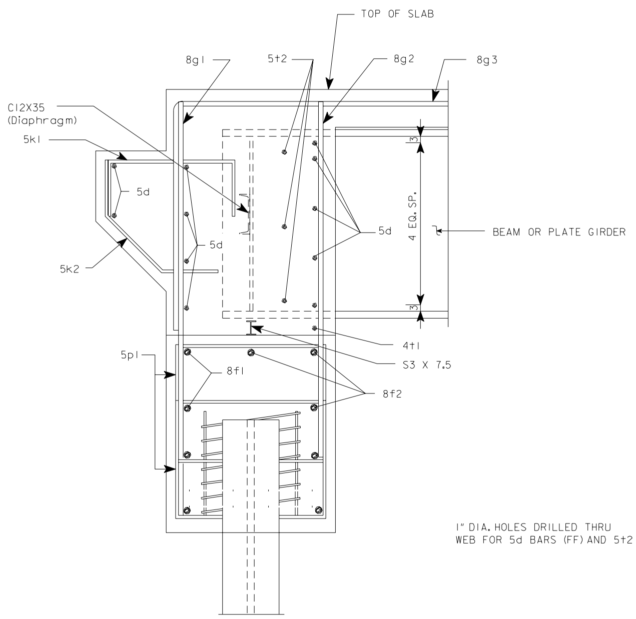


Figure 1. Longitudinal cross-section through an Iowa integral abutment for a steel girder bridge

EQUIVALENT CANTILEVER PILE MODEL

For an analysis of an integral abutment bridge, the ISU researchers proposed a simple equivalent cantilever model (Greimann et al. 1987). In the model, the pile may be either pinned or fixed at the head. The pinned-head model is appropriate for a timber pile head wrapped in carpet padding, and the fixed-head model illustrated in Figure 2 is the appropriate choice for the typical Iowa integral abutment supported by steel H-piles.

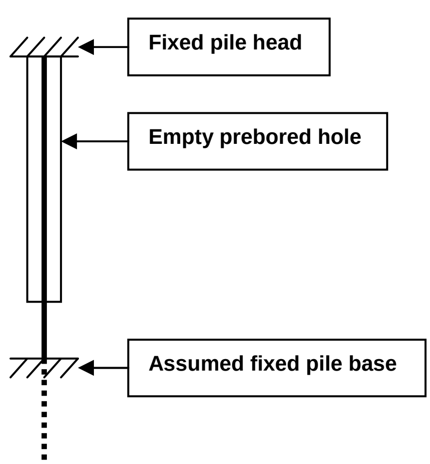


Figure 2. Fixed-head equivalent cantilever pile model

In the model, the pile may be embedded in soil over its full height or the upper portion may be freestanding in a prebored hole, the typical condition. In Iowa practice at various times, the prebored hole has been empty, filled with loose sand, or filled with bentonite slurry, but for the pile model, the hole is always assumed to be empty because any weak fill offers little support to the pile.

The equivalent cantilever pile is assumed to be fixed at some depth in the soil. Location of the assumed cantilever fixed end depends on prebored hole depth and soil properties and also depends on whether the equivalent cantilever length is being evaluated for horizontal stiffness, bending moment, or buckling of the pile. The ISU researchers developed a trial and error procedure for determining the assumed depth to fixity, which usually determines a distance in the range of three to five feet below the bottom of a prebored hole. The equivalent cantilever pile then can be checked for ductility and column capacity. The ductility check is necessary if the bridge is relatively long and its expansion or contraction will cause inelastic bending in the pile.

Whether a pile is sufficiently ductile is based on a comparison of rotation capacity and rotation demand at an inelastic hinge. For design purposes, the researchers added a safety factor to the rotation capacity.

Because the pile in an Iowa integral abutment is expected to bend primarily about its weak axis, the width-to-thickness proportion of the flanges is important. The pile has the greatest capacity to bend inelastically without distortion if the flanges are compact, a designation that indicates the pile can reach its full plastic moment and sustain an amount of inelastic rotation at the plastic hinge location without local buckling.

The basic rotation demand for AASHTO standard specifications Load Group IV, which includes temperature effects, is illustrated in Figure 3. In their methodology, ISU researchers also included an approximation for live load rotation of the end span in their ductility check. Because of the approximation, the check uses only the Δ dimensioned in the figure.

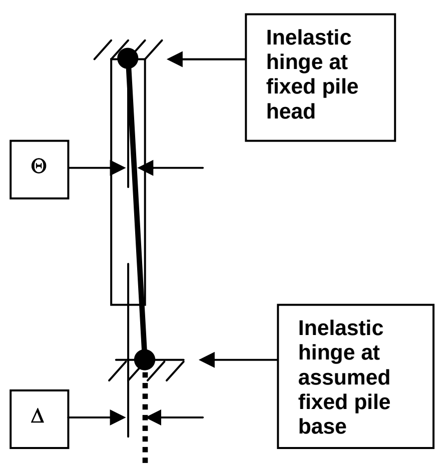
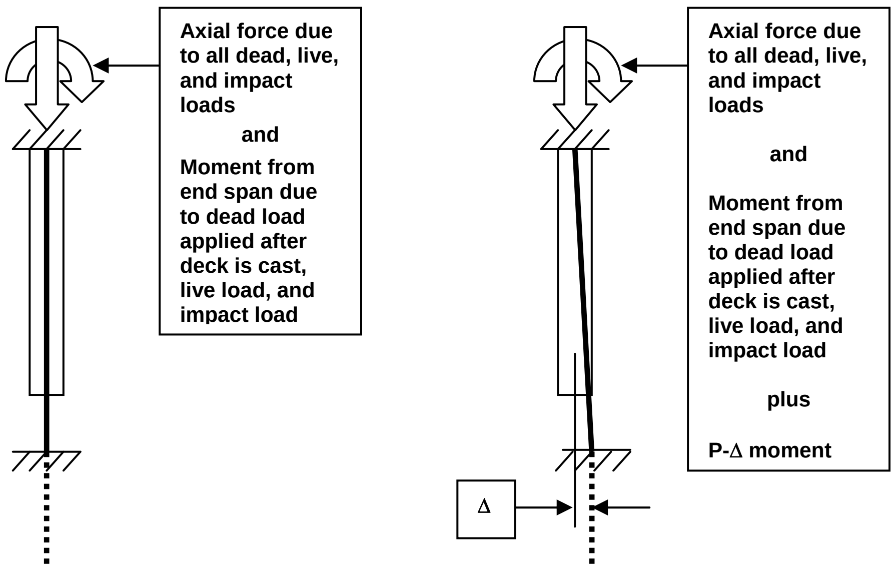


Figure 3. Rotation demand in the fixed-head equivalent cantilever model

Except for short bridges, Iowa places an integral abutment pile in a prebored hole filled with bentonite, which is assumed neither to load nor offer lateral bracing to the pile. Therefore, it is conservative to assume that the pile is unsupported between the bottom of the abutment and the assumed location of fixity below the prebored hole. The pile can then be checked as a column for stability and yield according to the usual allowable stress design procedure.

Under AASHTO standard specifications Load Group I, the basic dead and live load case, the column loading is obvious. There will be an axial force due to all dead, live, and impact loads on the pile and a moment from the end span due to loads that occur after the end diaphragm is cast with the deck, which include the dead load applied after deck is cast, live load, and impact load, as shown in Figure 4a.



a. Load Group I **b. Load Group IV**

Figure 4. Loading in the fixed-head equivalent cantilever model

Under Load Group IV, the thermal moment requires consideration. If the pile is limited to elastic stresses, it will not be able to sustain much horizontal movement and an integral abutment bridge will have a very limited permissible length. The ISU researchers termed this approach Alternative 1. It is appropriate for timber piles.

Because steel H-piles have the capacity to deform inelastically, the ISU researchers also suggested a different approach, Alternative 2. Assuming that the pile in the equivalent cantilever model hinges inelastically to accommodate the extreme thermal expansion or contraction of the bridge, the pile will be displaced, but the longitudinal strains in the pile at the plastic hinge locations that are caused by the expansion or contraction of the bridge superstructure are treated as residual strains that do not affect the strength of the pile. In the displaced condition, the moment due to thermal effects will be limited to a secondary moment, the product of the axial force and half the horizontal displacement, shown in Figure 4b. Only half the horizontal displacement is used because of the fixed-fixed end conditions in the fixed-head pile model. The secondary P-Δ/2 moment is much smaller than the primary elastic moment, due to abutment displacement, and allows a greater bridge length.

With Alternative 2, there are several fatigue considerations. First, there is the high-stress–low-cycle condition caused by annual thermal expansion and contraction of the bridge. Second, there is a low-stress–high-cycle condition caused by daily thermal expansion and contraction. Third, there is a low-stress–high-cycle fatigue condition caused by live and impact loads on the end span. University of Minnesota researchers recently examined the combined effects of the first two conditions for H-piles in weak-axis bending and concluded that the conditions usually need not be considered (Huang et al. 2004). ISU researchers completing a new integral abutment study are recommending use of American Institute of Steel Construction seismic compact section criteria in design, in order to consider loss of local buckling capacity under the first condition (Abendroth and Greimann 2005; AISC 2002).

Present AASHTO allowable fatigue stress for the base metal in the pile is 24 ksi for more than 2 million cycles. Bending stresses in the pile due to the third condition, fluctuating live plus impact load on the end span, are unlikely to exceed this value if the pile is required to meet ordinary service load column checks. Thus, successful column checks will serve indirectly as an approximate check for this fatigue condition. Evidently, there is no known field evidence to date of fatigue damage for any of the three conditions.

The column checks need to be performed for both AASHTO standard specifications Load Groups I and IV. Load Group I requires that computed stresses be checked against allowable stresses without increase, but Load Group IV permits a 25% allowable stress increase. In general, Load Group I column checks will limit the length of end spans, and Load Group IV checks will limit the length of the bridge.

APPLICATION OF MODEL TO OVERALL POLICY

Although the 1988 Iowa DOT bridge length guidelines permitted integral abutments for about two-thirds of Iowa bridges, experience in the field suggested that integral abutments be used for as many bridges as possible. In cases where integral abutment bridges were constructed with lengths and/or skews greater than the guidelines, those bridges were performing well. In cases where integral abutments were not used, deck expansion joints were difficult to install and maintain. In some cases, bearings and beam or girder ends were deteriorating due to deicer runoff through poorly performing deck expansion joints.

If the equivalent cantilever model and its assumptions are accepted, it is possible to develop policy and consider options to expand the use of integral abutments. The ductility and column checks include a limited number of variables that can be explored with the goal of improving the checks.

Consider the ductility check first. Rotation capacity will be at its maximum if the pile section is compact. Due to the use of scrap steel in electric furnaces, most H-piles produced today have an actual yield stress of 50 ksi, regardless of specification. This higher yield stress, however, requires stockier flanges for a section to meet the compact designation. To ensure that piles are compact at 50 ksi, Iowa now requires HP 10x57 piles for standard integral abutments, except for continuous concrete slab bridges that have lower rotation demand than beam or girder bridges.

Ductility demand depends on the rotation, Θ, shown in Figure 3. This rotation could be reduced if the thermal expansion or contraction, Δ, were reduced. A steel superstructure could be changed to concrete to reduce Δ, but that change is seldom an option in practice. However, if the depth to assumed pile fixity is increased, the rotation demand will be reduced. It is relatively easy and inexpensive to deepen the prebored hole to move the location of assumed fixity downward, and improved ductility is one reason Iowa increased the standard prebored hole depth to 10 feet.

Now consider the column checks. Basically those checks compare computed and allowable axial and bending stresses, as indicated in the simplified equations in Figure 5. Equation (1) checks column stability, and Equation (2) checks yield at points of support. To improve the first part of the each check, the computed axial stress, f_a, could be reduced by increasing the number of piles, which will generally have minimal cost. Another option for the stability check would be to increase the allowable axial stress, F_a, by specifying Grade 50 steel rather than Grade 36. Based on driving stresses, Iowa has made the change to Grade 50 and now is considering its beneficial effect on integral abutment policy.

$$\frac{f_a}{F_a}+\frac{C_m f_b}{\left(1-\frac{f_a}{F'_e}\right)F_b}\leq 1.0 \quad \text{Equation } (1)$$

$$\frac{f_a}{0.472F_y}+\frac{f_b}{F_b}\leq 1.0 \quad \text{Equation } (2)$$

Figure 5. Simplified combined stress equations

The second part of the column checks compares computed bending stress with allowable bending stress. The computed bending stress, f_b, could be reduced with a stiffer superstructure. However, a stiffer superstructure generally would require deeper beams or girders or a shorter end span, neither of which is desirable in practice. A better option is deeper prebored holes that will increase the flexibility of the piles and cause less moment. Deeper prebored holes, however, will reduce the allowable axial stress, so there is a limit to this option. Increasing the number of piles will also reduce the bending stress. In the second portion of the bending stress check, the allowable bending stress, F_b, can be increased with a change from Grade 36 to Grade 50 steel.

To set a general policy for integral abutments in 2002, the Office of Bridges and Structures set several parameters. These parameter decisions included the use of Grade 36 HP 10x42 piles for 6-ksi axial loading, 10-foot–deep prebored holes, a 45-degree limit for bridge skew, and very stiff clay as a conservative soil condition. These basic decisions along with temperature ranges and typical superstructure properties led to the end span and bridge length limits in the present design manual. Later, the design manual was revised to change the standard pile shape to HP 10x57 for prestressed concrete beam and steel plate girder bridges (OBS 2005). The HP 10x57's ductility was checked for a yield stress of 50 ksi, which did not affect the end span and length limits.

The standard policy length and skew limits are shown in Figure 6, along with data points for a group of recent overhead bridges. Based on bridge length, the 1988 policy limits envelop about 70% of the bridges, and the 2002 policy limits envelop nearly 90%. The exact percentage is unknown because the 2002 policy limits also include an end span length limit, and therefore some two-span steel girder bridges do not qualify for integral abutments, even though they do not exceed the bridge length and skew limits.

With the simple pile model and engineering judgment, since 2002 the Office of Bridges and Structures has set and adopted policy for use of integral abutments. These policy changes include the following:

- Compact pile sections
- Deeper prebored holes
- Separate length limits for concrete beam and steel girder superstructures
- Increased length limits: to 575 feet for concrete bridges and to 400 feet for steel bridges without skew
- End span length limits

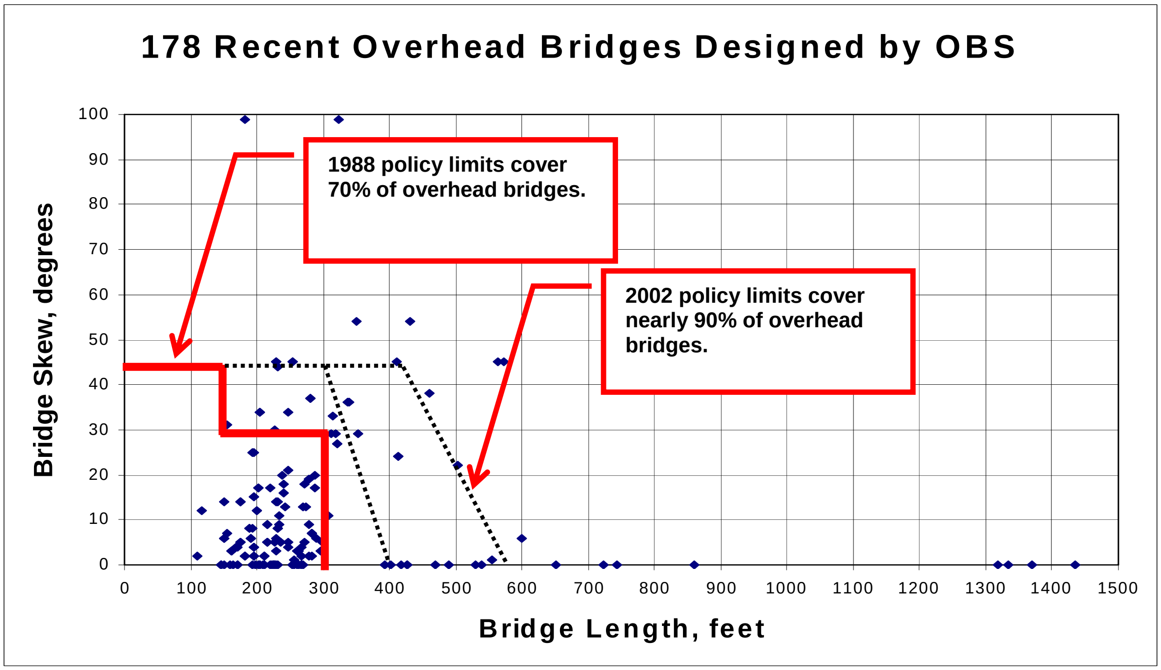


Figure 6. Bridge length and skew limits overlaid on recent overhead bridges

A parameter study for an increase in specified yield strength from 36 ksi to 50 ksi indicated that the present bridge length limits could theoretically be doubled. Ductility would be the controlling check. However, at the increased bridge lengths the annual temperature movement at each abutment would have a range of four inches or more, possibly too much to accommodate with simple pavement joint details. Therefore, it is unlikely that the Office of Bridges and Structures will increase length limits to the full theoretical limits as a general policy.

APPLICATION OF MODEL TO SPECIAL CASES

Long End Spans

The downtown Des Moines weathering steel overpasses for the I-235 rebuilding project in some cases clashed with the 2002 policies. Because the two-span overpasses are relatively short, they are not limited by length policy, but steel end spans of about 150 feet exceed the 2002 extended policy limit of 125 feet with 15-foot–deep prebored holes.

The I-235 weathering steel bridges with long end spans were checked on a case-by-case basis, and the Office of Bridges and Structures usually approved those with minimal skews. The individual checks allowed consideration of the specific design loads, superstructure stiffness, and actual soil conditions, all of which were typically favorable factors. In some cases, the individual checks required minimum-cost design modifications, such as more piles or deeper prebored holes. If Grade 50 piles had been considered, some of the modifications would have been unnecessary.

Downdrag

If embankments need to be constructed without adequate time for settlement to occur, or if abutment piles penetrate a soft cohesive layer that will compress under the weight of an embankment, the abutment piles need to be designed for downdrag. Prebored holes filled with bentonite relieve downdrag in the upper region of the piles stressed by movement of the abutment. Therefore, downdrag need not restrict use of integral abutments. An I-35 integral abutment bridge designed for downdrag by Shuck-Britson, Inc. is shown in Figure 7. Deep compressible soil layers, coupled with new abutment fills of significant magnitude, caused major downdrag loads on the abutment piles. Large 14-inch H-piles in 15-foot prebored holes accommodated the downdrag loads and allowed integral abutment movement without overstressing the piles.

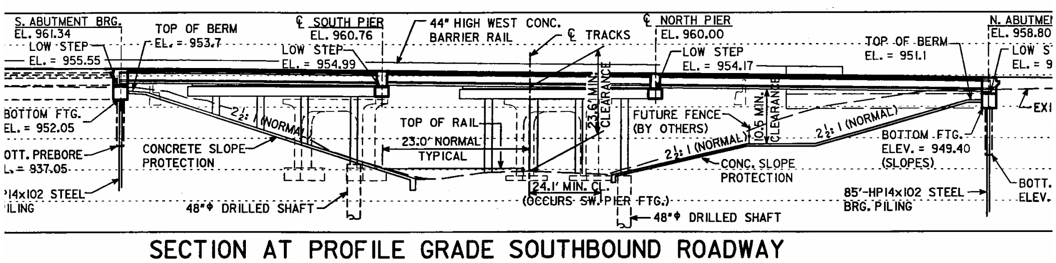


Figure 7. Southbound I-35 bridge over Union Pacific Railroad

Retaining Walls

Although most Iowa overpass bridge sites permit a sloping berm up to the abutment, in some cases there is insufficient room for the berm, and a retaining wall is necessary. If a retaining wall is placed in front of the integral abutment piles, the piles could cause additional pressure on the wall as they flex with the horizontal movement of the superstructure. As illustrated in Figure 8 (bridge designed by HDR, Inc.), to avoid the additional pressure the piles may be placed inside corrugated metal pipe (CMP) sleeves sufficiently large so that the piles can move without contacting the pipes. Pipes are filled with bentonite so they serve as cased, prebored holes. If the assumed point of pile fixity falls within the length of the CMP sleeves, the sleeves may be partially filled with saturated sand.

Figure 8. Piles in corrugated metal pipe sleeves for MLK Parkway bridge over I-235

Bedrock

If bedrock is relatively close to the surface, abutment piles may not have adequate length to flex at both abutments. If one abutment must be founded directly on bedrock but there is sufficient depth for piles to flex at the other abutment, the abutment on bedrock may simply be considered the center of the bridge. Piles at the other abutment can be checked for thermal movement based on the entire length of the bridge, rather than half the length. When bedrock is too close for the methodology that determines the assumed location of fixity, piles can be set in concrete in holes cored in the rock, as shown in Figure 9 (bridge designed by Calhoun-Burns and Associates, Inc.). The concrete sockets determine the location of fixity, and the piles can then be checked for ductility, stability, and yield.

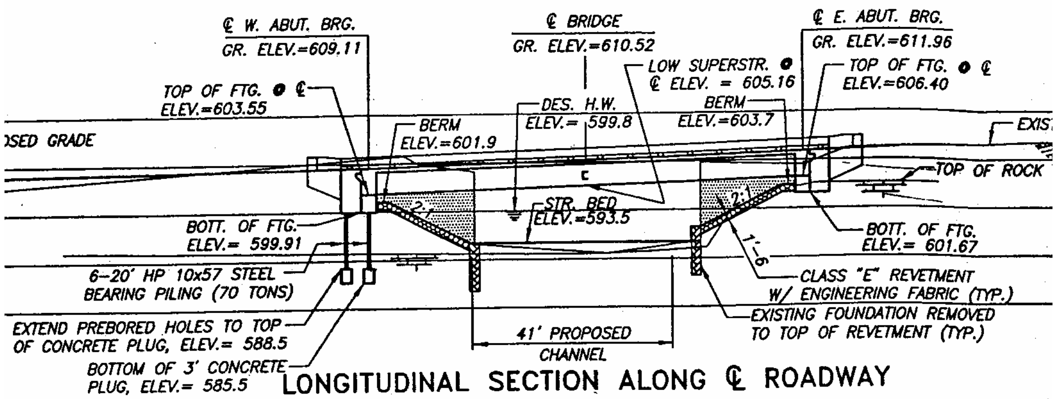


Figure 9. Van Buren County Road J56 over Flatrock Creek

SPECIAL CASES BEYOND THE MODEL

Sensitive Adjacent Structures

Occasionally, there are fragile or historic structures near the bridge site that could be damaged directly from pile driving vibration or indirectly from soil settlement caused by the vibration. In those cases, it is possible to place a drilled shaft as a support for each integral abutment pile as shown in Figure 10 (bridge designed by Parsons Transportation Group). A pile oriented for weak axis bending is simply embedded in the top of a drilled shaft, and the condition is similar to concreting piles in rock. The piles are made sufficiently long to flex with expansion and contraction of the bridge. For this foundation structure, the simple pile model is useful only for preliminary design; the structure should be analyzed in a more comprehensive manner.

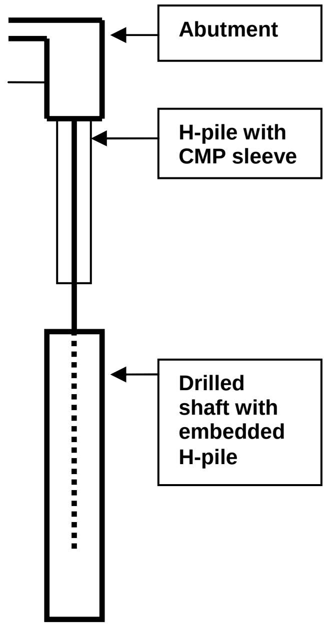


Figure 10. Concept for integral abutment for 9th Street over I-235

Unusual Site Conditions

For the relocated US 151 Maquoketa River crossing in Jones County, each of the continuous welded plate girder twin bridges were more than 1,000 feet long and had an average 3% grade. Both the lengths of the bridges and lengths of end spans were outside the 2002 policy limits for use of integral abutments.

Based on the site conditions and desire to avoid deck joints as much as possible, the Office of Bridges and Structures decided that the concept should be an integral abutment and fixed pier at the low end of each bridge and expansion piers and a finger-type expansion joint at the high end of each bridge. The integral abutment for one of the twin bridges is illustrated in Figure 11 (bridge designed by WHKS and Co.). Note the unusual double line of piles with webs oriented for weak axis bending.

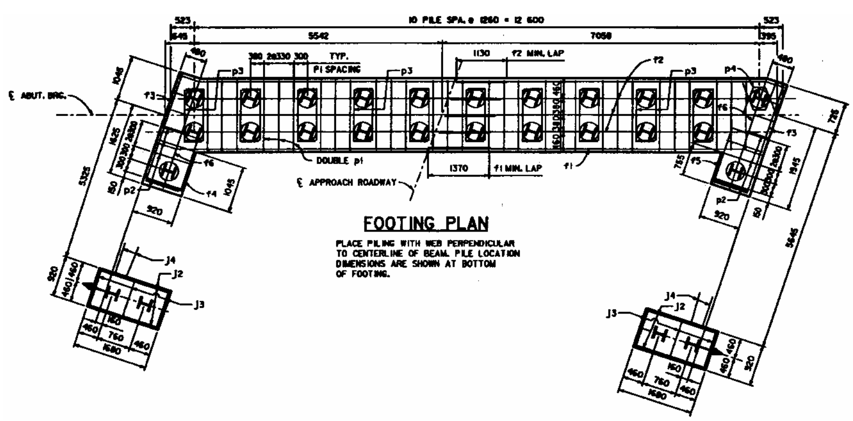


Figure 11. West abutment for U.S. 151 over Maquoketa River

SUMMARY

The simple cantilever pile model developed by ISU researchers is useful, not only for setting general policy, but also for considering special design conditions. The model has allowed the Iowa DOT to increase length limits for integral abutment bridges and thus expand the use of jointless bridges. The pile model has also allowed the use of integral abutment bridges for cases where downdrag is a concern and where bedrock is relatively close to the surface. Creative details such as placing piles in corrugated metal pipe sleeves and supporting piles with drilled shafts has further expanded the use of integral abutments.

Use of jointless bridges can reduce initial costs by eliminating expansion joints and bearings. The first cost of expansion joints and bearings is equivalent to several piles per abutment, which encourages the option of adding piles to make integral abutments feasible. Jointless bridges reduce maintenance costs because there can be no damage to beam ends or bearings due to leaking joints. Expanding the use of jointless bridges with integral abutments to longer bridges, longer end spans, and unusual site conditions has improved the overall life-cycle cost of Iowa bridges.

ACKNOWLEDGMENTS

The authors thank William Tucker, PE in the Office of Bridges and Structures at the Iowa DOT for his studies of applications of the equivalent cantilever pile model in the mid-1990s and Dr. Robert Abendroth, PE for his permission to cite the draft of the ISU integral abutment research report. The authors also thank the engineering consultants named in the text describing figures 7 through 11 for their innovative applications of integral abutments.

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Tama County's Steel Free Bridge Deck

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ABSTRACT

A major bridge problem in the United States is the corrosion of reinforcing steel and the subsequent deterioration of the surrounding concrete due to deicing salts. There have been efforts in the past to alleviate these problems by using reinforcement that will not corrode, including clad steel reinforcement, fiber-reinforced polymer (FRP) reinforcement, and non-corrosive MMFX steel reinforcement. Another innovative concept is the steel-free bridge deck, which has been developed in Canada. These decks are free of internal steel reinforcement and rely on the internal arching action of the concrete slab, when the slab is confined in both the longitudinal and transverse directions. Using shear studs for composite action between the concrete deck and the steel girders provides longitudinal confinement, while steel straps welded to the top flanges of the girders at regular intervals provide the transverse confinement. FRP reinforcement is included transversely and longitudinally in the decks for temperature and shrinkage reinforcement. The most significant potential benefits of steel-free deck bridges are the decks' durability

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and the material cost savings due to the significantly reduced amount of deck reinforcing. The favorable durability should provide reduced long-term maintenance costs.

To the authors' knowledge, the first steel-free deck bridge in the United States was constructed in Tama County, Iowa as part of the FHWA Innovative Bridge Research and Construction program. Since the original bridge deck had to be completely removed, Tama County used this opportunity to increase the width of this 41-foot simple span bridge from 24 to 28 feet. Conventional epoxy-coated reinforcement was used in cantilever overhangs. In this bridge, concrete with polypropylene fibers were used to reduce plastic shrinkage cracking and provide post-crack ductility for the slab. The Tama County steel-free bridge deck was designed using the provisions of the Ontario Highway Bridge Design Code.

Included in this project are the deck design, construction documentation, periodic load testing, and evaluation. This past summer, the bridge deck was placed with minimal difficulties. Approximately five months after construction, the bridge was load tested using both static and dynamic loadings. Prior to these tests, instrumentation was installed to measure strains and deflections at critical locations. This paper will provide details on the design and construction of the steel free bridge deck, as well as a comparison of the load test performance with the expected design behavior. This bridge is visually inspected periodically and will be tested a second time (approximately a year after the initial test) to determine any changes in its structural behavior.

Key words: arching—bridge decks—steel-free

INTRODUCTION

In areas where the use of deicing chemicals for winter roadway maintenance is common, corrosion of reinforcing steel in bridge decks is widespread (Purvis and Babaei 1994). Every year, this reinforcing corrosion causes deterioration to the concrete in bridge decks, resulting in millions of dollars being spent on repair, rehabilitation, and replacement (Gannon and Cady 1992). The elimination of corrosion in reinforcement could result in savings of millions of highway construction and maintenance dollars. Because this reinforcement corrosion is difficult and costly to abate, the most cost-effective method for elimination of the corrosion is to remove the reinforcing steel itself (Thorburn and Mufti 2001).

A bridge deck design method developed by researchers from Dalhousie University, Halifax, Nova Scotia and the Ministry of Transportation of Ontario, Canada has eliminated the reinforcing steel from bridge decks. This method takes advantage of the natural arching action present in deck slabs on girders. Tensile restraint is provided externally to the concrete deck slab and away from the harmful chlorides that initiate corrosion (Ventura and Cook 1998). This tensile restraint is provided by attaching steel straps transversely across the top flange of the girders. This paper documents the load testing and monitoring of a steel-free bridge deck in Tama County, Iowa, the first such structure constructed in the United States.

Tama County received funding through the Federal Highway Administration's Innovative Bridge Research and Construction program for the design, construction, and monitoring of an experimental bridge deck replacement with a reinforcing steel-free deck on a single-span 40-foot–long steel girder bridge. The Bridge Engineering Center at Iowa State University assisted in securing the funding and is responsible for the documentation of the design and construction of the steel-free bridge deck, load testing of the bridge, and evaluation of the field monitoring.

BRIDGE LOCATION AND DESCRIPTION

The bridge is located in Tama County, Iowa, on County Highway E64, approximately 2.5 miles east of US 63, over a branch of the Iowa River. This segment of roadway carries approximately 450 vehicles per day. The bridge deck was placed on the Tama County, Iowa, bridge on July 27, 2004 and the bridge was load tested on November 16, 2004.

The project was a bridge deck replacement of a 41-foot–long, 0° skew, steel I-beam bridge (see Figure 1). The deck was widened from 24 feet to 28 feet. The bridge cross-section (Figure 2) consists of seven beams. The existing beams were reused on this project. The five interior beams (S 24x79.9) were spaced at 3 feet, 10 inches, with the exterior beams (S 24x105.9) spaced at 5 feet. The exterior overhang was 1 foot, 6 and 1/2 inches from the edge of the slab to the centerline of the exterior girder. The beams were seated on stub abutments.

The bridge deck concrete is an Iowa C-4WR-C15-S35 mix with synthetic structural fiber added. TUF-STRAND SF structural grade polymer synthetic fibers from Euclid Chemical Company were added to the concrete mix at a rate of six lbs/yd^3. The structural fibers were not added to provide strength to the concrete, but to assist in controlling cracking and crack propagation.

Figure 1. Elevation view of the Tama County bridge

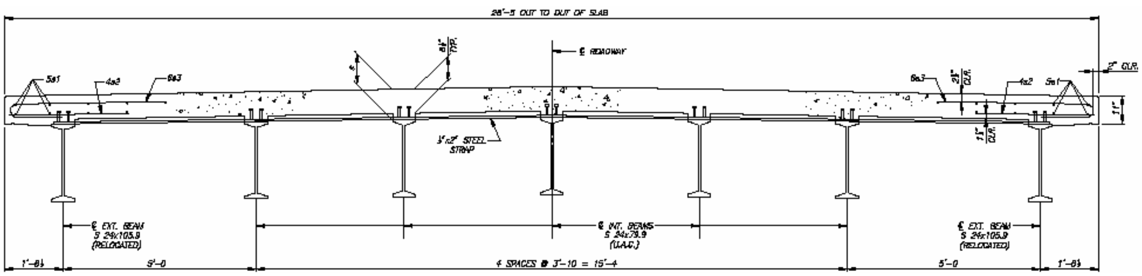

Figure 2. Cross-section view of the Tama County bridge

The steel-free bridge deck was designed using the Canadian Highway Bridge Design Code (CSA International 2000), with the exception of the cantilever portion. The bridge deck cantilever portion and the remainder of the bridge was designed using the American Association for State Highway Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (1996).

The only reinforcement steel placed in the deck concrete was located in the outer four feet of the deck. This epoxy-coated steel was used to provide tension reinforcement in the overhang portion of the deck, where external steel straps were not feasible. Corrosion-resistant fiber reinforced polymer (FRP) bars were placed in the entire deck for temperature and shrinkage reinforcement. The FRP bars were Hughes Brothers, Aslan 100, vinyl ester matrix, glass fiber-reinforced polymer rebar. Figure 3 shows the placement of the bridge deck reinforcement.

Tensile reinforcement for the deck was provided by welding 1/2-in. x 2-in. steel straps transversely across the top flange of the girders. Steel strap placement details are shown in Figures 4 and 5. The reinforcement straps provide tensile restraint below the deck and away from exposure to deicing chemicals. The straps provided lateral restraint to the girders, preventing them from spreading outward, and developing the tensile stresses that form in the deck (Ventura and Cook 1998). The straps were attached at four-foot intervals longitudinally along the girders.

Figure 3. Placement of deck reinforcement

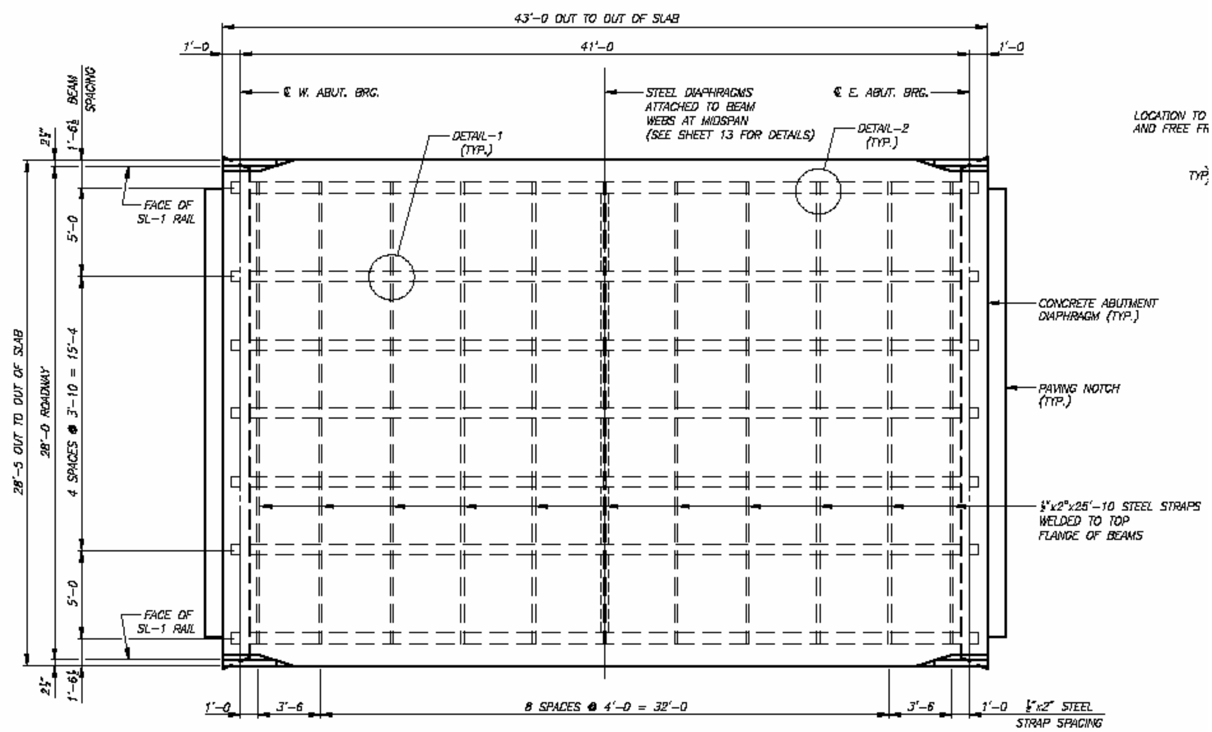

Figure 4. Plan view of the Tama County bridge

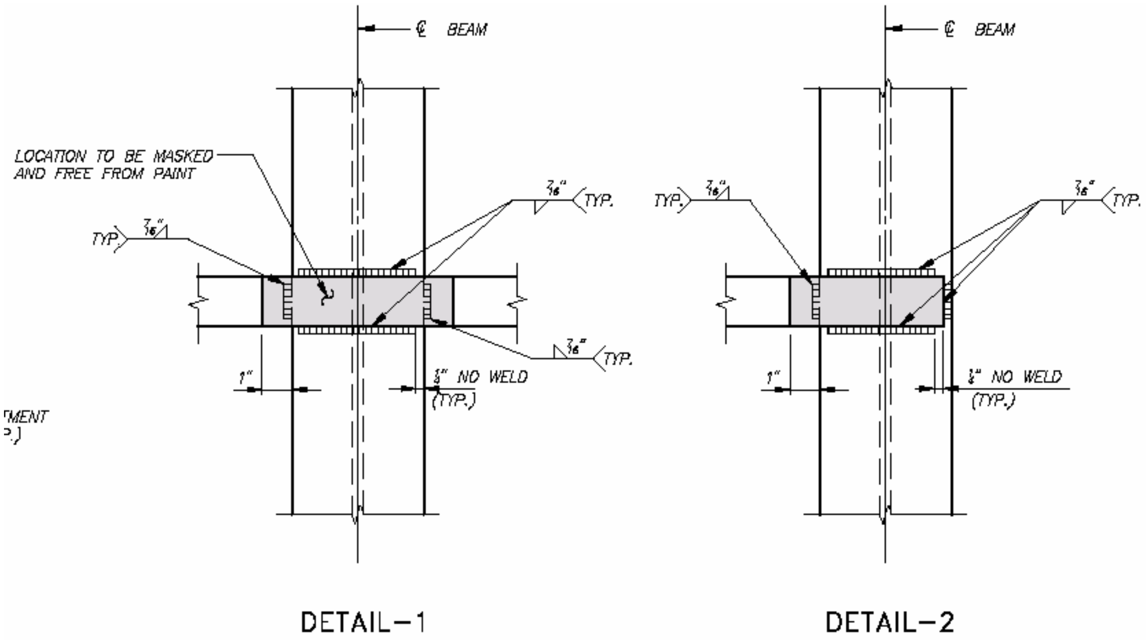

Figure 5. Tensile restraint strap details

INSTRUMENTATION AND EVALUATION METHODOLOGY

Girder deflections were recorded using ratiometric displacement transducers and an Optim Megadac data acquisition system. Strains were measured using the Structural Testing System and Intelliducers from Bridge Diagnostics, Inc. Figure 6 shows the location of the strain gages and displacement transducers.

Strain gages were attached to the underside of the top and bottom flanges of each girder at mid-span. Strain gages were also epoxied to the bottom of the tensile restraint straps at mid-span and at other various locations of interest. Concrete strain was measured in the bottom of the slab in various locations. By observing the level of strain in the straps compared to the level of strain in the slab, the effectiveness of the straps' ability to resist the tensile forces was determined. Figure 7 shows the placement of the strain gages on the structure. Deflection transducers were also placed on the underside of each girder at mid-span.

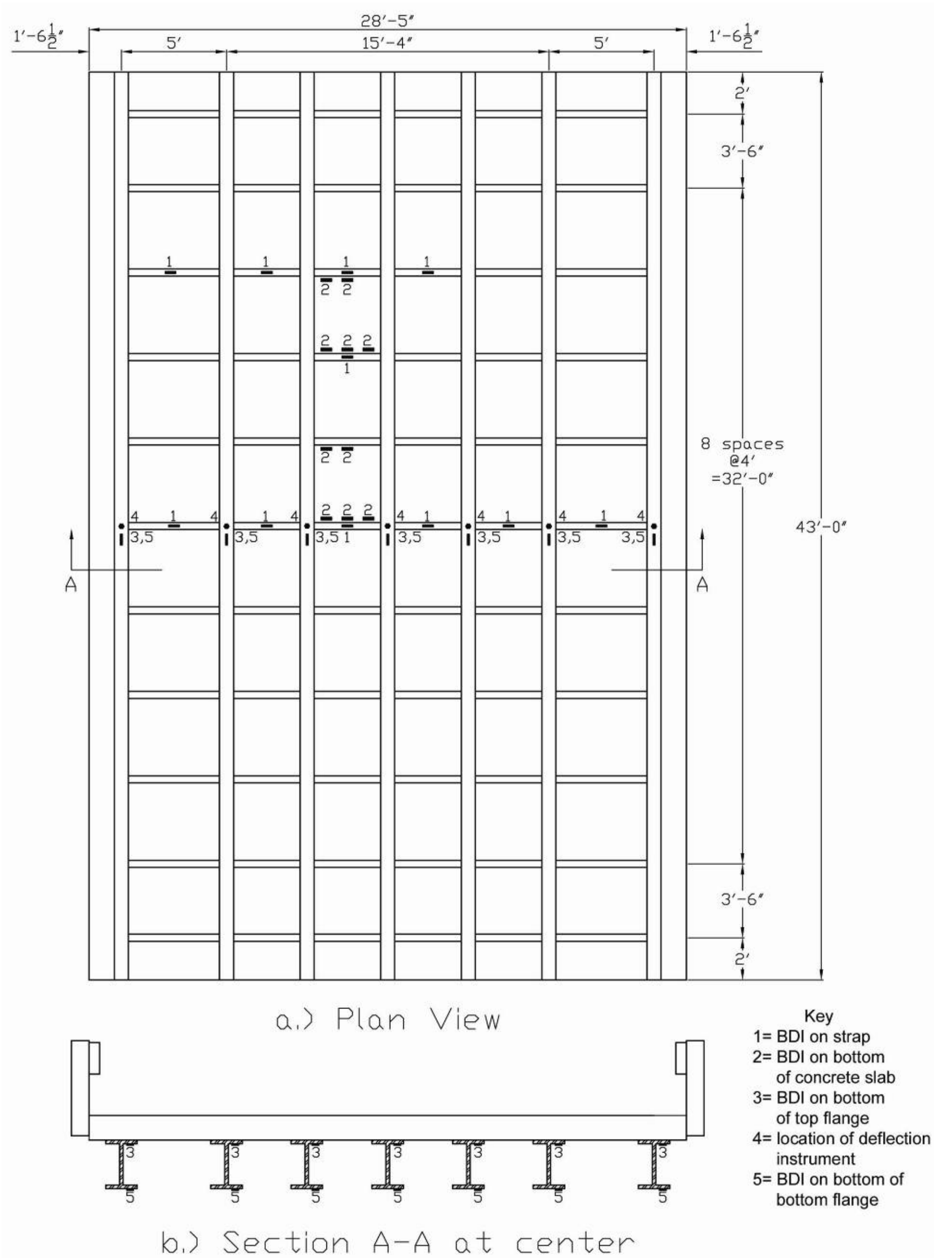

Figure 6. Location of strain gages and displacement transducers

Figure 7. Placement of strain gages

Live Loading

The bridge was loaded using two loaded tandem-axle dump trucks provided by the Tama County Secondary Roads Department. Trucks 1 and 2 weighed 51,680 lbs and 52,860 lbs, respectively. The bridge was loaded under seven different load cases. Two stationary load tests were conducted with both loaded trucks. Five load tests were performed with Truck 2 only, crossing at crawl speed.

Stationary Loading

The stationary loading was performed using both loaded trucks simultaneously. Load Cases 1 and 2 were applied once each. Load Case 1 was applied with the wheel line of each truck placed two feet from the edge of the slab. The two-foot offset was selected because the current AASHTO specifications (AASHTO 1996) dictate this as the minimum offset of the vehicle wheel line from the edge of the slab. The same two-foot offset was used for Load Cases 3 and 5 for single truck loading at crawling speed. Load Case 2 was applied with both trucks placed as close as possible to the center of the bridge, in order to induce the maximum load possible into the structure.

Crawling Loading

The crawling loading was performed with a single loaded truck (Truck 2). Load Cases 3 through 7 were each run twice. In Load Cases 3 and 5, the truck was again placed at a location two feet from the edge of the slab. The truck was centered on the bridge for Load Case 4. Load Case 6 was selected to maximize the load between two girders. The dual tires from the tandem were centered between the girders. Load Case 7 was used to place the duals directly over a girder for the maximum loading of a single girder.

RESULTS

Bridge Deflections

The maximum deflection for each girder ranged from 0.07 inches at Girder 7 to 0.09 inches at Girder 4. The AASHTO Specifications (1996) stipulate that the deflection due to service live load plus impact should not exceed 1/800 of the span length. The deflection limit for this bridge is 0.62 inches. The deflections recorded during the load test were well below the specification limitation for all girders under all load cases.

Deck Strain

The design compressive strength of the concrete was 3,500 psi. Test cylinders broken at 56 days show that the actual compressive strength was over 6,100 psi. The bridge was tested at an age of 120 days, indicating that even higher compressive strength was likely. The maximum tensile strain measured in the bottom of the concrete slab was 36 microstrain. The maximum tensile strain occurred under Load Case 4 in a bay adjacent to mid-span and adjacent to the centerline of the bridge. When converted to stress (assuming E_c=3,372 ksi), the slab experienced a maximum tensile stress of 121 psi, which is roughly half of the modulus of rupture for the concrete (237 psi) (CSA International 2000).

Tensile Restraint Strap Strain

The maximum tensile strain in the tensile restraint straps was 71 microstrain, resulting from Load Case 2. The maximum tensile stress in the straps (assuming E_s=29,000 ksi) was 2.1 ksi, which is less than 6% of the yield stress of the straps (36 ksi). The relationship between concrete strain and tensile restraint strap strain at girder mid-span is shown in Figure 8 for Load Case 6. The strain gages were located at the girder mid-span between Girders 3 and 4.

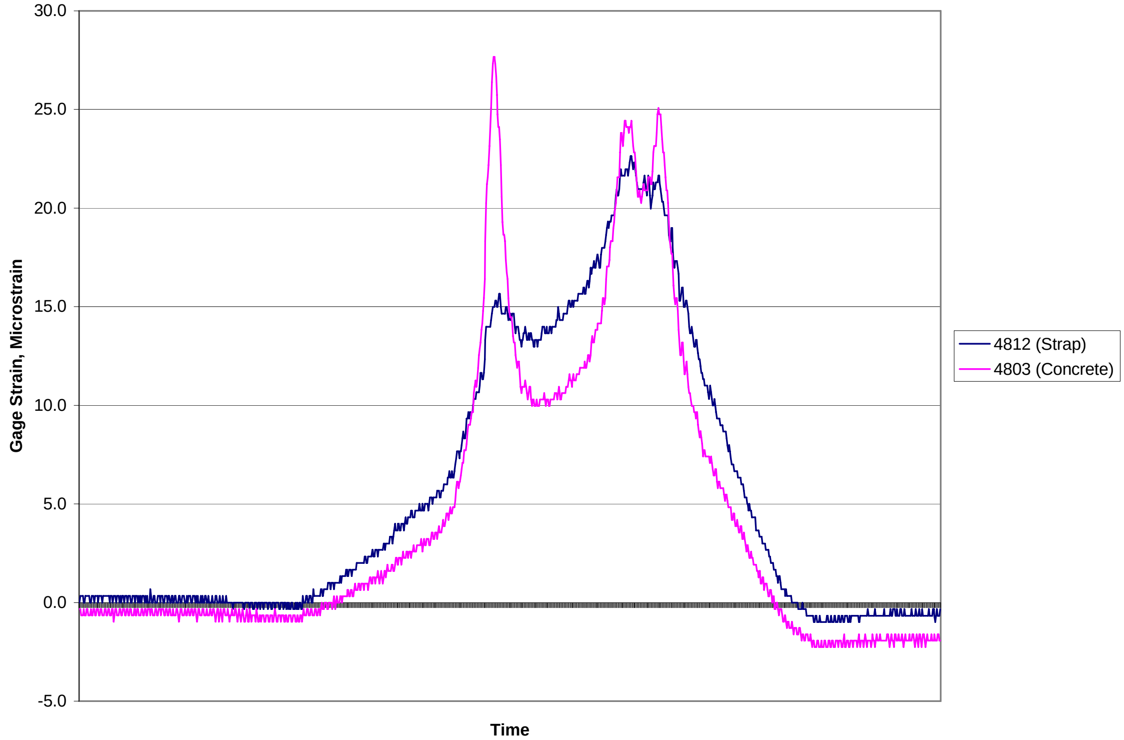

Figure 8. Tensile restraint strap strain vs. concrete strain at girder mid-span

Girder Top Flange Strain

The maximum mid-span tensile flexural strain (14 microstrain) in the girder top flange occurred under Load Case 1 in Girder 1. The maximum mid-span compressive flexural strain (5 microstrain) in the girder top flange occurred under Load Case 7 in Girder 2. Converting to stress shows that the maximum mid-span flexural stress in the girder top flange is 0.4 ksi compression, which is negligible in relation to the yield stress of the girders (30 ksi).

Girder Bottom Flange Strain

The maximum mid-span flexural strain (107 microstrain) occurred in the bottom flange of Girder 1 under Load Case 1. The maximum tensile stress in the girder bottom flange (assuming E_s=29,000 ksi) was 3.1 ksi, which is approximately 10% of the yield stress of the girders (30 ksi).

Lateral Live Load Distribution

The approximate distribution factors were calculated from the measured deflections in the structure at mid-span. Lateral load distribution factors were approximated from Equation 1 using test data from the bridge.

Distribution factor, $DF_i = \dfrac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$ (1)

Where,

DF_i = distribution factor of the *i*th girder (lanes/girder)
Δ_i = deflection of the *i*th girder
$\sum \Delta_i$ = sum of girder deflections
n = number of girders

Load distribution factors were compared to bridge design code distribution factors (AASHTO 1996). The experimental distribution factors for the exterior two girders were higher than the design code assumptions for Load Cases 3 and 5. The AASHTO distribution factors (1996) assume equal girder spacing. The unequal spacing of the exterior girders could account for some of the differences between the design values and the measured values. Another possible cause for the differences is the relatively low deflections that were measured on the bridge. Because the loads for Load Cases 4 through 7 were moving, there may have been some influence of vibration or impact of the load on the distribution of the loads. A small change in girder deflection due to vibration or impact could be significant enough to change the distribution factors computed from the data. Due to the low girder stresses calculated from the load test data, the girders are adequate, despite the slightly un-conservative estimates of load distribution.

SUMMARY AND CONCLUSIONS

This report summarizes the load test and load response of a single-span steel girder bridge with an innovative steel-free bridge deck system. The bridge was tested under seven load cases, two stationary loadings with two loaded trucks and five loadings with one loaded truck moving at crawling speed. The performance of the structure was monitored for strains and displacements at various locations. The response of the structure was compared to the design parameters used for the bridge (CSA International 2000; AASHTO 1996).

The bridge performed well under the loading. Girder deflections at mid-span were well below the limitations for serviceability in the AASHTO design specifications (1996). The measured strains in the girders and tensile restraint straps were all 10% or less of the yield strain of the steel. Concrete slab strains did not approach the cracking strain of the concrete. Overall, the single-span steel girder bridge with a steel-free deck system performed well under the load test conditions.

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Existing Challenges for Use of Passive GPS Devices for Routine Travel Data Collection

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ABSTRACT

Traveler route choice behavior is the cornerstone of numerous advanced traffic management technologies. However, few datasets of actual segment-by-segment travel routes have been collected and analyzed. This is understandable, given the difficulties faced by traditional survey methods, such as phone and mail surveys, when collecting route data.

With the improved accuracy of GPS devices, it is now feasible to collect route data using GPS devices, especially passive GPS devices, which are relatively low-cost, do not require additional computerized equipment, and are more suitable for collecting data for research purposes, given their small size and lack of a screen to distract users. However, despite the widespread availability of GPS technology, there are still very few comprehensive GPS-collected travel route datasets. Collecting data with passive GPS devices is not as straightforward as widely assumed. This is especially true if the data is to be used directly in identifying route choice behavior. When the GPS device is used to collect multiple-day travel data, the post-processing of the data is challenging. Automatic spatial models to identify trip ends and convert point data to link-by-link data is necessary but also extremely time and resource intensive.

This paper will summarize the results of several research projects conducted by the authors to study the methodological possibility of using GPS to collect travel route data. Spatial models were constructed in a GIS environment to accomplish the data processing and conversion. One unique feature of the study is that model calibration was conducted using real known route data so that the accuracy of the models was measured. This work will contribute to better utilization of GPS devices in travel data collection.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: automatic spatial models—GPS data sets—traveler route choice behavior

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Project-Level Air Toxics Analysis

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ABSTRACT

As part of an environmental impact statement for a new interstate river crossing near Stillwater, MN, agency and environmental community concerns over the health impacts of transportation air toxics prompted the Minnesota Department of Transportation to perform an unprecedented project-level analysis. Various studies have shown a correlation between health impacts and proximity to transportation sources of air toxics. Six pollutants (termed priority mobile source air toxics, or MSATs) are considered by the Environmental Protection Agency (EPA) to dominate health risks: diesel particulate matter, Benzene, 1,3-Butadiene, Formaldehyde, Acetaldehyde, and Acrolein. Because of a lack of meaningful standards for permissible concentrations of MSATs, dispersion analysis (such as is done for CO and NOx) was not done. Instead, total emissions were quantified on three scales: regional, local, and corridor.

The EPA MOBILE 6.2 emissions model was run to determine emission rates for the six priority MSATs for various roadway types, speeds, and future analysis years. These were then combined with output from the regional travel demand forecasts to determine the emission of each MSAT for each link in the regional highway transportation network.

The following are selected findings from this research:

- Emission rates of 1,3-butadiene, acetaldehyde, acrolein, benzene, and formaldehyde are generally lower under higher operating speeds. Emissions are therefore sensitive to changes in vehicle miles traveled (VMT) and vehicle hours traveled.
- Direct emission rates of diesel particulates are not sensitive to roadway speed. Emissions are therefore sensitive only to changes in VMT.
- Long-term projected decreases in emission rates over time are much larger than the difference between any alternatives, even on a regionally significant project.
- Differences in total emissions between alternatives can be significant and quantifiable on a small area or corridor scale.
- A well-validated regional travel forecasting model is necessary to perform this type of analysis.

Transportation-related air toxics have become a great concern in the environmental community as a result of the threat to public health that they pose. Assessment of the air quality implications of highway projects will increasingly become dominated by air toxics issues. This study provides a valuable method for quantifying air toxics emissions associated with a project.

Key words: air toxics—emissions—mobile source

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BACKGROUND

This paper documents a study of air toxics emissions performed for the Minnesota Department of Transportation (Mn/DOT). The analysis was performed in conjunction with the Supplemental Final Environmental Impact Statement (SFEIS) for a new St. Croix River crossing near Stillwater, Minnesota. This document is a revision of an earlier analysis (dated February 2005), updated for the current preferred alternative.

In addition to the six criteria mobile-source pollutants (ozone, particulate matter, lead, carbon monoxide [CO], sulfur dioxide, and nitrogen dioxide) that are typically associated with transportation sources, the effects of another broad group of air pollutants known as air toxics have been an emerging concern. The Environmental Protection Agency (EPA) has identified 21 air toxics emitted by vehicles. The EPA has found that among these mobile source air toxics (MSATs), six pollutants dominate health risks. These compounds are known as priority MSATs:

- Diesel particulate matter
- Benzene
- 1,3-butadiene
- Formaldehyde
- Acetaldehyde
- Acrolein

While studies have shown that there may be a correlation between health effects and proximity to roads with dense traffic, there remains a great deal of uncertainty associated with quantifying specific impacts and health risks associated with potential highway projects. For criteria pollutants such as CO, the EPA has issued standards for exposure and accepted methods of modeling concentrations at potential receptor sites. Using these approved techniques, predicted concentrations can be compared with standards and a determination made as to whether a proposed transportation project will cause an air quality impact. However, there are no EPA standards for air toxics. In addition, there is no accepted method for modeling future concentrations at specific receptors.

Air quality analyses performed for highway projects commonly include both emission and dispersion modeling. Emission models quantify pollutant emissions per vehicle for various speeds in future years. Dispersion models use emissions and traffic data to predict pollutant concentrations. The EPA recently released an updated version of their mobile source emissions model (MOBILE 6.2), which includes the capability of performing emission modeling for air toxics. The Federal Highway Administration (FHWA) is currently evaluating the use of dispersion modeling to predict air toxic concentrations adjacent to roadways.

During the inter-agency review process, the Minnesota Pollution Control Agency (MPCA) requested that Mn/DOT perform an air toxics analysis for this project. In researching this issue, Mn/DOT has found no precedent for a project-level air toxics analysis for a similar project. Methods used in this assessment were developed with the assistance of the MPCA and FHWA. The FHWA and EPA are in the process of developing potential methods to address this issue, but have not yet issued any formal guidance. In the absence of established methods or formal guidance, FHWA staff recommended that Mn/DOT use the MOBILE 6.2 emission model to calculate aggregate vehicle emissions in a study area for various conditions. The MPCA further recommended that the scale of the analysis include the road segments most affected by changes in traffic volumes as a result of the project. These methods can compare emission levels resulting from various future conditions on various geographic scales, but because dispersion modeling is not included, results do not yield exposure levels or assess whether changes in emissions are significant.

The FHWA is currently evaluating the use of dispersion modeling to predict air toxics hot spot concentrations adjacent to roadways. However, the FHWA has indicated that dispersion modeling methods for highway projects are not yet adequate and do not currently support dispersion modeling for air toxics.

METHODS

Geographic Areas of Analysis

Three geographic scales, the region, a study area, and individual roadway segments, were used to evaluate air toxics emissions. This was done to balance the desire to account for the entire impact of the project on air toxics emissions while still analyzing impacts on specific areas. To fully capture the effects of changes in traffic patterns to air toxics emissions, impacts in both Minnesota and Wisconsin were included in the analysis at all three scales.

Region

The area modeled for travel demand forecasts included the seven-county Twin Cities Metropolitan Area, southern Chisago County, St. Croix County, southern Polk County, and the portion of River Falls in Pierce County.

Study Area

This study area identifies a geographically well defined area where the greatest difference (between no-build and build) in traffic volumes occurs. It contains the major alternative river crossings to the north and south and major north-south roadways in Minnesota and Wisconsin. The study area accounts for approximately 70% of the difference in vehicle miles traveled between the no-build and build alternatives. The definition of the study area is the area bounded by I-94 on the south; STH 65 on the east; the St. Croix county line and TH 8 to the north; and I-694, Washington CSAH 15, and Chisago CSAH 24 to the west.

Segments

In addition to aggregate emissions calculated for the metro region and the study area, selected road segments were analyzed separately to assess changes in emissions on a smaller scale. The selected segments were chosen based on a ranking of analyzed roadways by the largest changes, either increases or decreases, in vehicle traffic volume from no-build to build conditions. A total of six road segments (listed below) were selected from the top-ranked segments and were analyzed and reported separately. Initially, only five segments were planned for analysis. The sixth (Stillwater Blvd.) was added to the analysis to represent a densely developed residential corridor.

1. I-94 from I-694 to STH 65
2. I-694 from I-94 to TH 36
3. Manning Ave./Stillwater Blvd. from I-94 to TH 36
4. STH 65 from I-94 to STH 64
5. TH 36/STH 64 from I-694 to STH 65 (existing alignment in the no-build alternative and the new alignment in the build alternative)
6. Stillwater Blvd./Olive St. from TH 36 to Main St.

Emission Modeling Using MOBILE 6.2

The currently approved EPA mobile source emissions model is MOBILE 6.2. It is based on empirical vehicle data and incorporates weather conditions, vehicle fleet composition, fuel chemistry, and operating characteristics. The MOBILE 6.2 model was used to obtain emission factors (in grams/mile) for all six priority MSATs for various road speeds and types (freeway, arterial, ramp, and local roads).

Per MPCA approval, the local area-specific input variables listed below were used; default MOBILE 6.2 input values were used for all other variables. A different MOBILE 6.2 model is used to produce emission factors for particulate matter than the model used for the other air toxics, so some input values may not apply to all pollutants.

Atmospheric Variables

- Absolute Humidity: 75.0 grains/lb
- Altitude: Low Altitude
- Evaluation month:
  - July (summer) and January (winter)
- Temperature:
  - Summer:
    - Minimum: 72° F
    - Maximum: 92° F
  - Winter
    - Minimum: 16° F
    - Maximum: 38° F

Fleet Variables

- Vehicle age: Based on data provided by the MPCA on August 9, 2004

Fuel Variables

- Gasoline
  - Fuel program: Conventional Gasoline East
  - Oxygenated fuels: Alcohol with a 99.9% Market Share and 2.7% Oxygen Content
  - Reid vapor pressure: 9.0 lbs/in.^{2}
  - Aromatics: 18.5% by volume
  - Benzene: 0.8% by volume
  - Olefin: 7.1% by volume
  - Percent evaporated at 200° F: 50.8%
  - Percent evaporated at 300° F: 85.6%
- Diesel
  - Diesel sulfur
    - Winter: 290 parts-per-million
    - Summer:
      - 2000: 300 parts-per-million
      - 2010: 43 parts-per-million
      - 2030: 43 parts-per-million

Reported particulate matter emissions are the total exhaust PM 2.5 component of sulfate, organic carbon, and elemental carbon. Other reported air toxic emissions are the total gaseous emissions of that pollutant.

The MOBILE 6.2 model was run for four roadway types (arterial, freeway, ramp, local), sixteen speeds (2.5, 3, 4, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, and 65 mph), and three analysis years (2000, 2010, and 2030). The results of the analysis are presented in the appendix.

Traffic Modeling

The MOBILE 6.2 model produces emission factors by facility type and speed. Facility types, congested speeds, and hourly traffic volumes were based on the regional travel demand forecast model, which was used to produce traffic forecasts for the project. Emission factors generated from the MOBILE 6.2 model, combined with traffic volume and speed information from the travel forecasting model, was used to calculate aggregate emissions within the study area or segment for the priority MSATs.

RESULTS

In general, emission rates for the priority MSATs (benzene, 1,3-Butadiene, Formaldehyde, Acetaldyde, and Acrolein) produced by the model are lower at higher operating speeds. MOBILE 6.2 emission rates for particulates are not sensitive to speeds or road types.

Results of the emissions analysis are presented in Tables 1 through 8. For each pollutant, an existing (year 2000) emission rate is listed along with predicted rates for years 2010 and 2030 no-build and build conditions. While emission rates vary considerably among pollutants, each pollutant shows a similar trend of decreasing emission rates significantly over time. Also similar is the relatively small difference in emission rates between the no-build and build conditions. Results for the segment analyses also include vehicle miles traveled per mile of segment and average speed.

Table 1. Regional air toxics emissions (grams/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	630,779	335,509	334,716	212,767	212,264
Acrolein	89,682	44,155	44,045	27,136	27,068
Benzene	4,398,507	2,409,044	2,405,670	1,480,471	1,478,398
1,3 Butadiene	608,289	290,668	290,212	177,280	177,001
Formaldehyde	1,799,824	926,375	924,203	587,049	585,673
Diesel particulates	2,689,176	1,366,923	1,367,026	354,029	354,056

Table 2. Study area air toxics emissions (grams/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	42,032	20,538	20,471	17,245	17,143
Acrolein	5,929	2,692	2,675	2,197	2,183
Benzene	295,404	150,007	149,784	121,109	120,928
1,3 Butadiene	40,766	18,034	17,985	14,472	14,432
Formaldehyde	119,808	56,674	56,332	47,578	47,291
Diesel particulates	184,548	87,846	88,567	29,606	29,849

Table 3. Air toxics emissions for Segment 1 (grams/mile/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	238	128	115	105	94
Acrolein	32	17	15	13	12
Benzene	1,732	970	880	762	691
1,3 Butadiene	236	116	105	91	82
Formaldehyde	675	353	317	289	259
Diesel particulates	1,135	597	551	193	178
Vehicle miles	1,112,794	1,544,752	1,439,571	2,159,932	1,993,296
Average speed (mph)	63	59	61	46	55
Segment length (miles)	19	19	19	19	19

Table 4. Air toxics emissions for Segment 2 (grams/mile/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	258	130	132	87	88
Acrolein	35	17	17	11	11
Benzene	1,878	989	1,004	640	650
1,3 Butadiene	256	118	120	76	77
Formaldehyde	731	359	364	239	243
Diesel particulates	1,224	604	614	165	167
Vehicle miles	307,685	400,636	409,098	472,850	480,272
Average speed (mph)	57	54	54	51	51
Segment length (miles)	5	5	5	5	5

Table 5. Air toxics emissions for Segment 3 (grams/mile/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	72	28	32	19	22
Acrolein	10	4	4	2	3
Benzene	518	208	237	135	154
1,3 Butadiene	71	25	28	16	18
Formaldehyde	205	77	88	52	59
Diesel particulates	334	125	143	34	39
Vehicle miles	109,324	107,620	110,146	128,807	147,194
Average speed (mph)	44	42	42	39	40
Segment length (miles)	7	7	7	7	7

Table 6. Air toxics emissions for Segment 4 (grams/mile/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	44	22	17	19	15
Acrolein	6	3	2	2	2
Benzene	315	162	128	139	109
1,3 Butadiene	43	19	15	16	13
Formaldehyde	125	60	46	53	41
Diesel particulates	204	98	78	35	28
Vehicle miles	122,901	155,734	111,978	243,268	193,103
Average speed (mph)	46	45	46	41	44
Segment length (miles)	12	12	12	12	12

Table 7. Air toxics emissions for Segment 5 (grams/mile/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	89	40	60	29	43
Acrolein	12	5	8	4	6
Benzene	638	301	453	209	314
1,3 Butadiene	88	36	54	25	37
Formaldehyde	254	111	166	80	120
Diesel particulates	409	183	275	53	80
Vehicle miles	514,507	642,112	872,541	807,576	1,125,909
Average speed (mph)	44	44	47	38	42
Segment length (miles)	27	27	25	27	25

Table 8. Air toxics emissions for Segment 6 (grams/mile/day)

Pollutant	2000	2010		2030	
		No-build	Build	No-build	Build
Acetaldehyde	63	25	18	17	13
Acrolein	9	3	3	2	2
Benzene	426	181	132	119	86
1,3 Butadiene	59	22	16	14	10
Formaldehyde	179	70	50	48	34
Diesel particulates	256	105	77	30	22
Vehicle miles	38,578	41,937	32,067	50,725	37,190
Average speed (mph)	30	34	35	32	34
Segment length (miles)	3	3	3	3	3

CONCLUSIONS

Trends in Emission Rates

The results presented above show that on-road emissions of air toxics are expected to decline over the next two decades. Despite growth in overall traffic volume, improvements in vehicle emission controls and fuel re-formulation will result in reduced emissions of air toxics. Modeling results show that year 2010 emission rates will be 45% to 56% lower than year 2000 emission rates. Year 2030 emission rates will be 59% to 71% lower than year 2000 emission rates for the five gaseous priority MSATS while diesel particulate emission rates for year 2030 will be 84% to 87% lower than 2000 emission rates.

Project Effects

Project effects on air toxics emissions are assessed by comparing emission rates for the no-build and build conditions. For large areas, the project effects are predictably small due to the dilution of local traffic changes in a larger road system. The results of this study also show a pronounced reduction in long-term emission rates for air toxics. Differences between the no-build and build conditions are relatively small compared to long-term trends. Over a large area, the project would reduce emission levels from the no-build condition. On a smaller scale, traffic pattern change could increase emissions in some areas while reducing emissions in others.

This study does not attempt to predict concentrations of air toxics and should not be used to assess whether changes in emission rates between the no-build and build conditions are significant. Therefore, no specific conclusions can be drawn regarding the potential for adverse effects as a result of the project.

Metro Area Emission Rates

On a metro area scale, construction of the project would reduce gaseous priority air toxics emission rates for the years 2010 and 2030 by 0.14% to 0.25% compared to the no-build condition. Diesel particulates would be increased by 0.01% as a result of the project.

Study Area Emission Rates

Construction of the project would reduce gaseous priority air toxics emission rates for the year 2010 by 0.15% to 0.64%. Year 2030 emission rates for the five gaseous priority MSATS would also be reduced by 0.15% to 0.63%. Diesel particulates would be increased by 0.82% as a result of the project.

Emission Rates along the Top Six Segments

Changes in emission rates for the six studied road segments are discussed in the list below. Long-term trends are similar to those discussed for the study area and metro area, so this discussion addresses the comparison between the no-build and build conditions.

Segment 1: I-94 from I-694 to STH 65

Construction of the project would reduce traffic on this segment, thereby reducing MSAT emissions by 8% to 10%. The resulting 2030 emissions would be approximately 65% below year 2000 levels.

Segment 2: I-694 from I-94 to TH 36

Construction of the project would increase traffic on this segment and would increase MSAT emissions by 1% to 2%. The resulting 2030 emissions would be approximately 70% below year 2000 levels.

Segment 3: Manning Ave./Stillwater Blvd. from I-94 to TH 36

Construction of the project would increase traffic on this segment and would increase MSAT emissions by 14% to 16%. The resulting 2030 emissions would be approximately 75% below year 2000 levels.

Segment 4: STH 65 from I-94 to STH 64

Construction of the project would reduce traffic on this segment, thereby reducing MSAT emissions by 21% to 22%. The resulting 2030 emissions would be approximately 70% below year 2000 levels.

Segment 5: TH 36/STH 64 from I-694 to STH 65

Construction of the project would increase traffic on the existing portion of this segment and would construct a new alignment on a portion of this segment. Increased traffic on this segment would increase MSAT emissions by 50% to 51%. The resulting 2030 emissions would be approximately 60% below year 2000 levels.

Segment 6: Stillwater Blvd./Olive St. from TH 36 to Main St.

Construction of the project would reduce traffic on this segment, thereby reducing MSAT emissions by 23% to 28%. The resulting 2030 emissions would be approximately 85% below year 2000 levels.

Cooperative Development of Wetland Mitigation Projects in Iowa

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ABSTRACT

Many transportation improvement projects near wetlands, such as bridge replacement, road widenings, geometric changes to improve safety, or roadway realignments, involve unavoidable but small impacts to streams and wetlands. For unavoidable wetland impacts of five acres or less, it is difficult to purchase a parcel adjacent to the project area or within the watershed to mitigate the wetland loss, leading to high project costs and potential project delays. A mechanism for paying a fee to an established wetland in lieu of building a small and (probably ineffective) mitigation site would be advantageous. This paper considers the need for cooperative wetlands mitigation in Iowa and activities to advance toward this goal.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: cooperative mitigation—environmental impacts—wetlands

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Development of Self-Consolidating Concrete for Bridge Girders and Evaluation of Its Fresh Properties

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ABSTRACT

Self-consolidating concrete (SCC) has been developed for use in precast prestressed concrete bridge girders in the state of Minnesota. Locally available materials were used with a number of cementitious and filler materials, including ASTM Type III cement, Class C fly ash, and blast furnace slag. SCC was successfully proportioned with both natural river gravels and crushed stone as coarse aggregates. Moreover, for the mix incorporating natural river gravels, air-entrained SCC was successfully developed without using a viscosity enhancing admixture. The effect of a number of parameters on the fresh properties of SCC, including but not limited to temperature, cement, type of coarse aggregates (natural and crushed), were studied.

A number of test methods (e.g., slump flow, L-box, and U-box) have been under development to evaluate the fresh properties of SCC. However, none of these test methods has been integrated into any American standards. A vertical column segregation test has been used to evaluate vertical segregation of SCC mixes. The slump flow test was employed to evaluate the flowability, while self-leveling and passing abilities of the mixes were investigated using a U-box. The L-box test procedure was modified to evaluate, not only flowability and passing ability, but also horizontal segregation resistance of SCC mixes. Although, in general, at least three to four test methods are typically used to evaluate fresh properties, the slump flow test and modified L-box test may be adequate to evaluate the fresh properties of SCC properly.

Key words: high-range water reducer—segregation—self-consolidating concrete

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INTRODUCTION

Self-consolidating concrete (SCC), which was originally developed in Japan due to a shortage of skilled labor and poor compaction of ordinary concrete, is a concrete mix that flows and fills the formwork under its own weight without mechanical vibration (Ramage, Kahn, and Kurtis 2004). In other words, SCC is required to fill the formwork with a void-free structure through congested reinforcement without segregation of its constituent materials.

Although SCC is a relatively recent development, it has demonstrated substantial economic and environmental benefits in terms of faster construction, reduction in manpower, better surface finish, easier and vibration-free placing, reduced noise level, and a safer working environment. Therefore, SCC has recently found a wide use in many countries for different applications and structural configurations (Lachemi et al. 2003). For example, SCC has been successfully pumped using a 250-foot pipeline for construction of a heavily reinforced tunnel in Yokohama City, Japan (Takeuchi, Higuchi, and Nanni 1994). Other areas where SCC is employed involve the filling of formwork with restricted access for consolidation of concrete. For instance, 2,745 yd^3 of ready-mix SCC was successfully used in the construction of 180 columns at the expansion of the Pearson International Airport in Toronto (Lessard, Talbot, and Baker 2002). Because there was insufficient overhead clearance to allow placement and consolidation of conventional concrete, SCC was the only solution (Lessard, Talbot, and Baker 2002).

The main challenge when producing SCC is not only to obtain sufficient flowability and stability, but also sufficient robustness, which is the insensitivity of SCC to small changes in constituent material properties and mix proportions (Hammer et al. 2002). Robustness of SCC is important, especially for precast concrete plants where large quantities of concrete are produced daily. Therefore, the proportioned SCC should be robust enough such that small variations in physical and chemical properties of constituent material will not significantly affect the fresh properties. Moreover, some variables such as free water content can fluctuate during production on a given day. Therefore, fresh properties of a good SCC mixture should not be sensitive to small fluctuations in the mixture proportions (Daczko 2002). Otherwise, whenever there is a small variation in material properties, new mixture designs would need to be developed and tested. This is neither economical nor feasible for precast concrete plants, where continuous production is required.

The required fresh properties of SCC, which are adequate flowability, passing and filling abilities, and segregation resistance, are achieved by effective proportioning of constituent materials and related admixtures. In the design of SCC, high-range water reducing (HRWR) admixtures are essential to achieve required flowability and high concrete strength (minimized water-cementitious material [w/cm] ratio). Stability of SCC is achieved with the selection of compatible constituent materials (i.e., cementitious material and aggregates), constituent material proportion, and viscosity-enhancing admixture (VEA) (Daczko 2002).

Because SCC consolidates without the help of any external force, such as mechanical vibration, it is the fresh properties of SCC that control the quality of the placement. Moreover, when the fresh state of SCC displays signs of segregation and insufficient flowability and deformability, the concrete will not perform as expected (e.g., poor mechanical and aesthetic properties). Therefore, it is essential to evaluate the fresh properties of SCC properly. Based on the existing literature, slump flow, visual stability index (VSI), L-box, U-box, V-funnel, mortar V-funnel, J-ring, filling capacity, and column segregation tests are some of the available testing methods used to evaluate fresh properties. Although a large number of test methods are currently available, none of them is incorporated into any American standards. Moreover, there is no single testing method that is adequate by itself to qualify a mix as SCC. In general, three to four test methods are used in conjunction to evaluate SCC mixes.

This paper outlines the preliminary results of a research project aimed at producing SCC using locally available materials from two precast concrete plants. The sensitivity of developed SCC mixtures to cement source, w/cm ratio, HRWR dosage, and temperature was studied. Observations of how these parameters can impact the mix proportions for precast applications are presented. This paper also discusses various ways that fresh properties of SCC can be evaluated. A modified L-box testing procedure, which may also help evaluate the segregation resistance of SCC, is discussed. The effect of U-box test filling height on the test results is also discussed.

RESEARCH METHODOLOGY

Cementitous Materials

For both plants, two sets of SCC mixtures were tested and evaluated. ASTM Type III cement was the only cementitous material used for the first sets of mixes. However, the plants were using different cement suppliers, and therefore two sources of Type III cement were used in this study. Moreover, the cement used for Plant A was obtained in four different shipments, which were designated as AS1, AS2, AS3, and AS4. For Plant B, the cement was obtained in a single shipment, which was designated as BS1. The chemical and physical properties from the mill reports of the cements from each shipment are given in Table 1, with the exception of AS4, for which no data are available currently. For the second set of SCC mixtures, pozzolanic materials were also used. Class C fly ash was used for Plant A, and blast furnace slag was used as the supplementary cementitious material for Plant B.

Table 1. Chemical and physical properties of cement

	Plant A				Plant B
Particular	**AS1**	**AS2**	**AS3**	**AS4**	**BS1**
Silicon dioxide (SiO_2), %	20.12	20.4	20.7		20.57
Aluminum oxide (Al_2O_3), %	4.81	5.14	5.31		4.82
Iron oxide Fe_2O_3, %	2.06	1.95	1.95		2.14
Calcium oxide (CaO), %	64.32	63.79	64.81		64.04
Magnesium oxide (MgO), %	1.94	2.24	1.67		2.42
Sulfur trioxide (SO_3), %	3.88	3.95	3.61		3.11
Sodium oxide Na_2O, %	0.22	0.25	0.29		0.25
Potassium oxide (K_2O), %	0.48	0.49	0.41		0.49
Mangaan trioxide (Mn_2O_3), %	0.04	0.04	0.03	N/A*	0.04
C_3S, %	62.55	56.04	57.75		60.04
C_2S, %	10.51	16.22	15.79		13.67
C_3A, %	9.26	10.31	10.77		9.16
C_4AF, %	6.27	5.94	5.94		6.52
Equivalent alkali (Na_2O)	0.54	0.57	0.56		0.57
Lime saturation factor	101.5	98.96	98.94		99.02
Al_2O_3/ Fe_2O_3	2.33	2.63	2.72		2.25
Blaine fineness, m^2/kg	N/A	593	563		644

* N/A = not available

Aggregates

For Plant A, two types of natural gravels with nominal maximum particle size of 3/4 inches and 3/8 inches were used as coarse aggregates. The bulk-specific gravity of these aggregates was 2.72, and their absorptions were 1.0% and 1.5%, respectively. Locally available natural sand with 2.71 bulk specific gravity, 3.3 fineness modulus, and 0.9% absorption was used. For Plant B, two types of crushed limestone with nominal maximum particle sizes of 3/4 inches and 1/2 inches were used as coarse aggregates, and natural sand with 3.2 fineness modulus was used as fine aggregate. The specific gravity and absorption values of the coarse aggregates and sand were 2.71 and 2.65, and 1.3 % and 1.2%, respectively.

Admixtures

Different admixtures were used for each plant. For Plant A, two different polycarboxylate-based high-range water-reducing admixtures were used at equal dosages of 9.8 fl oz/cwt. A fixed set-retarding agent (SRA) at a dosage of 0.98 fl oz/cwt was used for all mixtures to reduce the loss of fluidity. Also, a resin type air-entraining admixture (AEA) was employed at a fixed dosage of 0.37 fl oz/cwt. For Plant B, a polycarboxylate-based high-range water-reducing admixture, which was a different admixture than that used by Plant A, but from the same manufacturer, was the only admixture, and it was employed at a fixed dosage of 9.5 fl oz/cwt. All admixtures were provided by the same manufacturer for both plants. No VEA was used.

Mixture Proportion

As summarized in Table 2, except for mix B2-BS1-S, which incorporated slag, the investigated mixtures were prepared with ASTM Type III cement as the only cementitious material. The naming convention for the mixtures was coded according to the following scheme: X-Y-Q, where X represents the plant that provided the coarse and fine aggregates, Y represents the cement provider and shipment number (Table 1), and Q represent the modification from the plants reference mixture (i.e., WR for a change in the amount of HRWR, w for a change in the amount of water, and S for the addition of slag).

Mixtures A-AS1, A-AS2, A-AS3, A-AS4, and A-BS1 had the same mix proportions and same types of materials and dosage of admixtures. However, cement from different shipments was used for each mix to study the effect of cement shipment on SCC flowability. Mixtures B-S1 and B-S1-S were prepared using crushed coarse aggregates, cement, and admixtures obtained from Plant B. Mixture A-AS2-WR1, A-AS2-WRw2, and A-AS2-WRw3 were mixes proportioned with the same materials except w/cm and HRWR dosage. The HRWR dosage and/or w/cm were modified for each mix to have SCC mixes with different slump flow values to study the effect of flowability and filling height on U-box test results. Mixture A-AS2-WRw2 was also used as the reference mixture to study the effect of concrete temperature on SCC flowability. The mixtures A-AS3-w1, A-AS3-w2, A-AS3-WRw3, and A-AS4-WRw were mixtures designed to study the effect of HRWR dosage on flowability. The constituent materials were the same, with additional HWRW and/or water added to the reference mixture as indicated in the mix name. A-AS4-WRw was a similar mix in terms of the type and proportion of the constituent materials, but cement from a different shipment (AS4) was used.

Table 2. Mixture proportions of tested SCC

Mixture no.	Constituent Materials (lbs/yd^3)						Admixtures (oz/cwt)				
				Aggregates			HRWR				
	Water	Cement	Slag	CA I	CA II*	Fine	I	II	III	AEA	SRA
A-AS1	277	800	0	833	819	1289	9.8	9.8	0	0.37	0.98
A-AS2	277	800	0	833	819	1289	9.8	9.8	0	0.37	0.98
A-AS3	277	800	0	833	819	1289	9.8	9.8	0	0.37	0.98
A-AS4	277	800	0	833	819	1289	9.8	9.8	0	0.37	0.98
A-BS1	277	800	0	833	819	1289	9.8	9.8	0	0.37	0.98
B-BS1	275	773	0	861	655	1473	0	0	9.5	0	0
B-BS1-S	274	539	231	857	655	1467	0	0	9.5	0	0
A-AS2-WR1	277	800	0	833	819	1289	10.7	10.7	0	0.37	0.98
A-AS2-WRw2	312	779	0	811	798	1256	10.7	10.7	0	0.37	0.98
A-AS2-WRw3	337	765	0	796	783	1232	11.7	11.7	0	0.37	0.98
A-AS3-w1	266	807	0	840	826	1300	9.8	9.8	0	0.37	0.98
A-AS3-w2	293	791	0	824	809	1274	9.8	9.8	0	0.37	0.98
A-AS3-WRw3	312	779	0	811	798	1256	7.8	7.8	0	0.37	0.98
A-AS4-WRw	293	791	0	824	809	1274	7.8	7.8	0	0.37	0.98
A-AS3	342	761	0	792	779	1226	11.7	11.7	0	0.37	0.98

* Nominal maximum particle size was 3/8 in. for Plant A and 1/2 in. for Plant B

Mix Procedure

All mixtures were prepared in a drum mixer with a 3.5-ft^3 capacity. The mixing sequence consisted of homogenizing fine and coarse aggregates for about one minute before introducing premixed water with AEA. After one minute of mixing, cementitious materials were added, and the mixture was mixed for a further three minutes. After three minutes of mixing, HRWR and SRA were added. Then, the concrete was allowed to rest for three minutes to allow the admixtures to initiate. At the end of the rest, the concrete was remixed for two additional minutes.

TEST METHODS

The various tests were conducted in the following sequence: slump flow and visual assessment, L-box, U-box, and column segregation tests. The time required to carry out those tests was limited to 20 minutes total. The description and testing procedure of the test methods are given in the following sections.

Slump Flow, Visual Stability Index Rating, and T-20 Inch Test Methods

The slump flow test is used to assess the horizontal free flow of SCC in the absence of obstruction (PCI 2003). The slump cone can be used in either the upright or inverted position, because the values of slump flow are nearly the same for both cases (PCI 2003). In this study, the slump cone was used in the upright position throughout the experiments. The slump flow table was made of a 1/2-inch thick plexiglass sheet attached to a stiff wooden base plate. VSI is a rating involving the visual assessment of the slump flow patty to evaluate several parameters as an indication of the stability of SCC (PCI 2003). The mixtures were rated in 0.5 increments by visual examination according to guidelines provided by PCI (PCI 2003). T-20 inch is the time the concrete takes to reach the 20-inch diameter circle drawn on the slump plate, after starting to raise the slump cone. T-20 inch time, which is a secondary indication of flow, can preliminarily be used as an indication of production uniformity of a given SCC mixture (PCI 2003).

U-box Test

This test was developed for evaluating the self-compatibility and filling ability of SCC in heavily reinforced areas (PCI 2003). The apparatus consists of a vessel that is divided by a middle wall into two components, shown in Figure 1. A sliding gate is fit between the two sections, and three No.4 reinforcing bars are installed just to the left of the gate with center-to-center spacing of two inches. The left-hand section of the apparatus is filled in one lift of concrete, and after one minute of rest the sliding gate is opened, allowing concrete to flow into the other compartment. When the concrete flow stops, the height of concrete in each compartment is measured. The results are presented as the ratio of the concrete heights before (h_1) and after (h_2) the gate (i.e., h_2/h_1) (see Figure 1).

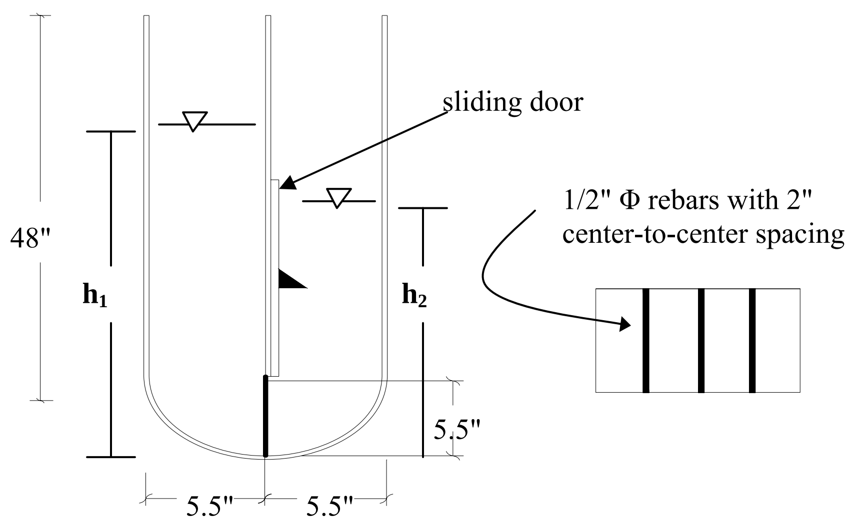

Figure 1. Modified U-box apparatus

The U-box apparatus used in this study was slightly different than that proposed by PCI (2003). The height of the filling component was increased from 24 inches to 48 inches to study the effect of filling height on the results. The U-box apparatus recommended by PCI (2003) has a total height of about 24 inches, and about 0.67 ft^3 of SCC is required to perform the test. Due to the large volume of concrete used, the apparatus is difficult to handle and subsequently clean (Ramage, Kahn, and Kurtis 2004). SCC mixtures A-AS2-WR1, A-AS2-WRw2, and A-AS2-WRw3, which had slump flow values of 19.5, 24.5, and 27.5 inches, respectively, were proportioned to study the effect of flowability and filling height. The U-box test was performed at four different filling heights: 48, 36, 24, and 18 inches. The h_2/h_1 values computed for each mixture and filling height are shown in Figure 2. The sliding door did not operate properly when the test was performed with a filling height of 18 inches for A-AS2-WR1. Except for the h_2/h_1 computed for that case, the results indicate that h_2/h_1 is not very sensitive to U-box filling height for SCC mixtures with poor, moderate, and good flowability. Also, the results show that the test results are less sensitive to U-box filling height as slump flow increases.

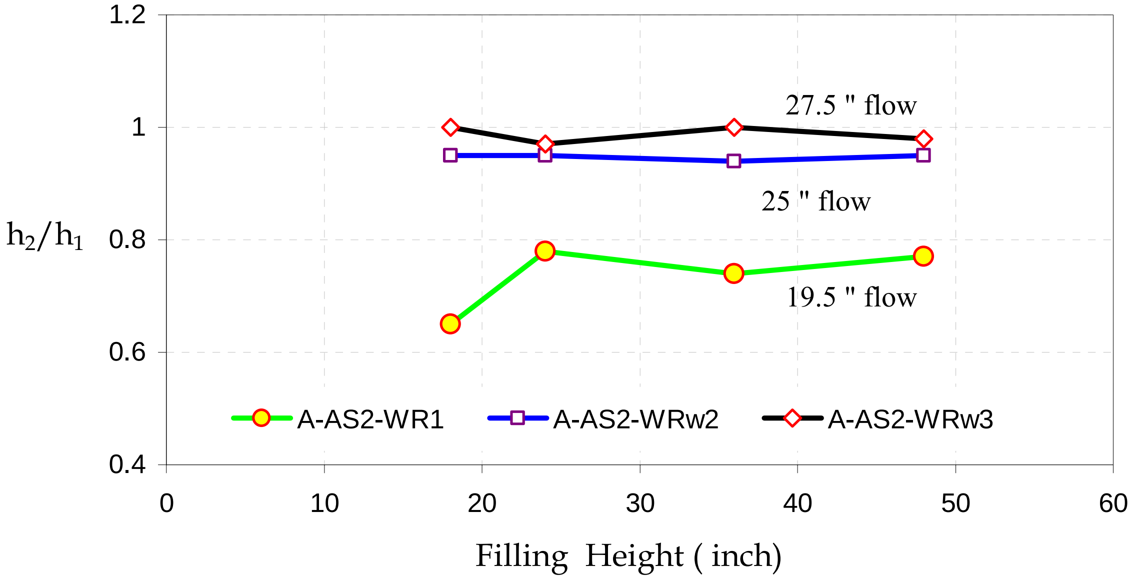

Figure 2. Relationship between U-box filling height and h_2/h_1 value of U-box

Column Segregation Test

This test method is intended to provide the user with a procedure to determine the vertical segregation and stability of SCC. The original apparatus (Brameshuber and Uebachs 2002) consists of 8-inch diameter, 26-inch high PVC Schedule 40 pipe that is separated into four equal sections each measuring 6.5 inches in height. However, this apparatus was modified, and the top 6.5-inch section was further divided into two sections measuring 2 inches and 4.5 inches. The 2-inch section was placed at the top to measure the segregation resistance for the top 2 inches of the column.

The mold was slightly overfilled in one lift. The surface of the concrete was leveled at the top of the mold by means of both lateral and horizontal motion of a thin steel plate (less than 1/16 inches in thickness). The same steel plate and technique was used to separate the column sections after a rest of 10 to 15 minutes. Concrete in each section was placed in containers, and the weight of the concrete was measured for each section. The concrete was then wet-washed through a No. 4 sieve, leaving the coarse aggregate on the sieve, which was subsequently oven-dried and measured for each column section.

The vertical segregation resistance was evaluated by means of a stability weight index (SWI) and stability volume index (SVI), which was defined as the oven-dried weight of coarse aggregate per unit weight of concrete and per unit volume of concrete relative to the base PVC section, respectively (i.e., section S1 in Figure 3).

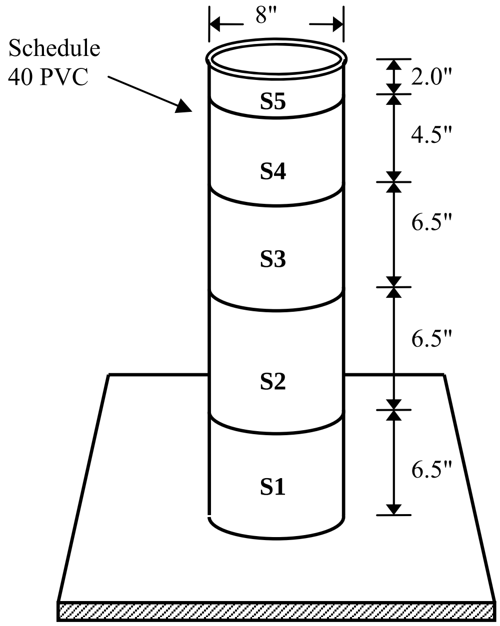

Figure 3. Column segregation test apparatus

L-box Test

This test assesses the flowability of SCC, and the extent to which it is subjected to blocking by reinforcement. The L-box test consists of an L-shaped apparatus (Figure 4). The vertical and horizontal sections are separated by a movable gate, in front of which a reinforcing bar obstacle is placed (Khayat, Assaad, and Daczko 2004). The vertical section is filled with concrete and left to rest for one minute. Then, the gate is lifted and concrete flows under its own weight through the reinforcement into the horizontal section. The concrete heights in the vertical section (h_1) and at the end of horizontal section (h_2) are determined. The h_2/h_1 value, which is the blocking ratio (PCI 2003), is calculated to evaluate the self-leveling characteristic and the degree to which the passage of the mixture through the obstacle is restricted.

The L-box test procedure was modified so that more information regarding the horizontal segregation resistance of the concrete could be obtained. For this purpose, the horizontal section was divided into three sections, each about 8.7 inches in length (see Figure 4). When the flow ceased, the concrete height was measured at a minimum of six points along the flow direction to determine the volume of concrete in each section. After allowing the concrete to sit for 5 to 10 minutes, thin steel plates (less than 1/16 inches in thickness) were used to separate each section. Then, the gate at the end of the horizontal section was opened, and concrete in each section was placed in containers. As soon as the concrete in each section (including the vertical section) was removed, the weight of concrete in each section was measured, and the concrete was wet-washed through a No. 4 sieve, leaving the coarse aggregates on the sieve. After the coarse aggregates were oven-dried, the weight of the coarse aggregates in each section was determined. The horizontal segregation resistance was evaluated by means of SWI and SVI. The SWI and SVI values are calculated relative to the base vertical section (i.e., section LV in Figure 4).

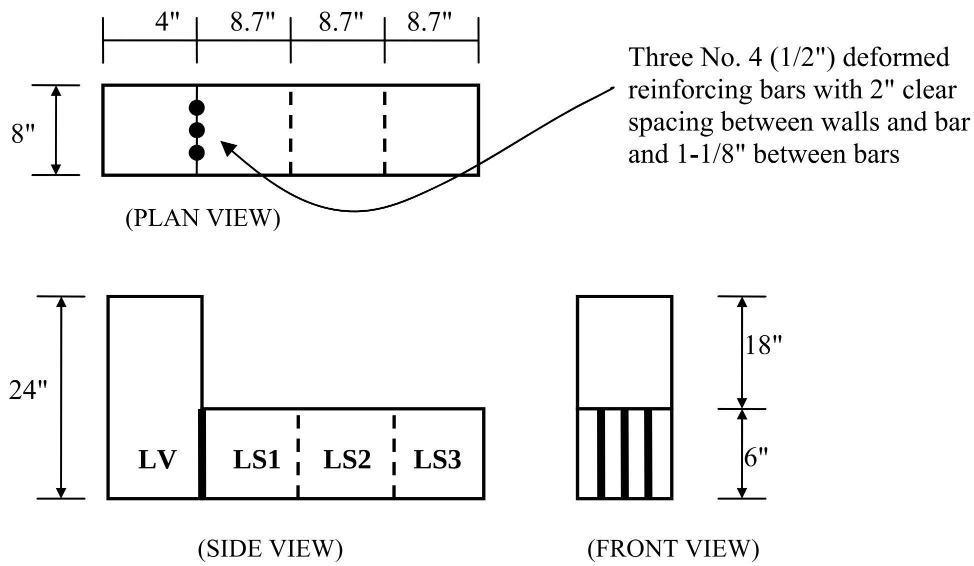

Figure 4. Schematic of L-box apparatus

RESULTS AND DISCUSSION

Flowability and Segregation Results of Base Mixtures

Figure 5 shows SWI values measured for the LV-LS1, LS2, and LS3 compartments of the L-box for mixtures B-BS1, A-AS1, and A-AS3. For an ideal mix, which is defined as a mix without any segregation and blockage tendency, SWI values should be unity and equal for every section of the box. Mixture B-BS1 had a high tendency of blockage and poor filling characteristic measured with the U-box and L-box (h_2/h_1), as shown in Table 3. Mixture A-AS1 had good passing and filling characteristics. Mixtures B-BS1 and A-AS1 had high segregation resistance, which was measured by means of the segregation test. On the other hand, mixture A-AS3 exhibited good passing characteristics (h_2/h_1=1 from L-box), but poor segregation resistance measured with VSI=2.5. Also, very little coarse aggregate was found in the top two inches of the segregation column (Table 3). It is interesting to note that SWI values computed for B-BS1 and A-AS1 mixtures, which had high segregation resistances, were smaller than unity, while SWI values were much larger than unity for A-AS3 mixture, which had a high tendency for segregation. Khayat, Assaad, and Daczko (2004) and Ramage Kahn, and Kurtis (2004) previously established good correlation between L-box and U-box test results. The limited results of the current study are in agreement with their findings.

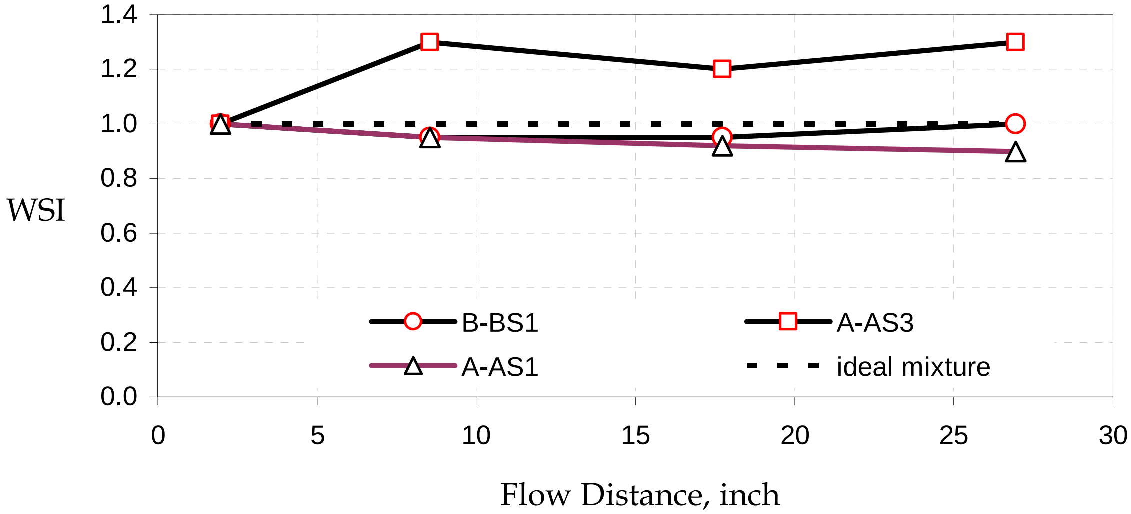

Figure 5. SWI measured with L-box test

Table 3. Fresh concrete properties of tested SCC

Mix No	Slump flow test			L-box	U-box	N*	WSI (column seg. test)				
	Flow (inch)	T_{50} (sec)	SVI	h_2/h_1	h_2/h_1		S1	S2	S3	S4	S5
A-AS1	26	2	1.0	0.70	0.90	>15	1.0	0.98	0.98	0.97	0.96
A-AS2	19.5	N/A	0			>10					
A-AS3	19.5	N/A	0			8					
A-AS4	19.5	N/A	0			3					
A-BS1	27	<1	1.5			4	1.0	0.98	1.03	1.07	1.05
B-BS1	24	7	1.5	0.55	0.50	6					
B-BS1-S	26	6	1.5	0.68	0.70	4					
A-AS2-WR1	19.5	N/A	0		0.77	2					
A-AS2-WRw2	24.5	2	1.0		0.95	2					
A-AS2-WRw3	27.5	1	1.5		0.98	2					
A-AS3-w1	19.5	N/A	0.5			1					
A-AS3-w2	22.0	4	1.0			1					
A-AS3-WRw3	23.5	4	1.0			1					
A-AS4-WRw	26.0	2	1.5			1					
A-AS3	32.5	<1	2.5	1.0	0.98	1	1.00	1.00	0.97	0.65	0.02

* N refers to number of slump flow test, for which average slump flow values were measured

Effect of Cement on Flowability

The effect of change in cement from shipment to shipment on flowability was determined based on the slump flow values measured for A-AS1, A-AS2, A-AS3, A-AS4, and A-BS1. These mixtures had the same aggregates, the same cement type and supplier (except A-BS1), and the same admixtures and dosages (see Table 2). All the cements from the supplier for Plant A, had similar chemical composition, as shown in the mill reports (see Table 1). However, the flow results for the four mixes using the supplier for Plant A showed large differences between the first shipment and the last three shipments. Although an average flowability of 26 inches was measured for A-AS1, the average flowability measured for A-AS2, A-AS3, and A-AS4 was only 19.5 inches. The average flowability measured for A-BS1, for which cement was obtained from the Plant B supplier, was 27 inches, similar to that achieved using the first shipment of cement from the Plant A supplier. Although it is not listed in this paper, two other conventional mixtures, with the same mixture proportion that was used for A-AS1, were prepared with cements AS2 and BS1. For those mixtures, w/cm was 0.51 and no admixtures (i.e., HRWR, SRA, and AEA) were used. The slump values (ASTM C143-00) measured for both mixtures were 6 inches, indicating that the cement source had no effect on conventional concretes made with the same aggregates. Repeatability between SCC batches using the same cement from the same shipment was excellent, varying by less than 1.0 inches. Because there was no significant physical and chemical difference between cement shipments AS1, AS2, AS3, and BS1, and because the same slump values were measured for the mixtures that did not employ admixtures, it is likely that the performance of the admixtures used was significantly controlled by some physical and/or chemical parameters of the cement, which are not clear yet.

Effect of Temperature on Flowability

The effect of temperature on SCC flowability was investigated by batching the A-AS2-WRw2 mixture at three different temperatures. During testing, the average room temperature was about 77±1° F. First, the mixture was prepared under laboratory conditions. In other words, the aggregates were at room temperature, and tap water from the city water supply was used as the mixing water. A slump flow of 24.5 inches, T20 of 2 seconds, and SVI of 1.0 were measured for the reference mixture. The concrete temperature, which was measured just before the slump flow test, was 76° F. For the cold temperature case, the mixture was re-prepared by using cold water as the mixing water. The same water supply was used for mixing water, and the mixing water and aggregates were refrigerated to cool them down. The moisture contents of the aggregates were determined by using aggregate samples that were placed in the aggregate containers in the temperature-controlled environment. Moreover, the mixer was filled with cold water just before batching to get data for low concrete temperatures. The concrete temperature was measured to be 45° F just before the slump flow test. A slump flow of 21 inches, T20 of 5 seconds, and SVI of 1.0 were measured for the mixture at 45° F. A third batch was prepared using hot water as the mixing water. The same water supply was used for hot water, which was heated in an oven. The aggregates were at room temperature (about 77° F). A slump flow of 27 inches, T20 of 1 second, and SVI of 1.0 were measured for the mixture. The measured concrete temperature was 91° F. The test results show that temperature may significantly affect flowability of SCC. Therefore, when there are large temperature fluctuations, fresh properties of SCC mixtures should be reevaluated.

Relationship between HRWR Dosage and Flowability

The relationship determined between slump flow values and HRWR dosage for A-AS3-w1, A-AS3-w2, A-AS3-WRw3, and A-AS4-WRw mixtures is shown in Figure 6. The results show that for each w/cm ratio there is a HRWR saturation dosage, after which increases in HRWR do not improve flowability significantly. The results also indicate that the saturation dosage is not sensitive to small changes in the w/cm ratio for the mixtures studied. The saturation dosages of HRWR determined for A-AS3-w2 and A-AS4-WRw, which had the same w/cm ratio and mixture proportion but cement from different shipments, were about 13.7 and 9.8 fl oz/cwt, respectively. Therefore, test results show that the saturation dosage for HRWR can be affected by properties of cement.

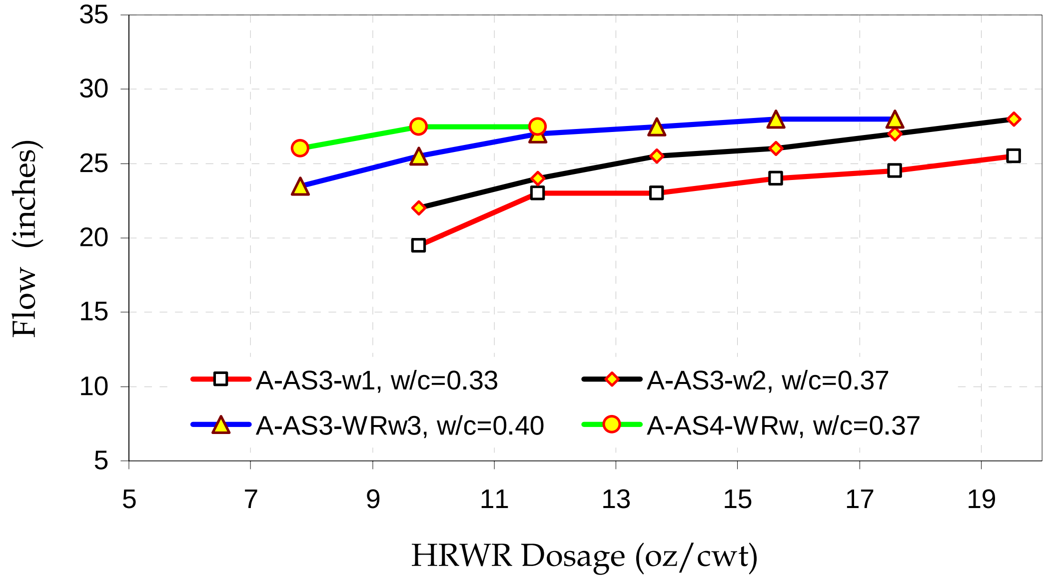

Figure 6. Relationship between HRWR dosage and slump flow

CONCLUSIONS

Based on the results of this study, the following conclusions can be made:

1. SCC with adequate fresh properties has been developed successfully with locally available materials for two precast concrete plants in the state of Minnesota.
2. U-box filling height has negligible effects on the test results (h_2/h_1).
3. The modified L-box testing procedure may be useful as a means of evaluating segregation resistance of SCC.
4. Cement shipments from the same supplier can significant affect flowability of SCC.
5. Fresh properties of SCC mixes are repeatable for the same cement shipment.
6. Flowability of SCC increases as concrete temperature increases.
7. Flowability of SCC does not improve significantly once HRWR saturation dosage is reached. HRWR saturation dosage is a function of at least cement and w/cm ratio.

ACKNOWLEDGMENTS

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Practical Approach to Estimating Realistic Depths of Abutment Scour

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ABSTRACT

Observations of bridge abutment scour lead to a practical new approach for estimating realistic scour IIHR-Hydroscience and Engineering depths at bridge abutments. Prior design approaches usually led to unrealistically deep estimates of abutment scour depth. The observations presented stem from flume experiments conducted with abutments with approach embankments subject to a range of erodibility conditions: fixed embankment on fixed floodplain; riprap-protected erodible embankment on readily erodible floodplain, and unprotected readily erodible embankment on readily erodible floodplain. The approach discards the old notion of linearly combining bridge-waterway constriction scour and local scour at the abutment structure, a notion that laboratory experiments do not support. Instead, the approach entails estimating an abutment-induced local amplification of constriction scour at the bridge opening, and separately estimating a maximum local scour depth at the abutment when exposed by embankment failure.

Key words: abutment—bridge waterways—scour

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INTRODUCTION

Scant situations of hydraulic engineering are more complex than those associated with scour in the vicinity of a bridge abutment, especially one located in a compound channel. Accordingly, few situations of scour depth estimation are as difficult. Therefore, it is not surprising that considerable uncertainty and debate has been associated with scour depth estimation for abutments, and that the existing estimation relationships are not well accepted; a concern is that existing relationships tend to predict scour depths that seem excessive. The present paper introduces a fresh approach to scour depth estimation for abutments. The approach is still in development; its estimation relationships are being formed using the findings from an extensive laboratory study currently underway.

As shown in Figure 1, abutment scour involves hydraulic erosion with consequent slope stability failure of the earth-fill embankment at the abutment. Many bridge abutments are located in compound channels whose geometry is rather complex. Additionally, many abutments are located where the channel is formed of several bed materials, occupying different locales within a bridge site; sands may form the bed of a main channel, silts and clay may predominate in riverbanks and underlying floodplains, and rocks may have been placed as riprap protection for the abutment, as well as sometimes along adjoining riverbanks. Early work on abutment scour focused on the simpler and perhaps idealized situations of scour. Commensurately, the existing relationships and guidelines apply to simplified abutment situations, such as an abutment placed in a straight rectangular channel, and can only be extended with considerable uncertainty to actual field conditions. Extrapolation often causes existing scour relationships to predict substantially greater extents of scour than actually may occur at many actual bridge sites.

Figure 1. Scour-induced failure of the earth-fill embankment at a spill-through abutment

A common feature of abutment scour suggests a reasonably straightforward approach to obtaining design estimates of scour depth at abutments. The feature is abutment and embankment contraction of flow through a bridge waterway. The flow locally around the abutment is part of the overall field of constricted flow through a bridge waterway, to the extent that it can be difficult to distinguish between what are conventionally termed local scour and contraction scour. The fresh approach explained in this paper treats abutment scour as a local amplification of contraction scour. Only when flow erodes and passes through an approach embankment, then fully exposing an abutment as if it were a pier, does local scour occur at an abutment. The writers are currently developing the approach further.

ABUTMENT CONSTRUCTION

Though many studies have focused on several of the component scour processes at play and have delineated sets of important parametric trends, few studies have considered the usual construction features of abutments and their approach embankments in compound channels, including the following items:

1. Most abutments comprise an abutment structure, such as the standard stub abutment used for spill-through abutments (Figure 2), which is a pile-founded structure. The other common type of abutment is a wing-wall abutment typically used for smaller bridges.
2. The earth-fill embankment approaching the abutment structure is erodible and subject to geotechnical instabilities.
3. The portion of the embankment near the abutment is usually riprap protected.
4. The floodplain (often extensively comprising cohesive soils) may be much less readily eroded than the main channel bed.

The fact that most abutments are usually piled structures with an earth-fill embankment influences scour depths at abutments. Most scour case studies (e.g., Ettema et al. 2002) show that the embankment fails before the abutment's foundation fails.

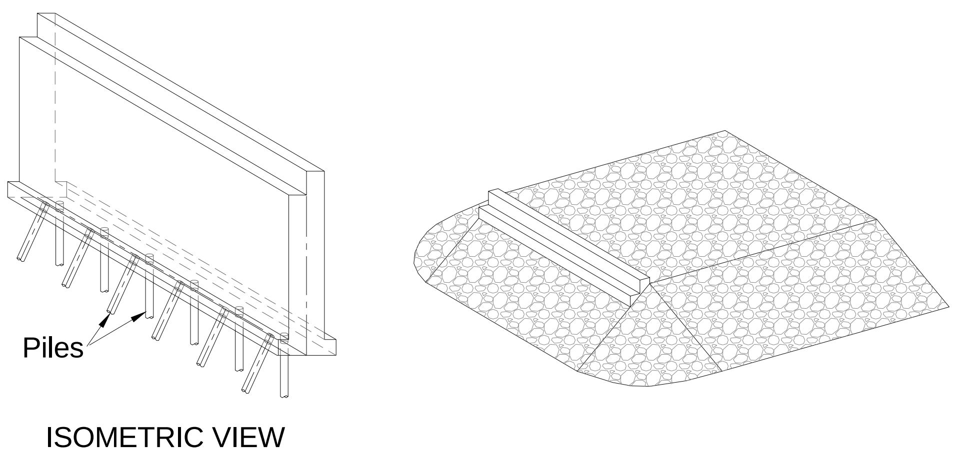

Figure 2. Most abutments are pile-supported (e.g., standard stub abutment used by the Iowa DOT) set in an erodible earth-fill, approach embankment

The writers conducted experiments with abutments in a compound channel subject to several conditions of embankment and floodplain erodibility: fixed embankment and floodplain (such as a floodplain formed of largely cohesive soil), erodible floodplain and riprap-protected embankment; and erodible floodplain with erodible embankment. The main channel had a bed of uniform sand. Figure 3 shows the scour that developed for one configuration of a fixed abutment on a fixed floodplain. The scour, by lowering the bed near the abutment, could potentially make the channel bank and embankment face unstable.

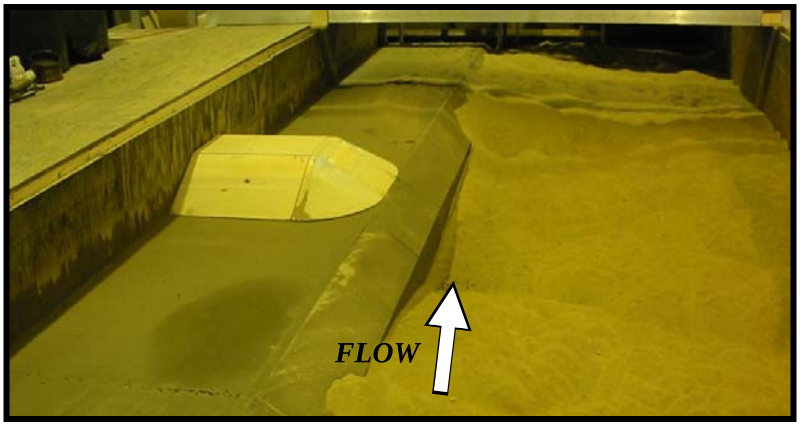

Figure 3. Scour in main channel with fixed floodplain and embankment

Most embankments are erodible, and it is common for the approach embankment near the abutment to fail and breach before the abutment itself fails, if indeed the abutment does fail. This observation is borne out by the writers' laboratory experiments, which were conducted with a floodplain simulated with sand, as shown in Figure 4. Observations from case studies in the field and from the writers' laboratory experiments show that as abutment scour develops the channel bank erodes, eventually causing the embankment side slope to undergo a slope stability failure. Failure and erosion of the embankment isolates the abutment, practically exposing it as if it were a pier. Also, embankment failure may somewhat relax contraction scour.

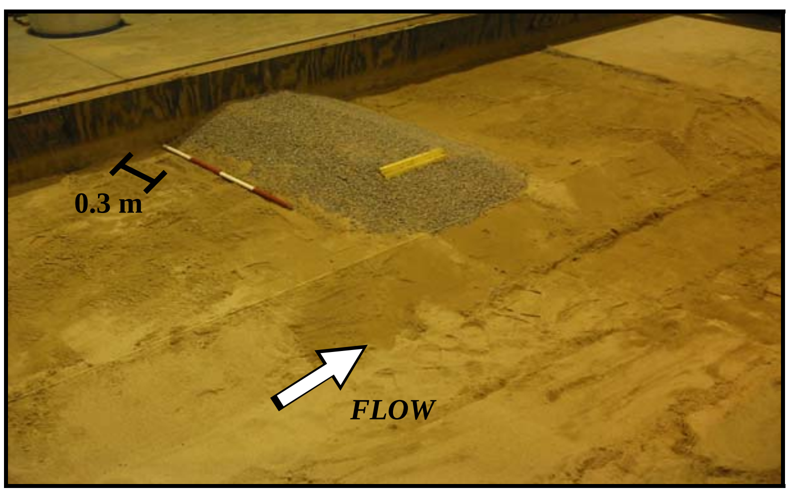

Figure 4. Layout of experiment with riprap-protected abutment on erobible floodplain

Moreover, the experiments show that maximum scour depth may not occur at the abutment. As the width of the floodplain increases and flow contraction concomitantly increases, the location of deepest scour can shift downstream of the abutment. Figure 5 depicts one scour condition resulting from the writers' experiments with an erodible wing-wall abutment; though the embankment failed partially, the deepest scour occurred a short distance downstream of the abutment. Evidently, the location of deepest scour varies with the flow field developed around the abutment. Figure 5 depicts the deepest scour condition occurring at the abutment structure itself. This condition occurred when the embankment was eroded such that the abutment structure became exposed, and scour developed as if the abutment were a pier.

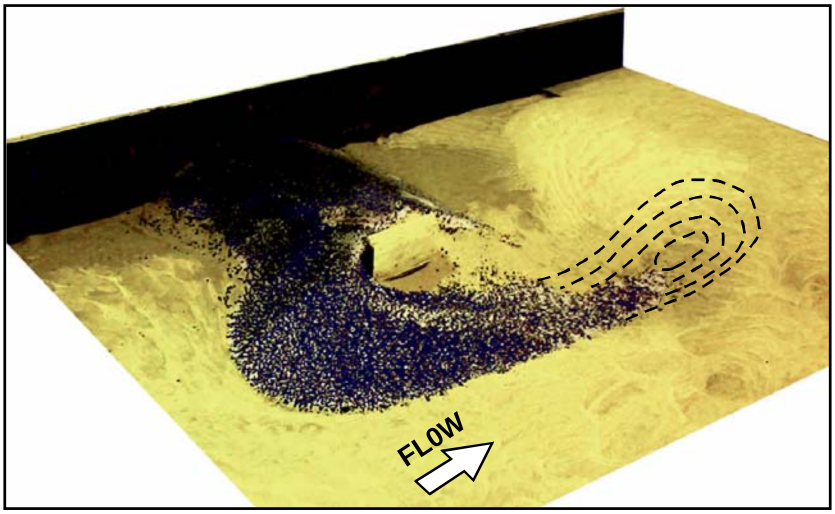

Figure 5: Scour near abutment with riprap-protected embankment

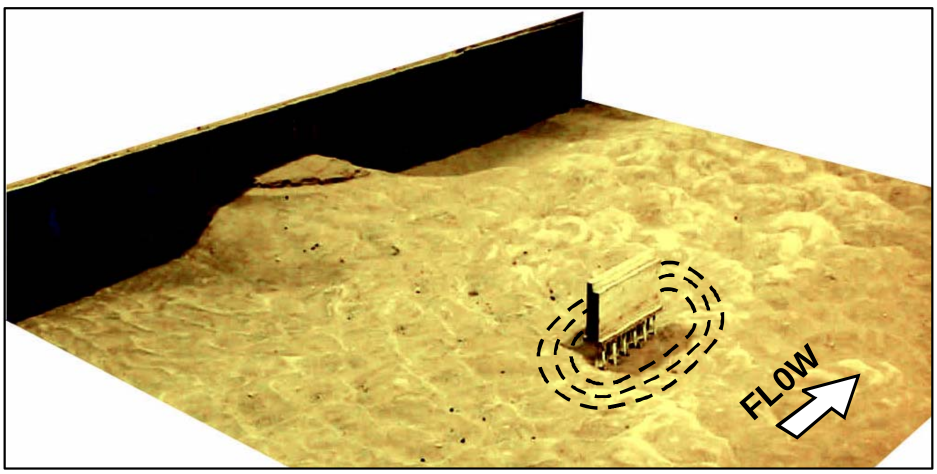

Figure 6. Scour at stub abutment exposed when embankment is breached

For some configurations of intact embankment, depending on the approach flow orientation and flow field generated by the embankment and abutment, the maximum scour depth may occur right at the abutment. Based on observations from the writers' experiments and a review of published data, it would seem that the maximum scour depth occurs right at the abutment in cases where the abutment and its embankment are taken to be a fixed, solid body that extends deeply into the bed of a channel. This form of abutment and embankment have been tested extensively in prior flume studies.

SCOUR DEPTH AT ABUTMENTS NEAR MAIN CHANNEL

The existing relationships for scour depth estimation treat abutments and approach embankments as fixed, solid structures extending deep into the bed. However, few abutments are built in this way. Illustrations such as Figures 1 through 4, as well as the writers' observations of scour development at piled-supported abutments and earth-fill embankments, suggest the need for a fresh approach to scour depth estimation.

The practical approach focuses on estimates of maximum flow depth associated with two primary scour forms:

1. *Maximum scour as near-abutment amplification of contraction scour*. The writers suggest that, especially for spill-through abutments, the deepest scour develops essentially as a near-abutment amplification of contraction scour, with the amplification caused by the increased flow velocity and turbulence local to the abutment and its approach embankment. This depth occurs when an abutment's embankment is either fully or largely intact, such that the flow is constricted through the bridge opening. A largely intact embankment here means that the flow has not broken through the approach embankment.

   Actually, for an abutment on a compound channel, deepest scour should be checked at two locations: in the main channel if the abutment is close to the main channel, and on the floodplain if the abutment is well set back from the main channel.

2. *Maximum scour as local scour at fully exposed abutment structure*. This scour form occurs when the embankment has eroded so that the abutment structure (e.g., standard stub or wing-wall) is fully exposed as if it were a pier.

Because contraction scour integrates the influences of several variables (e.g., approach flow depths and discharge, bed sediment), it is meaningful and convenient to relate maximum scour depth, Y_{max}, to contraction scour depth, Y_C. That is, for a fixed embankment and floodplain,

$$Y_{\max} = \alpha Y_C \tag{1}$$

The factor α amplifies Y_C near the abutment (as evident in Figure 3). The magnitude of α depends on flow velocity distribution at the bridge site, and it must account for turbulence. Site morphology, along with the presence of vegetation and sundry physical peculiarities, complicate estimation of flow distribution and scour depth for sites. In particular, it is difficult to identify precisely where flow velocity will be largest, turbulence greatest, and scour depth likely deepest. The relationship α has yet to be determined. The writers suggest that Equation (1) be expressed as

$$\left(\frac{Y_{\max}}{Y_1}\right) = C_T \left(\frac{q_{\max}}{q_1}\right)^{6/7} \tag{2}$$

where q_{max} is the unit discharge coinciding with the location of deepest scour in the main channel. If all the floodplain flow entered the main channel, in the situation of a long abutment extending practically across the floodplain, $q_{\max} = m\bar{q}_2$, where $\bar{q}_2 = (Q_{1m} + Q_F)/B_2$, Q_{1m} is the approach flow in the main channel, and Q_F is the approach flow over the floodplain. The values of m and C_T have to be determined from laboratory or numerical-simulation data.

An approximate relationship for flow depth at the location of maximum scour is

$$Y_{MAX} = Y_1 C_T m^{6/7} \left(\frac{\bar{q}_2}{\bar{q}_1}\right)^{6/7} \tag{3}$$

where $\overline{q}_1 = Q_{1m} / B_1$. For a long contraction, $m \approx 1$, $C_T \approx 1$, and thus Equation (3) simplifies to

$$Y_{MAX} = Y_1 \left(\frac{\overline{q}_2}{\overline{q}_1} \right)^{6/7} = Y_C \tag{4}$$

which essentially is the relationship proposed by Laursen (1960) for estimating the scour depth associated with live bed flow through a long contraction. Comparison of Equations (1), (3), and (4) indicates that

$$Y_{MAX} = \alpha Y_C = C_T m^{6/7} Y_C \tag{5}$$

The main difficulty to be overcome for design estimation of scour depth, therefore, is estimation of m and C_T.

SCOUR DEPTH AT ABUTMENTS SET BACK FROM MAIN CHANNEL

This condition of abutment failure is of primary concern for abutments on wide floodplains and those set well back from the main channel. Because clearwater flow predominantly occurs on floodplains, it is assumed herein that scour of a floodplain at an abutment occurs as clearwater scour. Moreover, it is assumed that the scour development is not affected by flow or scour of the main channel bed. Analysis leads to the following equation:

$$\left(\frac{Y_{\max}}{Y_F} \right) = C_T m^{6/7} \left(\frac{\tau'_F}{\tau_C} \right)^{3/7} \left(\frac{\overline{q}_2}{\overline{q}_F} \right)^{6/7} \tag{6}$$

Where τ'_F and τ_C are boundary shear stress on the floodplain and critical for boundary sediment entrainment, respectively. Ettema et al. (2005) provide details as to the derivation and use of Equation (6), as well as Equations (1) through (5).

ESTIMATION OF SCOUR AT EXPOSED ABUTMENT STRUCTURE

Scour depth prediction for this scour condition has to rely on an empirical relationship, owing to the complexity of flow and sediment entrainment from an abutment structure exposed once the approach embankment has been breached. The approach being developed by the writers is basically that used for estimating scour depth at a bridge pier; the exposed abutment is a pier-like structure.

LAB EXPERIMENTS RESULTS

The scour data are being obtained for three states of floodplain and embankment erodibility:

1. Fixed floodplain and embankment
2. Erodible floodplain and riprap-protected embankment
3. Erodible floodplain and an unprotected embankment

Figure 7 presents data on maximum flow depths, Y_{MAX}, plotted versus unit-discharge ratio, $\overline{q}_2 / \overline{q}_1$, for the fixed and erodible floodplain states. The data are for the simulated spill-through abutments. Also plotted in this figure is contraction flow depth, Y_C, versus $\overline{q}_2 / \overline{q}_1$, with Y_C estimated using the long contraction relation given as Equation (4). Here, L is abutment length, B_F is floodplain width, and 0.5B is half the width of the main channel.

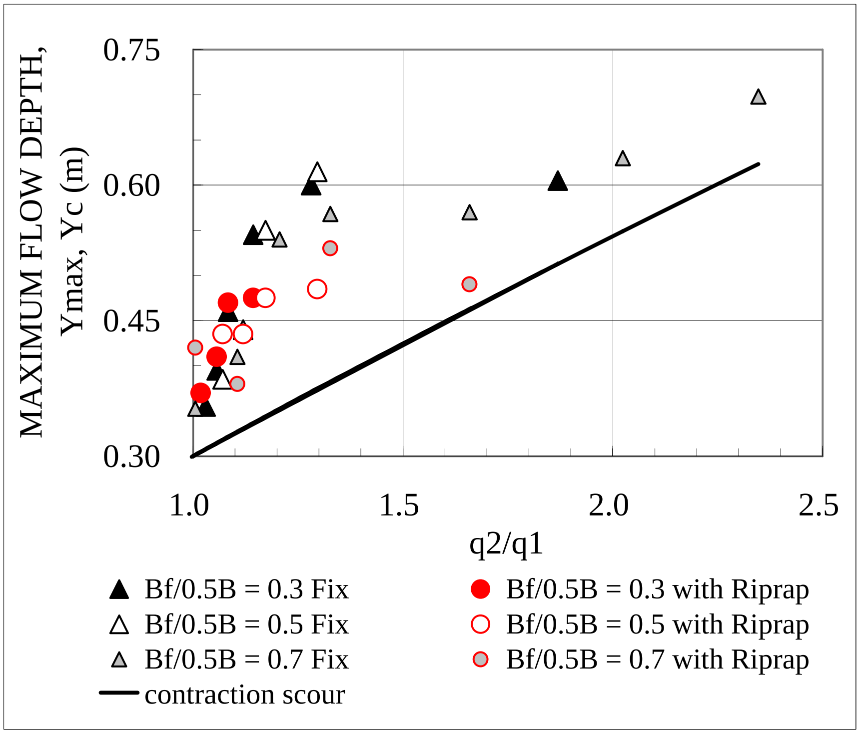

Figure 7. Variation of Y_{MAX} and Y_C with unit-discharge ratio $\overline{q}_2 / \overline{q}_1$ and lab data on spill-through abutments

To assess the depth amplification factor, $\alpha = C_T m^{6/7}$, expressed in Equation (5), Figure 8 plots the ratio Y_{MAX}/Y_C versus $\overline{q}_2 / \overline{q}_1$ for fixed floodplain and embankment. This figure provides some important insights:

1. The data appear to conform to a reasonably consistent trend.
2. At the lesser values of $\overline{q}_2 / \overline{q}_1$ (and flow contraction), Y_{MAX} substantially exceeds Y_C. Eventually as the bridge waterway becomes more contracted, $\overline{q}_2 / \overline{q}_1$ increases, and values of Y_{MAX} approach Y_C. This portion of the trend reflects the dominance of scour caused primarily by flow contraction as opposed to that attributable the local change in bed form height in the contraction combined with the turbulence generated by flow passing around the abutment and over the edge of the main channel bank.
3. It is intriguing that the values of Y_{MAX}/Y_C attain a maximum value of around 1.5~1.6 when $\overline{q}_2 / \overline{q}_1$ ≈ 1.2~1.3.

4. It also is intriguing that the values of Y_{MAX}/Y_C decline quite markedly after the maximum. The values then asymptote to a level of about 1.1.
5. The parameter floodplain width divided by channel half width, $B_f/0.5B$, exerts a small influence, especially in the maximum values of Y_{MAX}/Y_C. The maximum value of Y_{MAX}/Y_C is larger for the smaller value of $B_f/0.5B$. This influence is attributable to the fact that, in absolute lengths, the abutment is closer to the main channel, thereby causing more of the turbulence generated by the abutment to be diffused to the main channel.

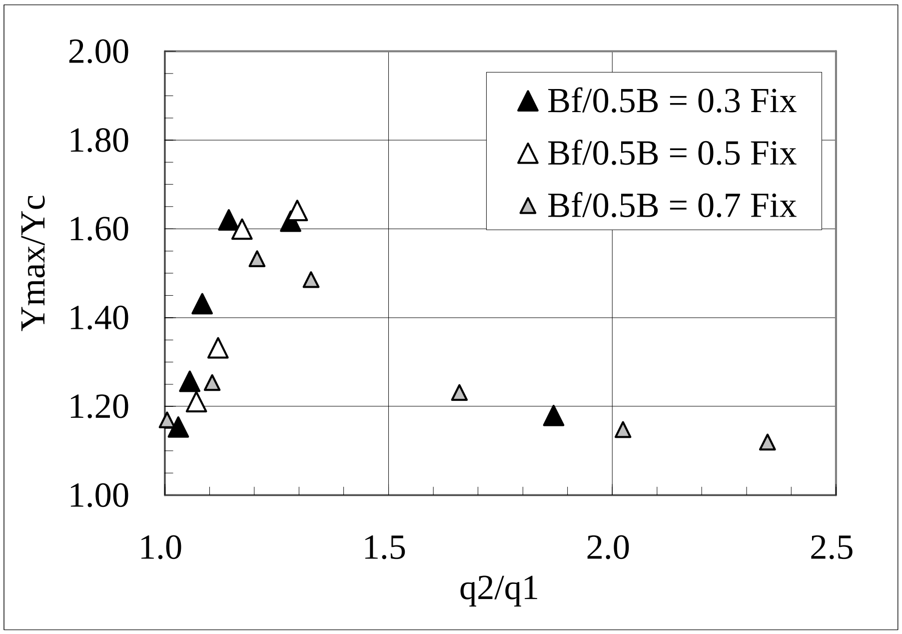

Figure 8. Variation flow depth increase, Y_{MAX}/Y_C, with $\overline{q}_2 / \overline{q}_1$; spill-through abutments on fixed (erosion-resistant) floodplain

Figure 9 plots the ratio Y_{MAX}/Y_C versus $\overline{q}_2 / \overline{q}_1$ for the erodible floodplain and riprap-protected embankment. This figure contains a combination of scour conditions 1 and 2:

1. For the lesser values of $\overline{q}_2 / \overline{q}_1$ (and flow contraction), Y_{MAX} substantially exceeds Y_C. Moreover, for some experiments, the value of Y_{MAX}/Y_C exceeds that obtained when the floodplain was fixed. For these latter experiments, scour condition 2 prevailed and produced a deeper scour than did scour condition 1.
2. As values of $\overline{q}_2 / \overline{q}_1$ increased, scour conditions 1 and 2 jointly increased the flow cross-sectional area at the abutment, and thereby relaxed the flow contraction, resulting in a leveling off of flow depths at the scour location.

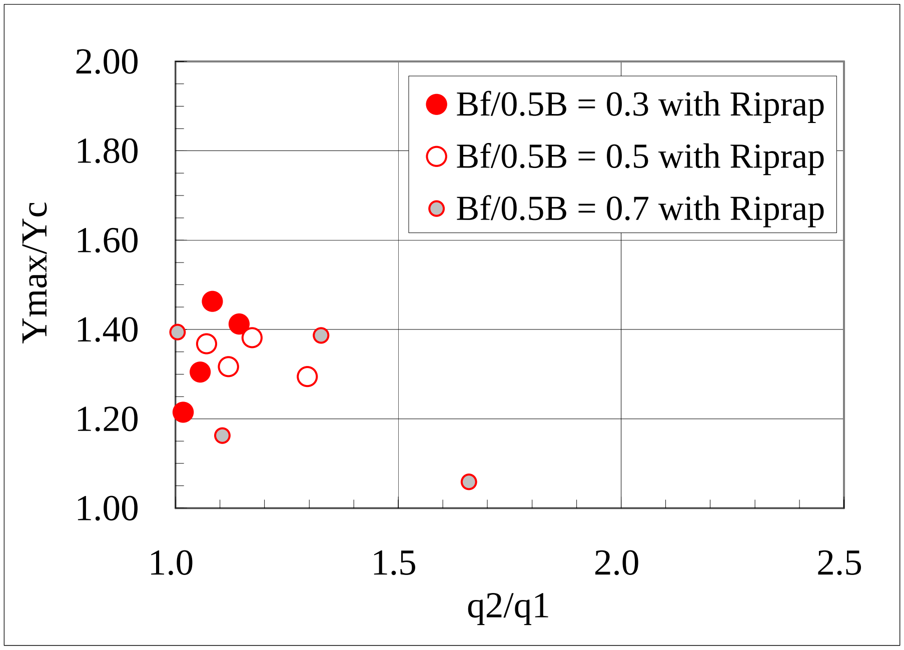

Figure 9. Variation flow-depth increase, Y_{MAX}/Y_C, with $\overline{q}_2 / \overline{q}_1$; spill-through abutments (armored with riprap) on erodible floodplain

Figure 10 includes data obtained with the wing-wall abutments for this scour condition; i.e., for the cases $B_f/0.5B$ = 0.3, 0.5, and 0.7, note that L/B_f = 1. When the floodplain and the embankment are erodible, the three scour conditions occurred. When the flow breached the embankment, the abutment itself becomes exposed, so that scour depth estimation must treat the abutment as if it were a pier-like structure. The writers are completing a design relationship for this condition.

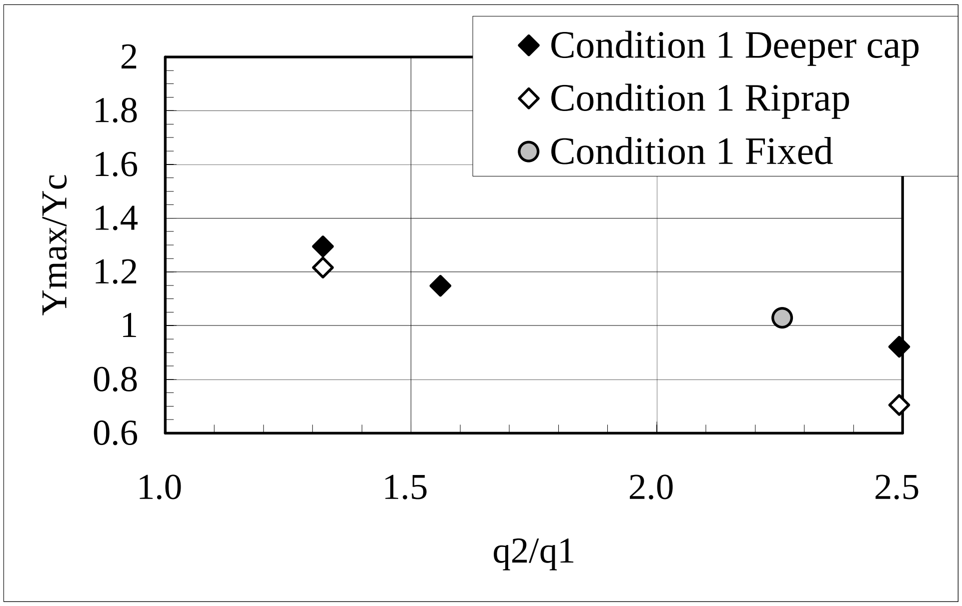

Figure 10. Variation flow-depth increase, Y_{MAX}/Y_C, with $\overline{q}_2 / \overline{q}_1$; wing-wall abutments on fixed or erodible floodplain

It is interesting to see that $Y_{MAX}/Y_C \approx 1$ when the floodplain and embankment were fixed. This finding suggests that the scour was largely due to flow contraction and was not much affected by turbulence generated by flow around the abutment. The flow field observations and measurements taken in the lab support this finding.

DESIGN APPROACH

The observations and data indicate that a practical and adequately reasonable approach to estimating scour depth at an abutment can be obtained using Equations (4), (5), and (6). To use these equations entails determining the unit discharge ratio, $\overline{q}_2 / \overline{q}_1$, along with *m* and C_T. A two-dimensional numerical flow model can be used to estimate $\overline{q}_2 / \overline{q}_1$ along with *m*, though C_T will have to remain empirically derived (with field verification) from laboratory data. The present data, though, suggest that approximate estimation can be made for scour of the main channel using the following equation:

$$Y_{MAX} = C_T m^{6/7} Y_C \approx \alpha Y_C = 1.75 Y_C \qquad (7)$$

This relationship is applicable to spill-through and wing-wall abutments. The suggestion of using α = 1.75 requires further verification, but results to date indicate it to be quite appropriate for design estimation. If no contraction scour is estimated to occur, Equation (7) gives Y_{max} as twice the design flow depth through the bridge waterway.

Further work is underway to complete the estimation of a comparable relationship for scour at abutments set back on floodplains.

CONCLUSIONS

The new and practical approach for scour depth estimation pursued by the writers holds good promise of being practicable and providing scour depth estimates closer to those observed in the field. This paper outlines the approach. The writers are currently conducting further experiments towards determining the relationships expressed in Equations (2) through (4). The outcome of the experiments may place the estimation approach on a suitably practical and reasonably accurate footing.

ACKNOWLEDGMENTS

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Use of In Situ Testing to Optimize Slope Design in Highly Weathered Shale

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ABSTRACT

Slope stability of embankments bearing over highly weathered shale encountered at shallow depths is a major concern in roadway design. This condition was encountered at a bridge location in southeastern Iowa, where the proposed 120.5-meter x 12.0-meter creek crossing dual bridges are part of a four-lane state highway. The maximum height of the embankment at the north abutment location is approximately 12 meters and 8 meters at the south abutment location, with 3H:1V slopes in the longitudinal and transverse directions. Conventional soil borings with standard penetration testing indicated the presence of weathered shale at very shallow depths, in some cases as shallow as 0.9 meters. The subsurface profiles indicated the presence of sloping shale at both the north and south abutments overlain by sandy lean clay. The average slope ratio of the top of the shale is approximately 10H:1V at the north abutment location and 3H:1V at the south abutment location. As per Iowa DOT guidelines, a 0.3-meter-thick layer of highly weathered shale was assumed at the clay/weathered shale interface and assigned undrained shear strength of 10kPa. Slope stability analyses indicated global slope instability with failure surfaces propagating through the sloping and highly weathered shale layer. As a result, various ground improvement, retaining wall, and increased bridge length alternatives were evaluated. These alternatives were estimated to cost between $3,000,000 and $5,000,000. In view of these high costs, an additional comprehensive subsurface exploration and testing program was developed and executed at the site to verify that the shear strength parameters, especially with respect to the highly weathered shale, were reasonable. The supplemental program consisted of conducting 35 in situ Iowa borehole shear tests at various depths in the 10 boreholes, particularly at the shale-clay interface. Slope stability analyses were performed using SLIDE software, including probabilistic analyses to evaluate the probability of potential global slope instability. The analyses indicated the slopes to be stable under both short-term (end of construction) and long-term (drained) conditions. The judicious use of in situ testing in addition to conventional laboratory testing resulted in the elimination of potential high-cost alternatives and kept the project within budget limits.

Key words: highly weathered shale—in situ testing—shear strength of shale—slope stability

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Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

Dual 120.5-meter x 12.0-meter pretensioned prestressed concrete beam bridges spanning a creek have been proposed as part of a four-lane facility in southeastern Iowa. Approach embankment fills on both the north and south sides of the creek with pile-supported abutments were planned to support the bridges. However, slope stability analyses performed for the slopes in front of the abutments indicated potential global instability with failure surfaces propagating through the sloping and highly weathered shale, which was assigned a shear strength parameter, c, of 10kPa, in accordance with Iowa DOT guidelines. As a result, various alternatives were developed. These alternatives were evaluated and estimated to cost between $3,000,000 to over $5,000,000. In view if these high costs, a comprehensive supplemental subsurface exploration and testing program was developed and executed at the site to verify that the shear strength parameters utilized in the slope stability analyses, especially with respect to the highly weathered shale, were reasonable. The additional cost of this program was approximately $60,000. The program was also executed to develop more realistic, site-specific data to optimize the design and justify and/or possibly reduce proposed mitigation measures and its estimated cost.

This paper summarizes the subsurface exploration, in situ testing and laboratory testing performed at the site. The paper also discusses the interpretation of the field and laboratory results, recommended design parameters, slope stability analyses performed, the authors' final design recommendations, and conclusions from the study.

GEOLOGY

The soils in the county where the project site is located are formed in glacial till, loess, and alluvium. The major glacial till deposits of the pre-Wisconsin age are the Nebraskan and Kansan glacial tills. Wisconsin-age loess covers most of the county and is an extensive parent material. Alluvium in this county is derived from loess and glacial till. In some areas, the streams are still cutting through shale and the flood plains are narrower and have a steeper gradient. The alluvium encountered in these areas is commonly coarser. Shale is the oldest parent material in the county. It comprises a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period (USDA 1977).

At the bridge location, the drainage basin has incised through the glacial overburden down into the bedrock. Ridge tops are capped with loess-derived soils, and farther down the slopes, the paleosol- and till-derived soils are present. The soils formed on the paleosol consist of silty clays, clays, and clay loams that are characteristically poorly drained, have a low permeability, a high water capacity, a high seasonal water table, and high shrink-swell potential. Below the glacial materials, bedrock is exposed and/or mantled by a thin veneer of soil. The bedrock primarily consists of clay shale, with sandstone and thin limestone layers. The slopes on these soils are moderately to very steep and runoff is rapid. The bedrock surface closely parallels the existing ground surface. As a result, the thickness of the overburden does not increase appreciably as the ground rises in elevation, especially at the south side of the creek.

INITIAL SUBSURFACE EXPLORATION AND TESTING AND DESIGN ALTERNATIVES

The initial subsurface exploration program was completed in 2001. The program involved the drilling of 16 borings that were located based on the Iowa DOT guidelines for bridge borings. The borings were drilled using 2.25-inch inside diameter hollow stem augers. All borings were terminated in shale and cores were obtained from four borings. Representative disturbed samples of the materials encountered in the borings were obtained with a standard two-inch outside diameter split-spoon sampler. Standard penetration tests (SPTs) were performed at five-foot intervals for the depth of the borings. Relatively undisturbed Shelby tube samples were obtained from various representative soil and weathered shale layers. Unconfined shear strength of select soil samples was estimated by testing with a pocket penetrometer.

A typical subsurface profile is presented in Figure 1. The subsurface profile consisted of a layer of sandy lean clay overlying a layer consisting of a mix of sand and clayey sand with gravel. This sand-clay layer was underlain by a layer of highly weathered shale. In general, the top of the highly weathered shale paralleled the ground surface. At the south side of the creek, the highly weathered shale was encountered within one to two meters below the ground surface.

Initial Soil Parameters

The soil parameters to be used in the slope stability analyses were developed based on the results of the laboratory tests and generally accepted correlations with the SPT N-values. The highly weathered shale layer was assigned an undrained shear strength value of 10kPa based on Iowa DOT guidelines. The soil parameters used in the slope stability analyses are presented in Table 1.

Table 1. Initial shear strength parameters (undrained)

Layer	UW	Strength Type	c (kPa)	ϕ (deg)	Remarks
Fill*	20	Mohr-Coulomb	29	12	Value as per Iowa DOT guidelines
Sandy Clay	20	Undrained	27	-	Data from single test
h.w.sh.	20	Undrained	10	-	Value as per Iowa DOT guidelines
m.w.sh.	22	Undrained	50	-	Assumed
s.w.sh.	23	Undrained	200	-	Assumed

* Fill is assumed to be compacted and drained during installation.
h.w.sh.= highly weathered shale, m.w.sh.= moderately weathered shale, s.w.sh.= slightly weathered shale

Initial Slope Stability Analyses

Stability of the embankment slopes was evaluated using the computer program WinStabl. A total of four sections (two in transverse directions along the north abutment, one in the transverse direction at the south abutment, and one in the longitudinal direction along the south abutment) were analyzed for slope stability. Each section was analyzed for the undrained (end of construction) loading. A typical WinStabl output is presented in Figure 2 and a summary of the analysis results is presented in Table 2.

The analysis indicated potential global slope instability with failure surfaces propagating through the sloping and highly weathered shale.

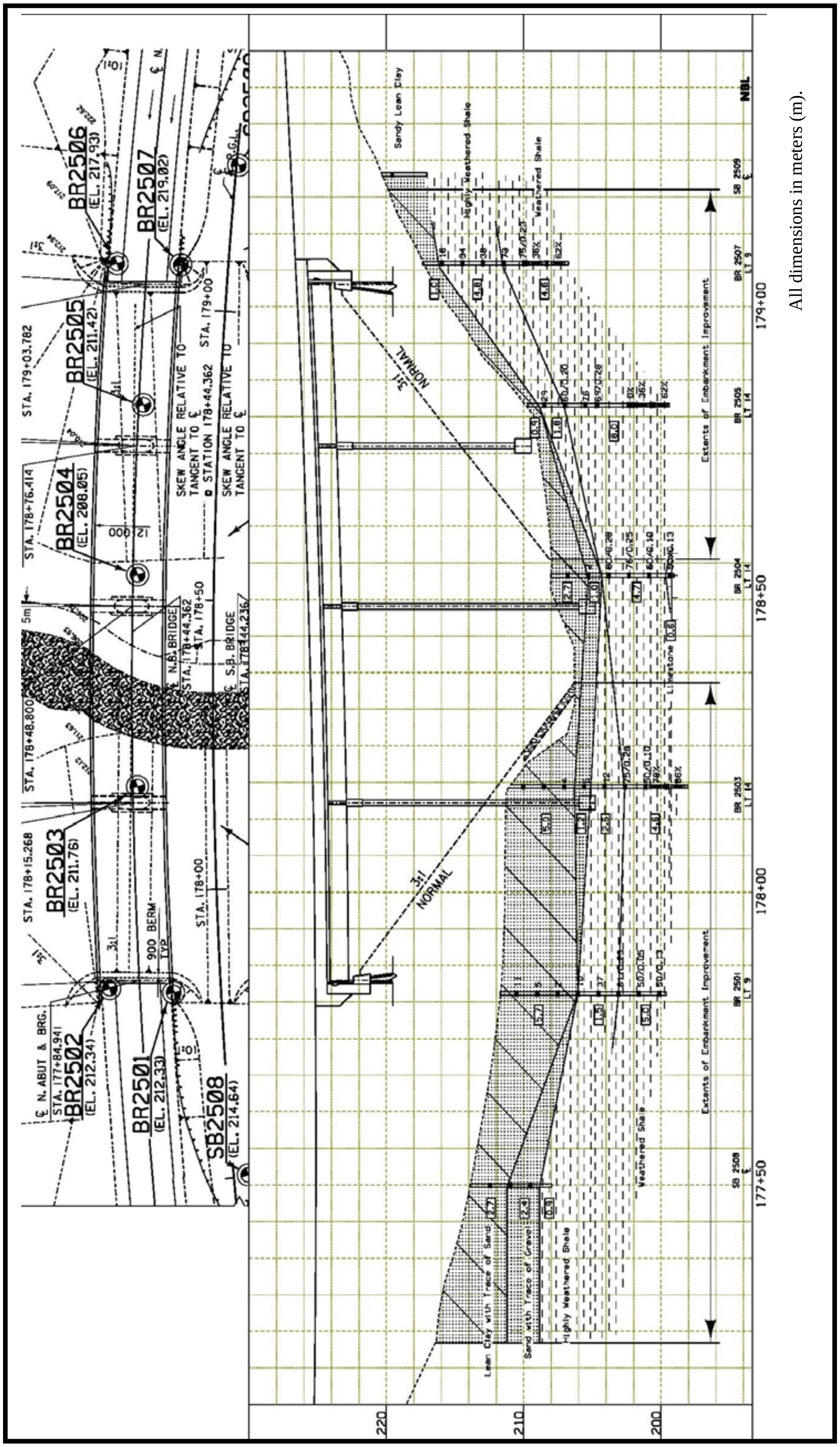

Figure 1. Subsurface profile, 2001 exploration (typical)

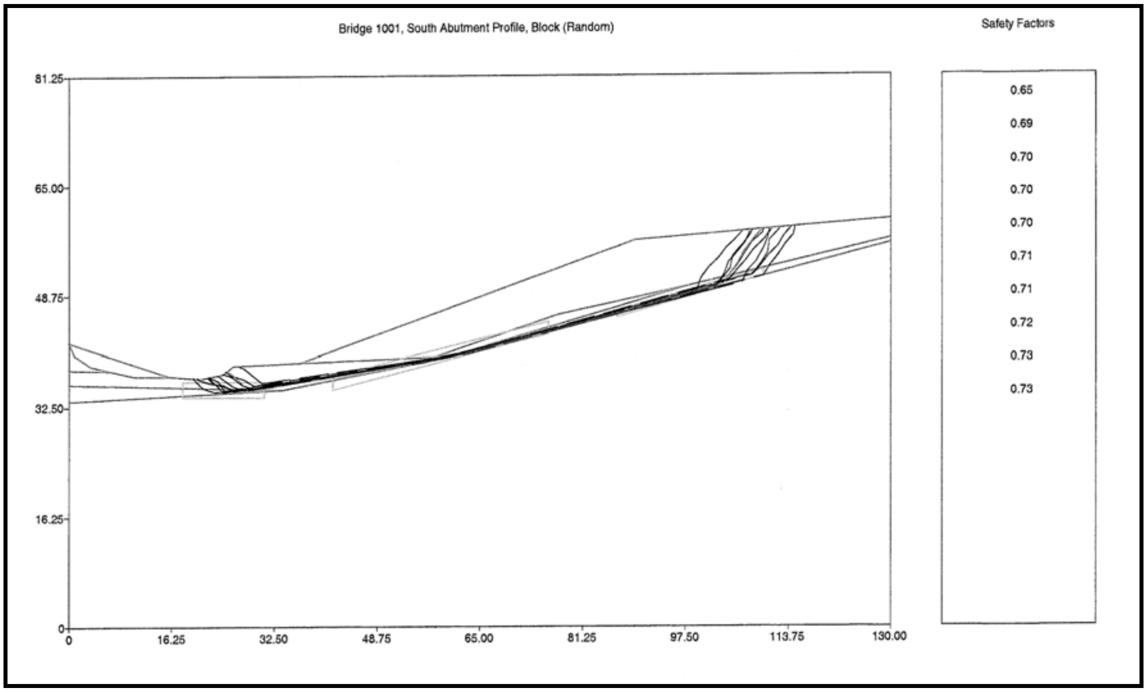

Figure 2. Typical WinStabl output (undrained analysis)

Table 2. Stability analyses results using initial shear strength parameters (undrained analysis)

Location of Slope Section Analyzed	Search Method	Factor of Safety	Remarks
North Abutment Transverse (Right) Sta.177+84	Circular	0.7	
	Block	0.7	
North Abutment Transverse(Left) Sta.177+84	Circular	0.7	
	Block	0.6	
South Abutment Profile (NBL)	Circular	0.9	
	Block	0.7	
South Abutment Transverse (Right) Sta.179+04	Circular	1.5	
	Block	1.0	Potential block failure

Conceptual Design Alternatives

As a result of the initial slope stability analyses, various ground improvement measures and retaining wall alternatives were evaluated. The evaluation included the feasibility and constructability, cost, and potential risks of the different alternatives. The total cost (TC) of each alternative included the entire support system, but excluded the cost of the bridge above the pile cap. This was done in order to provide a relevant comparison between some alternatives that provided foundations for the bridge abutments and support for the embankment, while others only provided embankment support to increase global stability.

Each alternative was assigned a risk factor (RF), ranging from 1 to 5, with 5 representing the highest risk. A suitability index (SI) was developed to compare the risk with the total cost using the following formula:

*SI = 1/[(TC in million dollars)*SQRT(RF)]*

The hypothesis behind the formula is that the most suitable alternative will be the least expensive one to construct with the lowest risk. A summary of the various mitigation measures, the total cost, risk factor, suitability index, and associated comments is presented in Table 3.

Table 3. Summary of mitigation measures

Alternative	Total Cost	Risk Factor	Suitability Index	Comments
North Abutment				
Excavate unsuitable soils and replace with suitable compacted backfill	$ 5,092,767	5	0.1	Deep excavations and dewatering pose significant constructability issues, especially on North side of bridge. It will likely require cofferdam or relocating the creek.
Geopiers® for ground improvement for global stability	$ 3,417,070	3	0.2	Risks associated with drilling and compacting granular/stone fill for geopiers under ground water table.
Tie-back wall with Geofoam fill	$ 2,315,819	1	0.4	Alternative assumes a tie-back wall in front of the abutment and extending to 30 meters on either side. The tieback wall will resist the lateral loads due to earth backfill embankment behind it.
Tangent drilled shafts	$ 2,122,985	1	0.5	This option significantly reduces amount of fill required because little or no fill is needed in front of the walls.
South Abutment				
Excavate unsuitable soils and replace with suitable compacted backfill	$ 889,107	4	0.6	Risks associated with temporary shoring. Additionally, relocating the creek or dewatering to facilitate excavation will expose shale resulting in softening and risk to long-term stability.
MSE Wall with drilled shaft supporting abutment	$ 912,306	1	1.1	Excavation required for MSE wall relatively shallow at the south side and is above water. Drilled shafts will contribute to the stability of the wall as well as serve as abutment foundations.
Geopiers® for ground improvement for global stability	$ 1,674,832	4	0.3	Risks associated with drilling and compacting granular/stone fill for geopiers under ground water table. Significant risk associated with the geopiers providing a path for water to soften shale at bottom of piers and jeopardizing the stability of the abutments.
Tie-back SPL wall at abutments	$ 865,263	2	0.8	Alternative assumes a tie-back wall in front of the abutment and west side. The tieback wall will resist the lateral loads due to earth backfill embankment behind.
Tangent drilled shafts	$ 1,069,632	1	0.9	This option significantly reduces amount of fill required because little or no fill is needed in front of the walls.

ADDITIONAL SUBSURFACE INVESTIGATION PROGRAM

Due to the significant costs associated with the various mitigation measures presented in Table 3, a supplemental subsurface exploration program was executed in 2004 to verify the shear strength parameters of the highly weathered shale and to obtain the in situ shear strength parameters. The 2004 program involved the drilling of 10 borings in and around the creek location. The borings were located to maximize the possible coverage of the area likely to require remediation measures. In situ testing comprising the Iowa borehole shear test (BST) was conducted at various depths and in the different layers encountered, especially at or near the soil-highly weathered shale interface, to obtain the in situ drained shear strength parameters. The depths of the soil-highly weathered shale interface were estimated based on the 2001 subsurface profiles and by obtaining split-spoon samples. Each boring was terminated in slightly weathered shale after coring for 1.5 meters. BSTs with high pressure plates or rock BSTs were conducted to obtain the strength values in moderately and slightly weathered shale layers. A total of 35 soil and rock BSTs were performed as part of this field exploration. Water levels in the borings were recorded after at least 24 hours after the end of drilling. Relatively undisturbed Shelby tube samples were usually obtained immediately below the interface or at the interface by drilling in an adjacent location.

Iowa Borehole Shear Test

The Iowa BST was developed by Dr. R. L. Handy at Iowa State University in 1967 (Handy and Fox 1967). The test is a drained shear test where the soil is relatively free-draining, since the drainage path from the shear head serration is short, and when the test is performed in the displacement range of about 0.5 mm/min or less. The test is applicable for all fine-grained soils and may be done even where a trace of gravel is present (Bowles 1997). The BST is a staged test, repeatedly consolidating and shearing the soil in essentially the same position in the boring. Drainage times, therefore, are cumulative.

The philosophy of the test is to perform a shear test in situ on the sides of a three-inch–diameter borehole in order to obtain independent measurements of soil friction and cohesion. An expandable shear head with diametrically opposed serrated shear pressure plates is lowered into the borehole to the desired depth. The serrations on the plates are oriented perpendicular to the axis of the hole. A constant normal force is then applied to the plates, causing the plates to grip the sides of the borehole. After a 5- to 15-minute waiting period to allow the surrounding soil to consolidate, the soil is sheared by pulling the shear head upwards until a peak shear force is recorded (Lutenegger and Timian 1987). Thus, a single point on the Mohr-Coulomb failure surface is obtained. The test is repeated to obtain additional data points at successively higher normal stresses to obtain a Mohr-Coulomb failure surface. The standard pressure plates may be replaced by smaller, high-pressure plates if the test is to be conducted in stiff or firm soils or soft shale.

The philosophy behind the rock BST is similar to the soil BST. The few notable differences being that the apparatus used is slightly differently and modified to fit the rock core hole and the shear stress is applied by means of a hydraulic jack. Unlike the soil BST, the rock BST is not a staged test. The shear head apparatus is retrieved after each test, cleaned, rotated about 45 degrees, and then re-inserted into the hole to obtain the next failure point at a higher normal stress than the previous point. The test is repeated until a failure surface is obtained.

Laboratory Testing

The 2004 testing program consisted of six consolidated-undrained (CIU) triaxial tests with pore pressure measurements, one consolidated-drained (CD) test, five direct shear (DS) tests, one consolidation test, and four unconfined compression (UC) tests on rock cores. All testing for the 2004 program was done as per applicable ASTM standards.

SUBSURFACE CONDITIONS

The final subsurface profiles developed for the bridges are based on the combined data available from the 2001 and 2004 exploration programs. The data were used to reevaluate and modify the 2001 subsurface profile. The final profiles indicate a substantial difference in the subsurface conditions between the north and south sides of the creek. The generalized subsurface profiles along the north and south sides of the creek are presented in Figures 3 and 4, respectively. A typical final profile is presented in Figure 5.

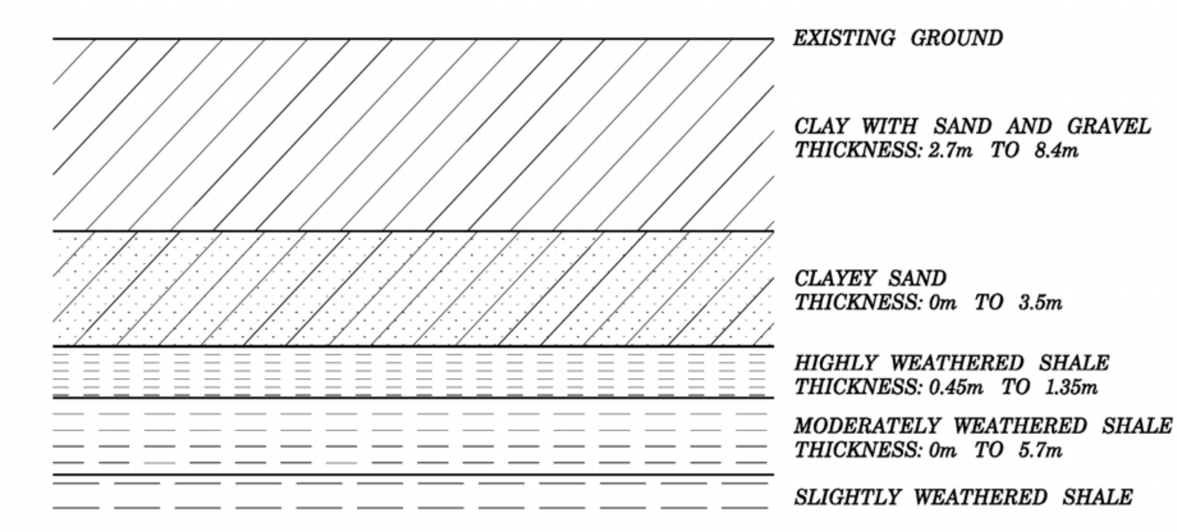

Figure 3. Generalized subsurface profile (north of creek)

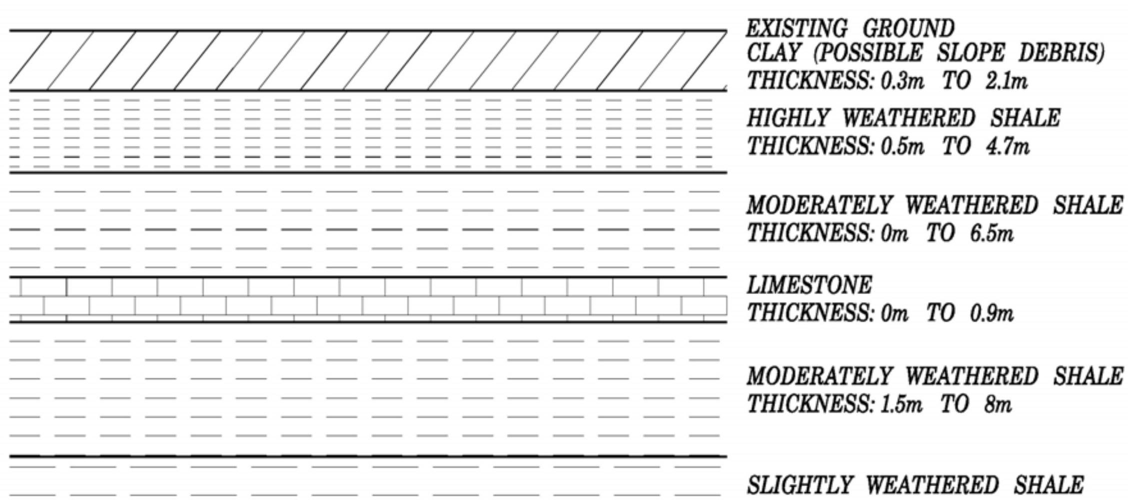

Figure 4. Generalized subsurface profile (south of creek)

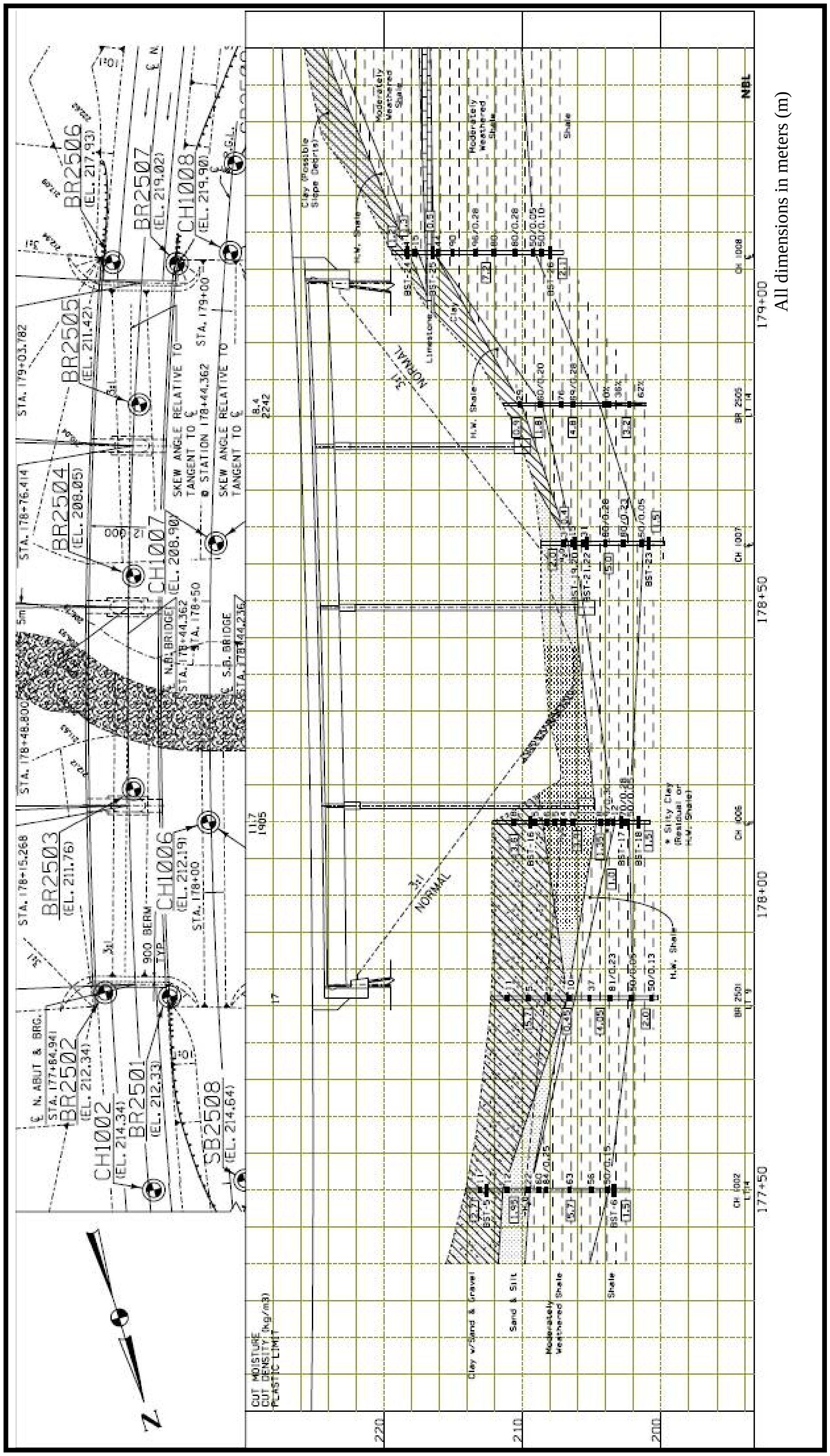

Figure 5. Final subsurface profile (typical)

INTERPRETATION OF RESULTS AND RECOMMENDED DESIGN PARAMETERS

BSTs were performed to obtain the drained shear strength parameters of the various soil and shale layers to estimate the shear strength of the layers under long-term drained loading. The main advantage of performing the BST as compared to the conventional laboratory tests is testing the soil in its in situ state of stress, thereby minimizing the uncertainty in soil parameters associated with sampling disturbance.

Laboratory tests, such as the DS tests and the CD test, were performed on samples obtained at or near the BST locations to compare the results with the BST results. These results were sorted according to the various layers. For each layer, the average values of the cohesion and friction angles and their respective standard deviations were calculated. The standard deviation is a measure of how widely the values are dispersed from the average values. The test results from the layers that will be excavated and replaced with fill as part of a settlement mitigation measure were not included in the statistical calculations. Table 4 presents a summary of the strength parameters sorted according to the various layers.

Table 4. Summary of drained (effective stress) shear strengths (sorted per layer)

Soil	Depth (m)	BST Results		CIU test results		DS test results		Remarks
		ϕ' (deg.)	c' (kPa)	ϕ' (deg.)	c' (kPa)	ϕ' (deg.)	c' (kPa)	
clay	0.9–6.80	18	39	25	7			
S.D.		2	18					
h.w.sh	0.6–10.30	16	42	24	7			
S.D.		5	17					
limestone /shale	2.92	21	0					Assume c'=0.
m.w.sh	3.20–10.50	20	82			20	19	
S.D.		9	113			2	8	
s.w.sh	7.80–12.50	23	675					
S.D.		11	1254					

S.D.= standard deviation
h.w.sh.= highly weathered shale, m.w.sh.=moderately weathered shale, s.w.sh.=slightly weathered shale

Table 5. Summary of undrained (total stress) shear strengths (sorted per layer)

Soil	Depth (m)	CU/CIU test results[1]		UC test results	Remarks
		ϕ (deg)	c (kPa)	Su (kPa)	
clay	1.20-5.40	11	51		
S.D.		7	51		
h.w.sh.	5.80-6.40	15	19		
s.w.sh.	7.40-11.80			605	
S.D.				569	

[1] CU tests done on samples from 2001 exploration program

CU, CIU, and UC tests were performed to obtain the undrained shear strength of the soil and shale under end-of-construction or short-term loading. The results were sorted according to the various soil layers. For each layer, the average values of cohesion and friction angle and their respective standard deviations, where applicable, were calculated. The test results from the layers that will be excavated and replaced with fill as part of a settlement mitigation measure were not included in the statistical calculations. Table 5 presents a summary of the strength parameters sorted according to the various layers.

The recommended design parameters under drained and undrained conditions for each layer for use in the global slope stability analyses are presented in Tables 6 and 7, respectively. The authors feel that these values are appropriate, since they reflect the stress state of the soil at the creek location and are therefore more realistic.

Table 6. Recommended design shear strength parameters (drained)

Layer	UW	Strength type	c' (kPa)	ϕ' (deg)	Std. dev.[1] (c'/ϕ')	Rel. min.[2] (c'/ϕ')	Rel. max.[2] (c'/ϕ')	Remarks
Fill*	20.4	Mohr-Coulomb	29	12				Iowa DOT guidelines
Clay w/ sand	20	Mohr-Coulomb	39	18	18/2	16/2	25/3	
Sand and silt	19	Mohr-Coulomb	-	30				
h.w.sh.	21	Mohr-Coulomb	42	16	17/5	21/5	24/7	
m.w.sh.	21	Mohr-Coulomb	82	20	113/9	76/9	252/18	
s.w.sh.[3]	21	Mohr-Coulomb	675	23				
Limestone**	22	Mohr-Coulomb	-	21				BST at shale/Lst interface

Table 7. Recommended design shear strength parameters (undrained)

Layer	UW	Strength Type	c (kPa)	ϕ (deg)	Std. Dev.[1] (c/ϕ)	Rel. Min.[2] (c/ϕ)	Rel. Max.[2] (c/ϕ)	Remarks
Clay w/ sand	20	Mohr-Coulomb	51	11	51/7	37/8	59/12	
Sand and silt	19	Mohr-Coulomb	-	30				Assumed
h.w.sh.	21	Mohr-Coulomb	19	15				Data from single test
m.w.sh.[3]	21	Undrained	100	-				Assumed
s.w.sh.[3]	21	Undrained	605	-				Average value from UC
Limestone**	22	Undrained	150	-				Assumed

[1] The standard deviation is a measure of how widely values are dispersed from the average value (the mean).
[2] The relative minimum and maximum values are the difference between the average value and the minimum and maximum values respectively.
* Fill is assumed to be compacted and drained during installation.
** Limestone layer encountered at south abutment locations only.
[3] Since the average shear strengths of moderately weathered shale and slightly weathered shale are either assumed as indicated or dispersed over a high range, no statistical distribution is recommended for these layers.

SLOPE STABILITY ANALYSES

Stability of the embankment slopes was evaluated using the computer program SLIDE. SLIDE is a 2D slope stability program for evaluating the stability of circular or non-circular failure surfaces in soil or rock slopes. SLIDE analyzes the stability of slip surfaces using vertical slice limit equilibrium methods. In a traditional slope stability analysis, it is assumed that the values of all model input parameters are known exactly, or average values are used. Hence, for a given slip surface, a single safety factor value is calculated. This type of analysis is referred to as a deterministic analysis.

In a probabilistic slope stability analysis, statistical distributions can be assigned to model input parameters, such as material properties, support properties, loads, water table location, etc. to account for the degree of uncertainty in the parameters. Input data samples are randomly generated based on the user-defined statistical distributions. A given slip surface may then have many different safety factor values calculated. The result is a distribution of safety factors from which a probability of failure (PF) for the slope can be calculated (Rocscience 2003).

Global Minimum Method

The global minimum probabilistic analysis in SLIDE is commonly used in slope stability analysis. With this method, the deterministic slope stability analysis is first carried out to determine the slip surface with the overall (global minimum) factor of safety for all the slip surfaces analyzed.

The probabilistic analysis is then carried out on the global minimum slip surface, using the samples generated for each random variable. This means that the slope stability calculation is repeated N times (where N = number of samples) for the global minimum slip surface. This will result in N calculated safety factors. The PF is then the number of analyses that result in a safety factor less than the minimum desired value, divided by the total number of samples. Each analysis method in SLIDE (e.g., Bishop, Janbu, etc.) can result in a different global minimum slip surface. Hence, the probabilistic analysis is carried out independently on each global minimum slip surface which may result from each analysis ethod.

The global minimum probabilistic analysis assumes that the PF calculated for the (deterministic) global minimum slip surface is representative of the PF for the entire slope.

A total of seven critical sections (one along each lane per abutment in the longitudinal direction, two in the transverse direction along the north abutment, and one in the transverse direction at the south abutment) were analyzed for global slope stability. Each section was analyzed for the undrained (end-of-construction) loading and the drained (long-term) loading case. For each case, each section was further analyzed for slope stability for circular and non-circular failure surfaces using modified Bishop and corrected Janbu methods. The sections at the south abutment locations were analyzed while assuming the existing soils and highly weathered shale would be excavated and replaced with fill as part of a settlement mitigation measure.

Undrained (End-of-Construction) Analyses

Global slope stability analyses for the undrained loading condition were performed on the seven sections as described in the preceding section using the recommended undrained design parameters for the various soil layers, as presented in Table 7. The analyses were performed assuming the water table to be static and as shown in the subsurface profiles. The 24-hour water levels were considered as the water table surface. For clayey soils, the increase in undrained shear strength due to consolidation under the embankment fill was ignored. The analyses were performed assuming a uniformly distributed load of 51kN/m to be acting on the roadway surface to account for the 2.5-meter–high temporary surcharge recommended as part of the settlement analysis. The results of the SLIDE analyses are presented in Table 8 and a typical graphical output is presented in Figure 6.

Table 8. Stability analyses results using recommended shear strength parameters (undrained analysis)

Location of slope section analyzed	Search method	FS with average measured shear strength (deterministic)	Minimum FS with average measured shear strength and statistical distribution	Average FS with average measured shear strength and statistical distribution	PF with average measured shear strength and statistical distribution (%)	Remarks
North abutment profile (NBL)	Circular	1.5	1.3	1.5	1.3	OK
	Block	1.4	1.2	1.4	12.8	
North abutment profile (SBL)	Circular	1.5	1.2	1.5	3.1	
	Block	1.4	1.2	1.4	10.5	
North abutment transverse (right) Sta.177+82	Circular	1.8	1.3	1.8	<0.1	
	Block	1.7	1.3	1.7	<0.1	
North abutment transverse (left) Sta.177+82	Circular	1.7	1.5	1.8	<0.1	
	Block	1.6	1.4	1.6	<0.1	
South abutment profile (NBL)	Circular	1.7	1.7	1.7	<0.1	
	Block	1.5	1.5	1.5	<0.1	
South abutment profile (SBL)	Circular	1.7	1.7	1.7	<0.1	
	Block	1.6	1.6	1.6	<0.1	
South abutment transverse (right) Sta.179+04	Circular	2.2	2.2	2.2	<0.1	
	Block	2.0	2.0	2.0	<0.1	

Notes:

1. South Abutment analysis done assuming clay, sand, and h.w.sh. layer excavated and replaced with fill.
2. A UD dead load of 51kN/m due to 2.5-meter–high temporary surcharge was assumed to be acting on the embankment fill at the north abutment location.
3. Error codes generated by SLIDE have been checked and found not to have any impact on the results.

As presented in Table 8, the slopes appear to be stable, with safety factors greater than the Iowa DOT minimum of 1.3 and with a P.F. less than 0.1 for all sections except the sections along the north abutment profile under both lanes. For these sections, the deterministic safety factor is greater than 1.3. However, the minimum safety factor is less than 1.3 for the probabilistic analysis, with the highest PF of 12.8% for the section at the northbound lane in block failure with a safety factor of 1.2. It is the authors' opinion that these PFs are acceptable, especially since the deterministic safety factors are greater than 1.3 and the minimum safety factors are greater than 1.0. These PF values are also acceptable, since the analysis has been carried out assuming a temporary surcharge on the embankment fill, and the slopes should be stable with safety factors higher than 1.3 once the surcharge has been removed.

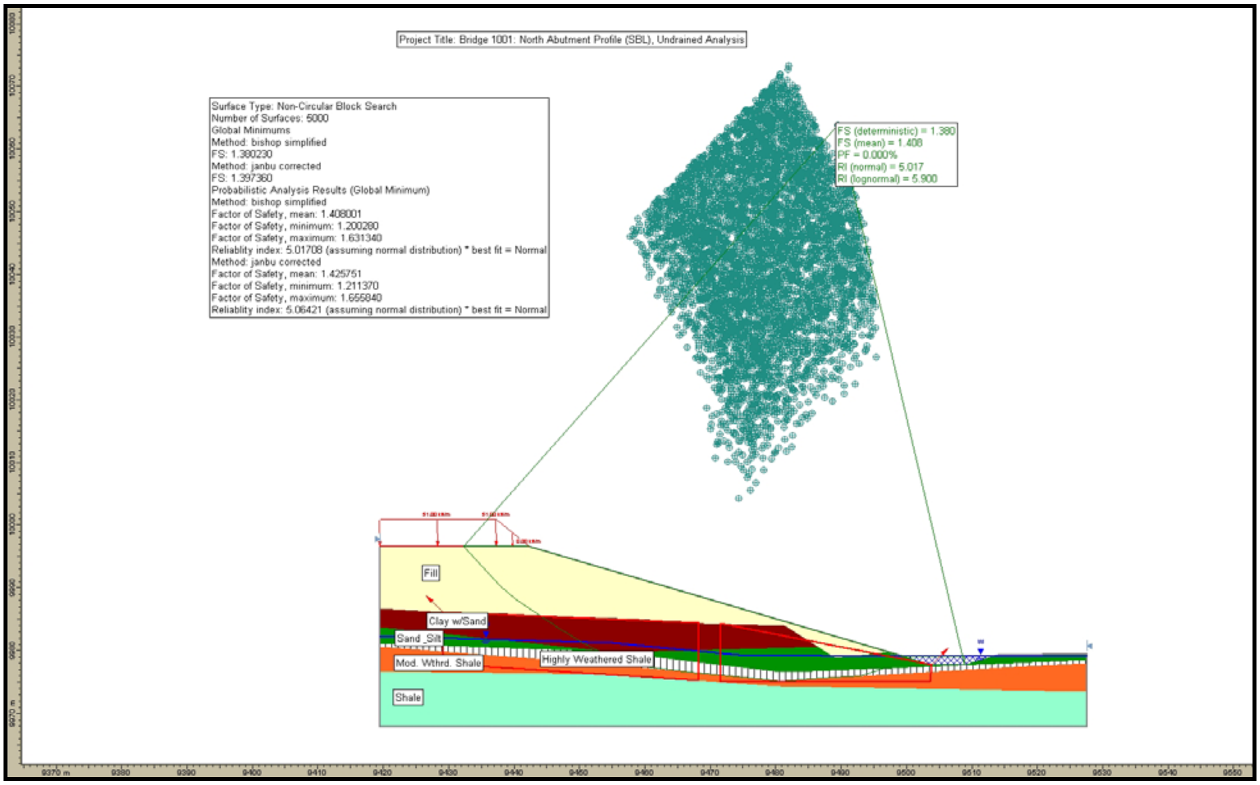

Figure 6. Typical SLIDE output (undrained analysis)

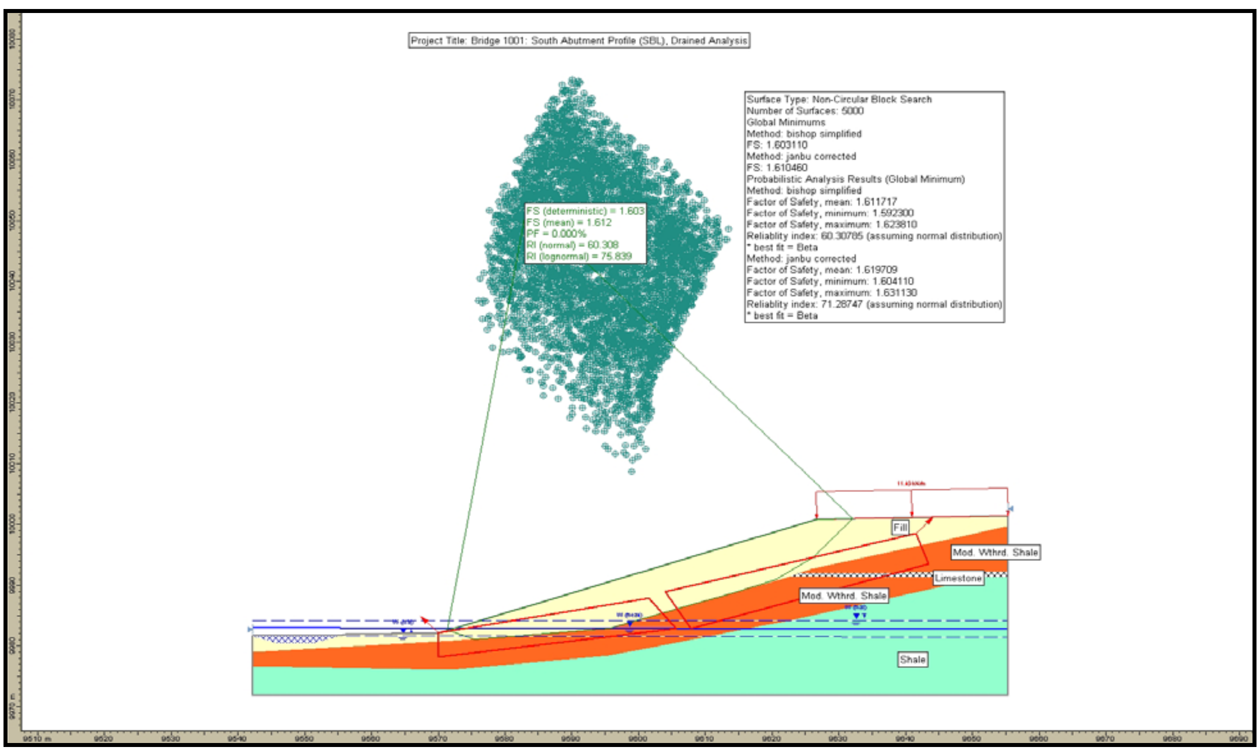

Figure 7. Typical SLIDE output (drained analysis)

Drained (Long-Term) Analyses

Global slope stability analyses for the drained loading case were performed on the seven sections using the recommended drained design parameters and statistical distributions for the various soil layers, as presented in Table 6. The drained case (long-term) loading condition was analyzed assuming a uniformly distributed live traffic load of 11.49kN/m acting on the roadway surface. Statistical distribution was assumed for the water table by assuming the water table to vary between the 24-hour levels recorded during the 2004 exploration as a minimum and the 500-year flood level as a maximum. An exponential distribution was used to simulate the occurrence of the 500-year flood level (i.e., the majority of samples would be generated closer to the minimum water table location, and the maximum water table location would only have a small probability of occurring).

A typical graphical output is presented in Figure 7 and the results of the analyses are presented in Table 9. As presented in Table 9, the slopes appear stable under the long-term conditions, with safety factors greater than 1.3 and PFs less than 0.1.

Table 9. Stability analyses results using recommended shear strength parameters (drained analysis)

Location of slope analyzed	Search method	FS with average measured shear strength (deterministic)	Minimum FS with average measured shear strength and statistical distribution	Average FS with average measured shear strength and statistical distribution	PF with average measured shear strength and statistical distribution (%)	Remarks
North abutment arofile (NBL)	Circular	1.7	1.6	1.8	<0.1	
	Block	1.8	1.7	1.8	<0.1	
North abutment profile (SBL)	Circular	1.8	1.7	1.9	<0.1	
	Block	1.7	1.4	1.8	<0.1	
North abutment transverse (right) Sta.177+82	Circular	2.2	2.1	2.2	<0.1	
	Block	2.1	1.7	2.2	<0.1	
North abutment transverse (left) Sta.177+82	Circular	2.0	2.0	2.0	<0.1	
	Block	2.0	1.6	2.0	<0.1	
South abutment profile (NBL)	Circular	1.7	1.6	1.7	<0.1	See Note 1 below for south abutment analysis
	Block	1.5	1.5	1.6	<0.1	
South abutment profile (SBL)	Circular	1.7	1.6	1.7	<0.1	
	Block	1.6	1.6	1.6	<0.1	
South abutment transverse (right) Sta.179+04	Circular	2.1	2.1	2.1	<0.1	
	Block	2.0	2.0	2.0	<0.1	

Notes:

1. South abutment analysis was performed assuming clay, sand, and h.w.sh. layer excavated and replaced with fill.
2. Water table was also used in probability analyses by assuming the existing water level as minimum and 500-yr flood level (El. 210m) as max.
3. A UD live load of 11.49kN/m was assumed to be acting on the embankment fill.
4. Error codes generated by SLIDE have been checked and found not to have any impact on the results.

CONCLUSIONS AND RECOMMENDATIONS

1. Slope stability analyses indicate the slopes at the north side of the creek to be stable under the additional embankment and 2.5-meter temporary surcharge in undrained loading and traffic loading and 500-yr flood conditions in a drained loading. Hence, no slope mitigation measures were required at either the north or south sides, from the slope stability perspective. However, at the south side, the highly weathered shale is encountered at very shallow depths below the ground surface. It was recommended that this shale be excavated and replaced by compacted fill to mitigate settlement and avoid downdrag on the piles supporting the abutments.
2. Additionally, it is recommended that inclinometers be installed during the construction of the embankment, scheduled to start sometime in 2006, so that the actual performance of the embankment can be monitored and the reliability of the analyses and design approach can be ascertained from the inclinometer readings.
3. The use of in situ testing was a cost-effective way of optimizing slope design. Roughly, for every $1 spent on in situ testing, the estimated construction cost savings are between $30 and $50.
4. The use of in situ testing, such as the borehole shear test, requires experienced personnel to perform the test. The quality of the results is greatly impacted by the operator's skill.
5. In general, it is recommended that at any given location where highly weathered shale is likely to be encountered, the undrained slope stability analyses be performed using the Iowa DOT value of 10kPa. If the analysis indicates a high probability of failure with safety factors less than 1.0 and is likely to impact a large area and result in a significant cost to the project, additional subsurface investigation, including in situ testing, may be conducted to verify more realistic shear strength parameters. It is the authors' opinion that such an approach would possibly result in optimal design and reduce the construction cost at a relatively minor additional expense during the design.

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Prediction of National Airport Pavement Test Facility Pavement Layer Moduli from Heavy Weight Deflectometer Test Data Using Artificial Neural Networks

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ABSTRACT

The National Airport Pavement Test Facility (NAPTF) was constructed to generate full-scale testing data to investigate the performance of airport pavements subjected to complex gear loading configurations of new generation aircraft. During the first test program, the NAPTF test sections were simultaneously subjected to Boeing 777 trafficking in one lane and Boeing 747 trafficking in another lane using the National Airport Pavement Test Machine. To monitor the effect of time and traffic on pavement structural responses, heavy weight deflectometer (HWD) tests were conducted on the trafficked lanes and the untrafficked centerline of flexible test sections as trafficking progressed.

The primary objective of this study was to develop a tool for backcalculating NAPTF non-linear flexible pavement layer moduli from HWD data using artificial neural networks (ANN). A multi-layer, feed-forward network that uses an error-backpropagation algorithm was trained to approximate the HWD backcalculation function. The synthetic database generated using the non-linear pavement finite element program, ILLI-PAVE, was used to train the ANN. Using the ANN, the asphalt concrete moduli and subgrade moduli were successfully predicted. Further research is required to develop ANN models for predicting the granular layer moduli. These results could be used to compare the relative effect of Boeing 777 and Boeing 747 trafficking on the elastic moduli and characterize the seasonal variation in moduli values. The same concept could also be used for backcalculating non-linear pavement moduli of highway pavements for input into mechanistic-empirical analysis and design.

Key words: artificial neural networks—ILLI-PAVE—National Airport Pavement Test Facility—pavement moduli

INTRODUCTION

The Federal Aviation Administration's (FAA) National Airport Pavement Test Facility (NAPTF) is located at the Atlantic City International Airport, New Jersey. It was constructed to generate full-scale test data needed to develop pavement design procedures for the new generation of large civil transport aircraft, including the Boeing 777 and Boeing 747. During the first series of tests, two gear configurations, a six-wheel tridem landing gear (Boeing 777) in one lane and a four-wheel dual-tandem landing gear (Boeing 747) in the other lane, were tested simultaneously. Heavy weight deflectometer (HWD) tests were conducted at regular time intervals as trafficking continued. The primary objective of this study was to develop a tool for backcalculating NAPTF non-linear pavement layer moduli from HWD test data using artificial neural networks (ANN).

The elastic layer moduli backcalculated from non-destructive test results are good indicators of pavement layer condition (Xu, Ranjinathan, and Kim 2001) as well as required inputs for the a priori mechanistic design of a flexible pavement. The backcalculation approach is particularly appealing for characterizing subgrade soils that display large variability in subgrade modulus (as large as 35%–50% over few miles of a pavement) (Thompson, Tutumluer, and Bejarano 1998).

Conventional elastic layer program (ELP)-based backcalculation software assumes that pavement materials are linear-elastic, homogenous, and isotropic. The non-linearity or stress-dependency of resilient modulus for unbound granular materials and cohesive fine-grained subgrade soils is well documented in the literature (Hicks 1970; Thompson and Robnett 1979). Previous studies have observed the non-linearity of underlying layers at the NAPTF. Gomez-Ramirez and Thompson (Garg and Marsey 2002) reported the presence of material non-linearity at NAPTF by separately analyzing the individual layer compression from multi-depth deflectometer (MDD) readings. Garg and Marsey (2002) have similarly observed the stress-dependent nature of the granular and subgrade layers in NAPTF flexible test sections. Therefore, it is more realistic to use non-linear layer moduli for conducting NAPTF pavement structural analysis and for studying the variation in moduli with trafficking.

ILLI-PAVE is a two-dimensional axi-symmetric pavement finite-element software developed at the University of Illinois at Urbana-Champaign (UIUC) (Raad and Figueroa 1980). It incorporates stress-sensitive material models and provides a more realistic representation of the pavement structure and its response to loading (NCHRP 1990). Based on extensive repeated laboratory testing data at UIUC, Thompson and Robnett (1979) indicated that the breakpoint resilient modulus (E_{Ri}), typically associated with a repeated deviator stress of about six psi, is a good indicator of the subgrade soil's resilient modulus. The Asphalt Institute's Thickness Design Manual MS-1 (Asphalt Institute 1982) recommends E_{Ri} (subgrade modulus at a deviator stress of six psi) as the subgrade modulus input for ELP analysis. The E_{Ri} is also one of the subgrade material property inputs to ILLI-PAVE. However, there is no commercial ILLI-PAVE-based backcalculation program currently available. Previous research at UIUC showed that the asphalt concrete moduli and non-linear subgrade moduli (E_{Ri}) could be successfully predicted using an ANN trained with the ILLI-PAVE database (Gopalakrishnan and Thompson 2004). Ceylan et al. (2004) demonstrated the use of ANNs trained with ILLI-PAVE results as pavement structural analysis tools for the rapid and accurate prediction of critical responses and deflection profiles of flexible pavements subjected to typical highway loadings. Ongoing research at Iowa State University on backcalculting pavement moduli using ANN is being performed under Dr. Halil Ceylan's supervision. However, the current research specifically focuses on developing a tool for backcalculating NAPTF pavement non-linear moduli using ANN trained with ILLI-PAVE results.

NATIONAL AIRPORT PAVEMENT TEST FACILITY

The NAPTF test pavement area is 900 feet (274.3 meters) long and 60 feet (18.3 meters) wide. The first test series installation included a total of nine test sections (six flexible and three rigid) built on three different subgrade materials: low-strength (target CBR of 4), medium-strength (target CBR of 8), and high-strength (target CBR of 20). Two different base sections are used in flexible test sections: conventional (granular) and stabilized (asphalt concrete). The low-strength and the medium-strength flexible test sections alone are considered in this study. The naturally occurring sandy soil material at the NAPTF site underlies each subgrade layer.

Pavement Sections

Each NAPTF test section is identified using a three-character code, where the first character indicates the subgrade strength (L for low, M for medium, and H for high), the second character indicates the test pavement type (F for flexible and R for rigid), and the third character signifies whether the base material is conventional-aggregate (C) or asphalt-stabilized (S). Thus, test section MFC refers to a conventional-base flexible pavement built over a medium strength subgrade, whereas test section MFS refers to a stabilized-base flexible pavement built over a medium-strength subgrade. Cross-sectional views of the as-built NAPTF flexible test items considered in this study are shown in Figure 1. Note that P-401 asphalt concrete was used in the surface layer and in the stabilized base layer as well.

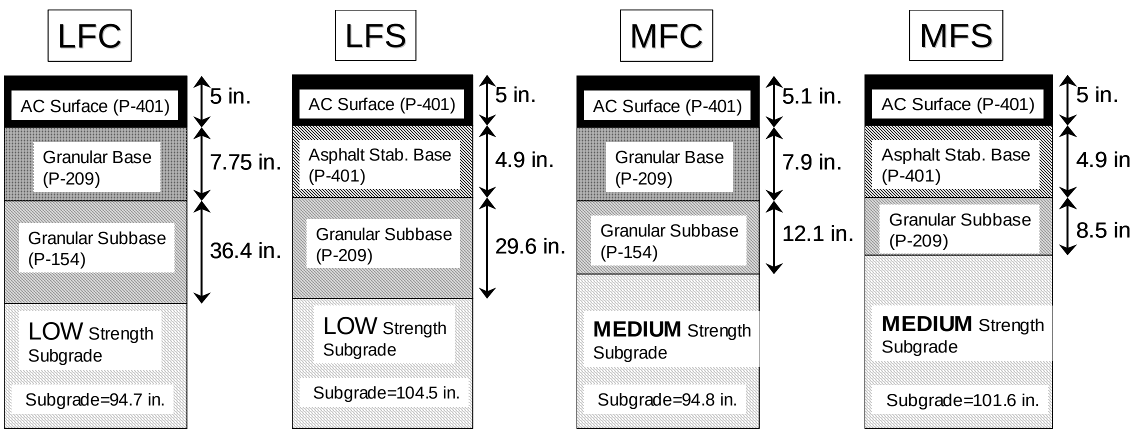

Figure 1. Cross-sectional views of as-built NAPTF flexible test sections

Traffic Testing

A six-wheel dual-tridem gear configuration (Boeing 777) with 54-inch (1,372-mm) dual spacing and 57-inch (1,448-mm) tandem spacing was loaded on the north wheel track, while the south side was loaded with a four-wheel dual-tandem gear configuration (Boeing 747) having 44-inch (1,118-mm) dual spacing and 58-inch (1,473-mm) tandem spacing. The wheel loads were set to 45,000 lbs (20.4 tons) each and the tire pressure (cold) was 188 psi (1,295 Kpa). In the LFC (a conventional aggregate-base pavement built over low-strength subgrade) and LFS (an asphalt-stabilized base pavement built over low-strength subgrade) test sections, the wheel loads were increased from 45,000 lbs (20.4 tons) to 65,000 lbs (29.4 tons) after 20,000 initial load repetitions. Throughout the traffic test program, the traffic speed was 5 mph (8 kmh).

NAPTF Flexible Pavement Failure Criterion

The NAPTF failure criterion is based on the criterion utilized by the U.S. COE MWHGL Tests (Ahlvin et al. 1971). It is defined as 1-inch (25.4-mm) surface upheaval adjacent to the traffic lane. This is considered to reflect a structural or shearing failure in the subgrade.

Data Availability

All test data referenced in this paper are available for download on the FAA Airport Pavement Technology website: http://www.airporttech.tc.faa.gov/naptf/.

HEAVY WEIGHT DEFLECTOMETER TESTS

For HWD testing, the FAA HWD KUAB Model 240, configured with a 12-inch loading plate and a 27–30 msec pulse width, was used. The deflections were measured with seven seismometers at offsets of 0-inch (D_0), 12-inch (305-mm) (D_{12}), 24-inch (610-mm) (D_{24}), 36-inch (914-mm) (D_{36}), 48-inches (1,219-mm) (D_{48}), and 60-inch (1,524-mm) (D_{60}) intervals from the center of the load.

The HWD tests were performed at nominal force amplitudes of 12,000 lbs or 12 kip (53.4 kN), 24,000 lbs or 24 kip (106.8 kN), and 36,000 lbs or 36 kip (160.2 kN). These tests were performed on the centerline, Boeing 777 traffic lane (Lane 2) and Boeing 747 traffic lane (Lane 5). The HWD test sequences were repeated at 10-foot (3.1-meter) intervals along the test lanes. The location and orientation of HWD test lanes are illustrated in Figure 2.

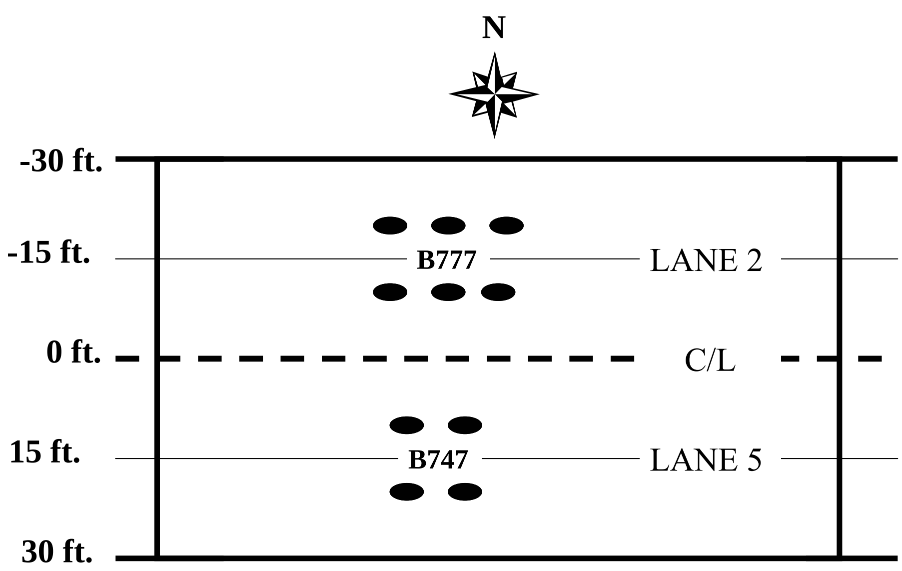

Figure 2. NAPTF HWD test lanes

Pavement Temperature

The temperature of the asphalt concrete layer at the time of FWD testing has a significant influence on the surface deflections. During the NAPTF construction, static temperature sensors were installed at different depths along the test sections to record the pavement temperatures at different times of the day. The HWD tests were all conducted between January 11, 2000 and June 6, 2001. The asphalt concrete temperature varied between 40 ^{0}F to 75 ^{0}F during the entire duration of traffic testing.

DATABASE GENERATION USING ILLI-PAVE

To generate the synthetic database for training the ANN, each NAPTF flexible test section was modeled in ILLI-PAVE. The as-constructed layer thicknesses (see Figure 1) were used for each test section. The individual pavement layers were characterized as follows. The asphalt concrete layer and the sand layer were treated as linear elastic material. Stress-dependent elastic models along with Mohr-Coulomb failure criteria were applied for the base, subbase, and subgrade layers. The stress-hardening K-θ model was used for the base and subbase layers:

$$M_R = \frac{\sigma_D}{\varepsilon_R} = K\theta^n \quad (1)$$

Where M_R is resilient modulus (psi), θ is bulk stress (psi), and K and n are statistical parameters.

The following relationship exists between K and n (R^2 = 0.68, SEE = 0.22) (Rada and Witczak 1981):

$$\text{Log}_{10}(K) = 4.657 - 1.807n \quad (2)$$

The stress-softening bilinear model was used for the subgrade layer:

$$\begin{aligned} M_R &= M_{Ri} + K_1.(\sigma_d - \sigma_{di}) \quad \textit{for } \sigma_d < \sigma_{di} \\ M_R &= M_{Ri} + K_2.(\sigma_d - \sigma_{di}) \quad \textit{for } \sigma_d > \sigma_{di} \end{aligned} \quad (3)$$

Where M_R is resilient modulus (psi), σ_d is applied deviator stress (psi), and K_1 and K_2 are statistically determined coefficients from laboratory tests.

A total of 5,000 input cases were generated for each test section by randomly varying the asphalt concrete and subgrade layer moduli and the 'K_b'-'n_b' and 'K_s'-'n_s' values (note that K and n are related) for the base and subbase layers, respectively. The effect of 36-kip HWD loading was simulated in ILLI-PAVE and the pavement surface deflections were computed. Initially, it was decided to use separate ANN models for each section. Of the total number of data sets for each test section, 3,750 data vectors were used in training the ANN and the remaining 1,250 data vectors were used to test the network after the training was completed. The range of layer properties used in training the ANN is summarized in Table 1.

ARTIFICIAL NEURAL NETWORK ARCHITECTURE

A generalized n-layer feedforward ANN that uses an error-backpropogation algorithm (Haykin 1994) was implemented in the Visual Basic (VB 6.0) programming language. The program can allow for a general number of inputs, hidden layers, hidden layer elements, and output layer elements. Two hidden layers were found to be sufficient for solving a problem of this size, and therefore the architecture was reduced to a four-layer feedforward network. A four-layer feedforward network consists of a set of sensory units (source nodes) that constitute the input layer, two hidden layers of computation nodes, and an output layer of computation nodes. The following notation is generally used to refer to a particular type of architecture that has two hidden layers: (# inputs)-(# hidden neurons)-(# hidden neurons)-(# outputs). For example, the notation 10-40-40-3 refers to an ANN architecture that takes in 10 inputs (features), has 2 hidden layers consisting of 40 neurons each, and produces 3 outputs.

Table 1. Range of pavement layer properties used in generating the ANN training database

Pavement layer	Thickness (inches)	Elastic layer modulus (ksi)	Poisson's ratio
Asphalt concrete	5 - MFC &LFC 10 - MFS & LFS	100 – 2,500	0.35
Base	8 - MFC & LFC 8.5 - MFS 29.5 - LFS	K_b: 1.6 – 20 n_b: 0.2 – 0.8	0.35
Subbase	12 - MFC 36.4 - LFC	K_s: 1.6 – 20 n_s: 0.2 – 0.8	0.35
Subgrade	95 - MFC & LFC 105 - MFS & LFS	1.6 – 20	0.45
Sand	120 - Medium 144 - Low	45	0.4

An ANN-based backcalculation procedure was developed to approximate the HWD backcalculation function. Using the ILLI-PAVE synthetic database, the ANN was trained to learn the relationship between the synthetic deflection basins (inputs) and the pavement layer moduli (outputs).

Initialization of Weights

The first step in back-propogation learning is to initialize the network. It is recommended that the initialization of the synaptic weights of the network be uniformly distributed inside a small range. A range of -0.2 to +0.2 was used for random initialization of all synaptic weight vectors in the network.

Nonlinear Activation Function

The model of each neuron in the hidden layer(s) and output layer of the network includes a nonlinearity at the output end. The presence of a nonlinear activation function, φ(.), is important because otherwise the input-output relation of the network could be reduced to that of a single-layer perceptron. The computation of the local gradient for each neuron of the multilayer perceptron requires that the function φ(.) be continuous. In other words, differentiability is the only requirement that an activation function would need to satisfy.

For this problem, an asymmetric hyperbolic tangent function (tanh) was chosen for which the output amplitude lies inside the range $-1 \leq y_j \leq +1$. Since we require the final outputs to be real values instead of binary outputs, a linear combiner model was used for neurons in the output layer, thus omitting the nonlinear activation function.

Performance Measure (RMSE)

In order to track the performance of the network, the root mean squared error (RMSE) at the end of each epoch was calculated. An epoch is defined as one full presentation of all the training vectors to the network. The RMSE at the end of each epoch is defined as the following:

$$RMSE = \sqrt{\frac{\sum_{j=1}^{N}\left[d_j - Y\left(X_j\right)\right]^2}{N}}$$

Where d_j is the desired response for the input training vector X_j, and N is the total number of input vectors presented to the network for training.

For the network to learn the problem smoothly, a monotonic decrease in the RMSE is expected with an increase in the number of epochs. A smooth learning curve was achieved with a learning-rate parameter (η) of 0.001.

ANN INPUTS AND OUTPUTS

Deflection basin parameters (DBPs) derived from falling weight deflectometer (FWD) and/or HWD deflection measurements are shown to be good indicators of selected pavement properties and conditions (Hossain and Zanniewski 1991). Recently, Xu et al. (2001) used DBPs in developing new relationships between selected pavement layer condition indicators and FWD deflections by applying regression and ANN techniques. Apart from the six independent deflection measurements (D_0 to D_{60}), some of the commonly used DBPs were included as inputs for training the ANN. The DBPs considered in this study are shown in Table 2. Each DBP supposedly represents the condition of specific pavement layers. For example, AUPP is sensitive to the asphalt concrete layer properties, whereas BCI and AI4 are expected to reflect the condition of subgrade. The desired outputs from the ANN are asphalt concrete modulus (E_{AC}), subgrade modulus (E_{Ri}), base modulus parameter (K_b or n_b), and subbase modulus parameter (K_s or n_s). Note that by predicting either K or n, the other parameter can be determined using the relation proposed by Rada and Witzcak (1981).

Table 2. DBPs considered in this study

Deflection basin parameter (DBP)	Formula
AREA	$AREA = 6(D_0 + 2D_{12} + 2D_{24} + D_{36})/D_0$
Area under pavement profile (AUPP)	$AUPP = (5D_0 - 2D_{12} - 2D_{24} - D_{36})/2$
Area index	$AI_4 = (D_{36} + D_{48})/2D_0$
Base curvature index (BCI)	$BCI = D_{24} - D_{36}$ $BCI2 = D_{60} - D_{48}$
Base damage index (BDI)	$BDI = D_{12} - D_{24}$
Deflection ratio	$DR = D_{12}/D_0$
Shape factors	$F_1 = (D_0 - D_{24})/D_{12}$ $F_2 = (D_{12} - D_{36})/D_{24}$

SELECTION OF BEST-PERFORMANCE NETWORKS

Separate ANN models were used for each desired output rather than using the same architecture to determine all the outputs together. The most effective set of input features for each ANN model were determined based on both engineering judgment and the experience gained through past research studies conducted at UIUC. Parametric analyses were performed by systematically varying the choice and number of inputs and number of hidden neurons to identify the best-performance networks. As it was found that the prediction accuracy of the network remained the same for hidden layers greater than or equal to two, the number of hidden layers was fixed at two for all runs. The learning curve (RMSE vs number of epochs) and the testing RMSE were studied in order to arrive at the best networks. A previous study that focused on the MFC section alone showed that the base and subbase moduli parameters were the hardest to predict (Gopalakrishnan and Thompson 2004). During the course of this study, the same conclusion was reached for other test sections. It was concluded that further research is needed to develop robust ANN models for predicting the base and subbase moduli parameters.

RESEARCH RESULTS

A summary of the sensitivity analyses performed to select the best-performance networks for predicting asphalt concrete modulus (E_{AC}) and subgrade modulus (E_{Ri}) in NAPTF test sections are shown in Table 3. Note that the ANN inputs are similar for all four test sections. In Figure 3, the ANN-predicted moduli values and the target values are compared using the 1,250 test data vectors for each NAPTF section. Excellent agreement is found between the predicted and target values for both E_{AC} and subgrade modulus E_{Ri} in all four test sections, except for E_{Ri} in LFS section, where an R^2 value of 0.81 was obtained.

Table 3. Summary of best-performance ANN pavement moduli prediction models

NAPTF section	Output	Inputs	Network architecture	Training RMSE	Testing RMSE
MFC	E_{AC}	$D_0 \sim D_{60}$	6-40-40-1	71 ksi	69 ksi
	E_{Ri}	$D_0 \sim D_{60}$, BCI, AI_4	8-40-40-1	0.86 ksi	0.82 ksi
LFC	E_{AC}	$D_0 \sim D_{60}$	6-40-40-1	100 ksi	97 ksi
	E_{Ri}	$D_0 \sim D_{60}$, BCI, AI_4	8-40-40-1	1.29 ksi	1.18 ksi
MFS	E_{AC}	$D_0 \sim D_{60}$	6-40-40-1	69 ksi	67 ksi
	E_{Ri}	$D_0 \sim D_{60}$, BCI, AI_4	8-40-40-1	0.81 ksi	0.78 ksi
LFS	E_{AC}	$D_0 \sim D_{60}$	6-40-40-1	90 ksi	90 ksi
	E_{Ri}	$D_0 \sim D_{60}$, BCI, AI_4	8-40-40-1	2.36 ksi	2.15 ksi

One of the major reasons for developing this ANN-based backcalculation procedure is to evaluate the structural integrity of the NAPTF pavement test sections reliably as they were subjected to traffic loading. The NAPTF test sections were subjected to trafficking until they exhibited failure. The MFC test section was the first one to fail at 12,952 load repetitions exhibiting 3 to 3.5 inches of rutting and severe cracking. In the MFS section, localized failure occurred in the Boeing 777 traffic lane toward the west end. At 19,900 passes, 3.5 inches of rut depth was observed on the Boeing 777 traffic lane, with upheaval outside the traffic path. Trafficking was terminated on the Boeing 777 traffic lane, but it continued on the Boeing 747 traffic lane. The Boeing 747 lane failed at 30,000 passes. HWD tests were not conducted on the MFS section beyond 19,900 passes. The low-strength test sections (LFC and LFS) showed few signs of genuine distress, even after 20,000 passes, and therefore the wheel loading was increased from 45,000 lbs to 65,000 lbs. The trafficking was terminated in the low-strength test sections after 28,000 passes of 65,000 lbs. While the MFC and MFS sections failed at the subgrade level, the LFC and LFS sections failed in the surface layers, signifying tire pressure or other upper layer failure effects, but not subgrade level failure (Gervais, Hayhoe, and Garg 2003).

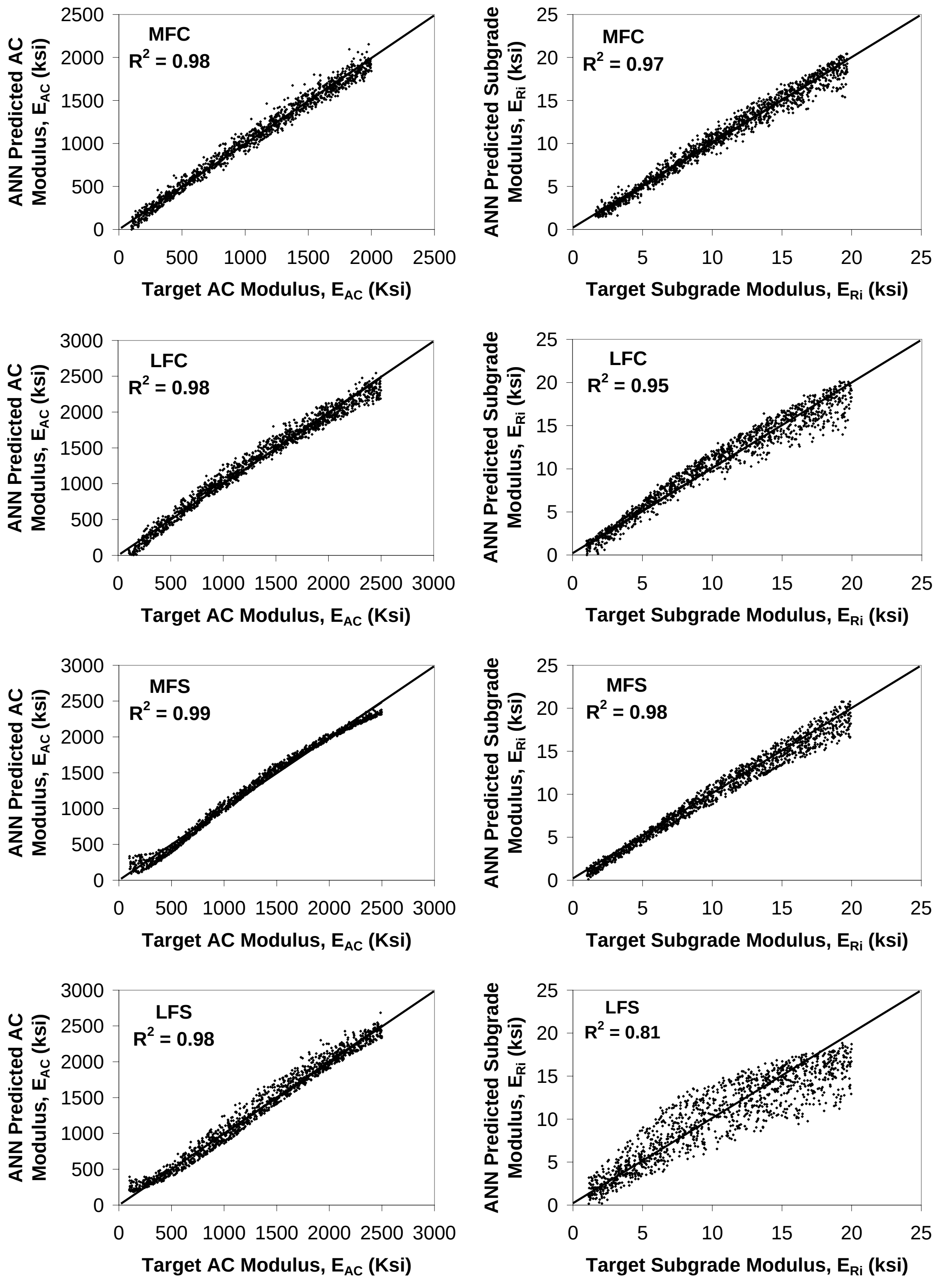

Figure 3. ANN prediction of NAPTF asphalt concrete moduli (left) and subgrade moduli (right)

ANN vs FAABACKCAL

Using the 36-kip HWD test data acquired at the NAPTF, the asphalt concrete moduli and subgrade moduli were backcalculated with the best-performance ANNs for all four sections. The results were then compared with those obtained using FAABACKCAL, an ELP-based backcalculation program. FAABACKCAL was developed under the sponsorship of the FAA Airport Technology Branch and is based on the LEAF layered elastic computation program. In this program, the pavement layer moduli and subgrade moduli are adjusted to minimize the root mean square (rms) of the differences between FWD/HWD sensor measurements and the LEAF-computed deflection basin for a specified pavement structure. A standard multidimensional simplex optimization routine is then used to adjust the moduli values (McQueen, Marsey, and Arze 2001). A stiff layer with a modulus of 1,000,000 psi and a Poisson's ratio of 0.50 was used in backcalculation. Based on the as-constructed conditions, the stiff layer was set at 10 feet for the medium-strength test sections and at 12 feet for the low-strength sections. The most recent version of the FAA backcalculation software is called BAKFAA. The detailed backcalculation results for NAPTF sections using FAABACKCAL are reported elsewhere (Gopalakrishnan and Thompson 2004).

The plots comparing the results of ANN-based backcalculation with those obtained using FAABACKCAL are shown in Figure 4 for asphalt concrete moduli and in Figure 5 for subgrade moduli. The gray arrow in the plots for the LFC and LFS sections indicate where (after 20,000 load repetitions) the wheel load was increased from 45,000 lbs to 65,000 lbs. In Figure 4, the variation in asphalt concrete temperature as a function of the number of load repetitions (N) in each test section is also included and is indicated by a gray line. In these plots, the changes in layer moduli in the Boeing 777 traffic lane and the Boeing 747 traffic lane are due to both traffic loading as well as variation in temperature and climate. The changes in pavement material properties in the untrafficked pavement centerline are only due to environmental effects. The trends for asphalt concrete moduli are very similar using both approaches. The asphalt concrete moduli values are significantly influenced by asphalt concrete temperature. In the MFC and LFC sections, the ANN approach seem to be more sensitive to trafficking and temperature, indicated by a sharp decrease in the moduli values towards the end of trafficking. In the LFC and LFS sections, the structural degradation resulting from increasing the wheel load from 45,000 lbs to 65,000 lbs after 20,000 repetitions is clearly reflected in the moduli values. Compared to the centerline moduli values, the traffic lane moduli values decrease further after 20,000 repetitions.

In Figure 5, the Y-axis is magnified in the individual plots as the subgrade moduli values varied over a narrow range compared to the asphalt concrete moduli. Note that the subgrade modulus predicted by ANN is the stress-dependent breakpoint subgrade modulus (at a deviator stress of about 6 psi) and is indicated as E_{Ri}. The subgrade modulus backcalculated by FAABACKCAL is indicated as E. The initial magnitudes of subgrade modulus obtained using both the approaches are similar for the MFS and LFS sections. For the MFC and LFC sections, the initial subgrade moduli obtained using the ANN are about 5 psi higher than the those obtained using FAABACKCAL. The sharp decrease in the subgrade modulus in the Boeing 777 traffic lane of the MFS section towards the end of the trafficking, as captured by the ANN results, could be due to the localized failure, as mentioned earlier. The NAPTF rutting study results showed that the eest end of the Boeing 777 traffic lane in the MFS section exhibited a rapid increase in surface rutting after 12,000 load repetitions (Gopalakrishnan and Thompson 2003).

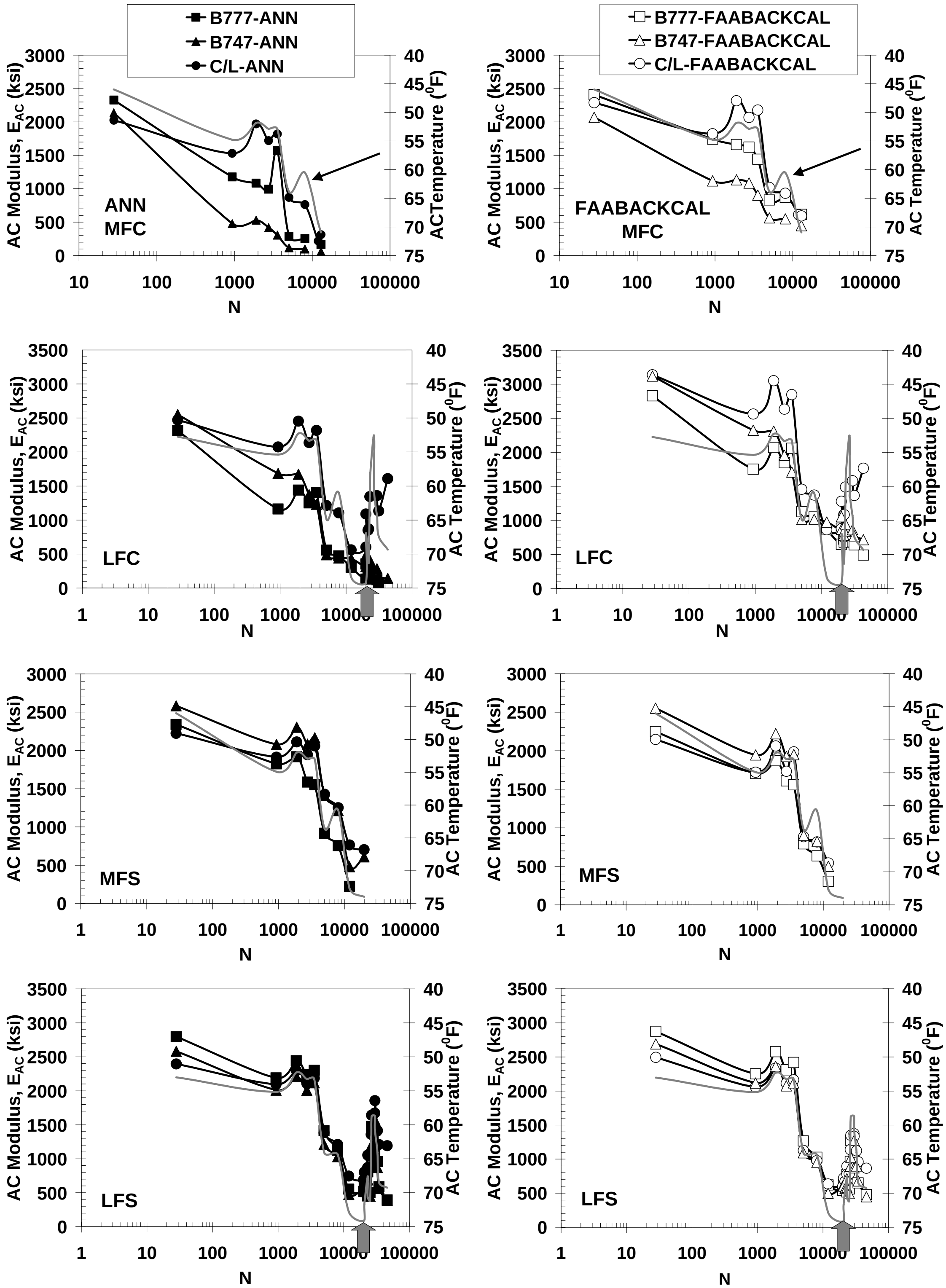

Figure 4. Comparison of ANN-predicted asphalt concrete moduli (left) with FAABACKCAL asphalt concrete moduli (right) for NAPTF test sections

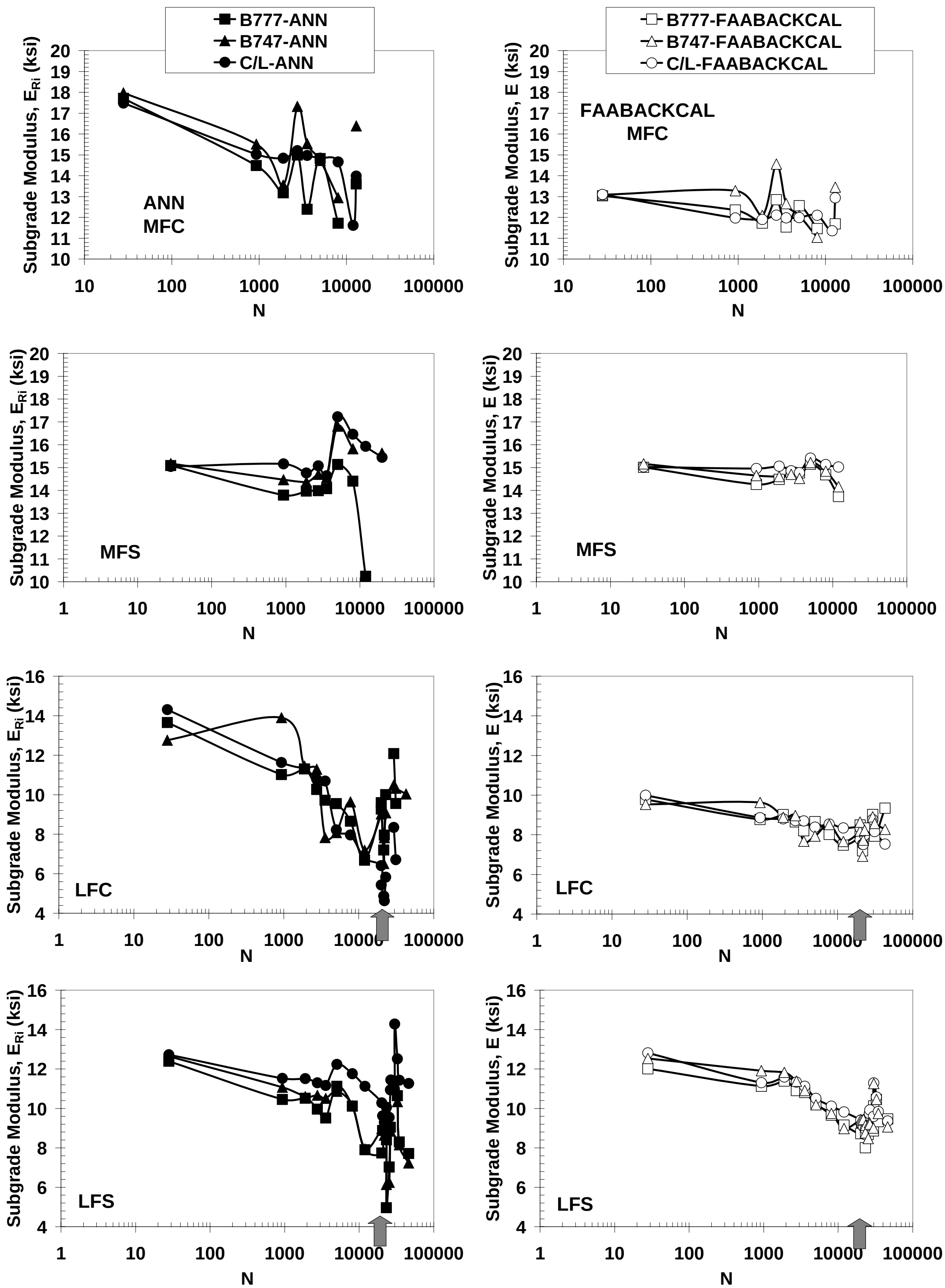

Figure 5. Comparison of ANN-predicted subgrade moduli (left) with FAABACKCAL subgrade moduli (right) for NAPTF test sections

SUMMARY AND CONCLUSIONS

The primary objective of this study was to develop a tool for backcalculating NAPTF non-linear pavement layer moduli from HWD data using ANN. The NAPTF sections were modeled in ILLI-PAVE and a synthetic database was generated for a range of moduli values. A multi-layer feedforward network that uses an error-backpropagation algorithm was successfully trained to approximate the HWD backcalculation function using the ILLI-PAVE database. ANN models were successfully developed for predicting asphalt concrete and non-linear subgrade moduli. However, the base/subbase moduli could not be predicted using ANN. The best-performance ANN models were used to predict asphalt concrete and subgrade moduli from NAPTF HWD deflection basins collected at regular intervals during trafficking. The results were compared with results obtained using FAABACKCAL, a conventional ELP-based backcalculation software.

The results definitely indicate that the non-linear pavement layer moduli could be successfully backcalculated from FWD/HWD deflection basins using an ANN trained with ILLI-PAVE results. The backcalculated results could be used to establish realistic a priori moduli inputs to mechanistic pavement structural analysis. Although regression models could also be developed for predicting layer moduli using ILLI-PAVE results, a significant advantage of using the ANN-based approach over the regression approach is that the functional forms of the relationships are not needed a priori. Also, the robustness of the ANN can be improved by including the field data sets in the training process, as they implicitly incorporate noise and errors typically seen in field measurements. Further research is needed to develop robust ANN models for predicting base/subbase moduli parameters. It is proposed that by including the ANN-predicted E_{AC} and E_{Ri} values as inputs to the ANN and by training the ANN with the ILLI-PAVE results for the 12-kip and 24-kip HWD loads, the chances of accurately predicting base/subbase moduli will increase. It is also noted that the ANN moduli backcalculation concept developed in this study could be used to backcalculate highway pavement non-linear moduli successfully.

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Internal Traffic Control Plans

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ABSTRACT

While a temporary traffic control plan describes how a specific work zone is to be set up to ensure the safety of the motoring public, construction equipment and vehicles within the work space are not addressed. An internal traffic control plan (ITCP) is a tool that project managers can use to coordinate and control the flow of construction vehicles, equipment, and workers operating in close proximity within the activity area to ensure the safety of workers. Recent federal regulations require safety plans of all construction contractors prepared by competent persons.

The concept of ITCP was developed as a promising intervention in research conducted by the Laborers' Health and Safety Fund for the Federal Highway Administration. This study found that only one-third of highway worker injuries are traffic-related, and that approximately half of the fatal injuries of workers on foot in the work space involve a backing construction vehicle. Among interventions that were developed during this research was the provision of an internal traffic control plan to control traffic within the work space. Model ITCPs were developed for paving, dirt spread, and trenching operations.

Additional accident data on worker injuries was collected and analyzed by Pratt et al. of the National Institute for Occupational Safety and Health (NIOSH). In 2002, NIOSH sponsored research to prove the applicability of the ITCP concept to paving operations on freeway work zones. Two asphalt paving and two portland cement concrete sites were observed in this research, and ITCPs were developed for each site. An ITCP development guide to aid contractors in preparing ITCPs was also developed in this research. This paper will describe the preparation of ITCPs and discuss field observations of asphalt and concrete paving operations. While additional research is needed, this intervention shows promise to reduce worker injuries in highway construction zones.

Key words: internal traffic control plan—worker injury—work space safety

BACKGROUND

Purpose of an Internal Traffic Control Plan

The *Manual on Uniform Traffic Control Devices* (MUTCD) (2003) defines a temporary traffic control (TTC) plan in Section 6C.01: "A temporary traffic control plan describes temporary traffic control measures to be used for facilitating road users through a work zone or incident area." The MUTCD also specifies that "TTC plans range in scope from being very detailed to simply referencing typical drawings contained in this Manual, standard approved highway agency drawings and manuals, or specific drawings contained in the contract documents."

However, in establishing TTC plans as a fundamental part of temporary traffic control, no provisions were made to control vehicle or pedestrian worker movements within the work space. The work space is shown only as a shaded area or black hole in most typical applications. In Section 6B.01–Fundamental Principles of Temporary Traffic Control of the MUTCD, the following guidance is given: "Road user and worker safety and accessibility in TTC zones should be an integral and high-priority element of every project from planning through design and construction."

The recently published MUTCD has several new guidance statements that relate to worker protection, and for the first time these statements are referenced to long standing OSHA regulations for workplace safety. Specifically in Section 6D.03–Worker Safety Considerations, two recommendations are as follows:

- "E. Activity Area–planning the internal work activity area to minimize backing-up maneuvers of construction vehicles should be considered to minimize the exposure to risk.
- F. Worker Safety Planning–a competent person designated by the employer should conduct a basic hazard assessment for the work site and job classifications required in the activity area. This safety professional should determine whether engineering, administrative, or personal protection measures should be implemented. This plan should be in accordance with the Occupational Safety and Health Act of 1970, as amended, "General Duty Clause" Section 5(a) (1) – Public Law 91-569, 84 Stat. 1590, December 29, 1970, as amended, and with the requirement to assess worker risk exposures for each job site and job classification, as per 29 CFR 1926.20 (b) (2) of "Occupational Safety and Health Administration Regulations, General Safety and Health Provisions."

Considerable research has addressed the problem of injuries to motorists traveling through work zones. Until recently, the problem of worker injuries has received relatively less attention. Studies by the Laborers' Health and Safety Fund of North America (LHSFNA) reported that highway construction workers had high rates of fatal injuries compared to other construction workers and to all workers. Both the LHSFNA report and Pratt et al. (2001) reported that only one-third of worker fatalities in work zones were attributable to workers being struck by road users entering the work space. The remaining two-thirds occurred when pedestrian workers were struck by construction vehicles or equipment or when vehicle or equipment operators were killed in vehicle-related incidents. Pratt et al. (2001) found that backing equipment, particularly dump trucks, accounted for half the fatalities of pedestrian workers in work zones.

Principles of an Internal Traffic Control Plan

The movement of workers and equipment within the work space should be planned in a manner similar to the way the TTC plan guides road users through a work zone. Thus, the ITCP was proposed by Graham-Migletz Enterprises, Inc., as a way to prevent worker injuries and fatalities during the LHSFNA study.

The purpose of this paper is to detail how an ITCP for paving operations is developed and used. The research described was completed under contract 200-2002-00596 “Pavement Operation Internal Traffic Control Plans” for the National Institute for Occupational Safety and Health (NIOSH). The ITCPs used as examples in this guide were developed for two paving operations observed in Tucson and Casa Grande, Arizona, in December 2002 and January 2003, respectively. The complete development guide and ITCP plans for two PCC paving projects were also prepared under this contract.

The development of ITCPs for paving operations was based on the following principles of safe construction traffic control, developed by Graham and Migletz (1997):

- Reduce the need to back up equipment
- Limit access points to work zones
- Establish pedestrian-free areas where possible
- Establish work zone layouts commensurate with type of equipment
- Provide signs within the work zone to give guidance to pedestrians, equipment, and trucks
- Use FAA/CG principles where applicable
- Design buffer spaces to protect pedestrians from errant vehicles or work zone equipment

A model plan for asphalt paving under traffic developed in earlier research is shown in Figure 1. This model plan also included safety points for paving operations and listing of personnel and equipment involved in the paving operation.

The NIOSH project specified that only freeway projects where traffic was separated from the paving operation either by a median or by temporary traffic barriers be observed. In this type of operation, the access and egress of trucks delivering asphalt to the job site is also included as part of the ITCP.

Responsibility for Developing the ITCP

The following criteria are necessary for those developing the ITCP:

1. The ITCP is developed by one or more members of the contractor’s staff and should be part of the project’s safety plan. It should be prepared after the contract is awarded but prior to the start of construction. A typical organizational chart of a paving contractor is shown in Figure 2.
2. The safety officer, if qualified, should be in charge of the development of the ITCP.
3. This officer should meet the OSHA requirements of a “competent person.”

Section 29 CFR1926.32 defines a “competent person” as one who is capable of identifying existing and predictable hazards in the surroundings or working conditions that are unsanitary, hazardous, or dangerous to employees, and who has authorization to take prompt corrective measures to eliminate them. The competent person should have sufficient experience and training to recognize and eliminate safety violations and other hazardous situations, as failure to observe safety standards and other safe work practices could result in serious injury or death.

A competent person is needed throughout a project's duration and should have a role in the development and monitoring of the ITCP. A competent person who can spot safety hazards and make changes to the ITCP, if needed, should be on-site during all paving operations. If the safety officer is not an engineer, s/he will need to work with an engineer to develop the ITCP. The engineer should be aware of safe traffic control practices and meet the requirements of a “knowledgeable” person as stated in the MUTCD.

The MUTCD recommends that any changes in the temporary traffic control plan should be approved by an official knowledgeable (i.e., trained and/or certified) in proper temporary traffic control practices. The on-site person should be knowledgeable in traffic control and paving practices and safety, or if there are two people, they should work together to develop and modify the ITCP.

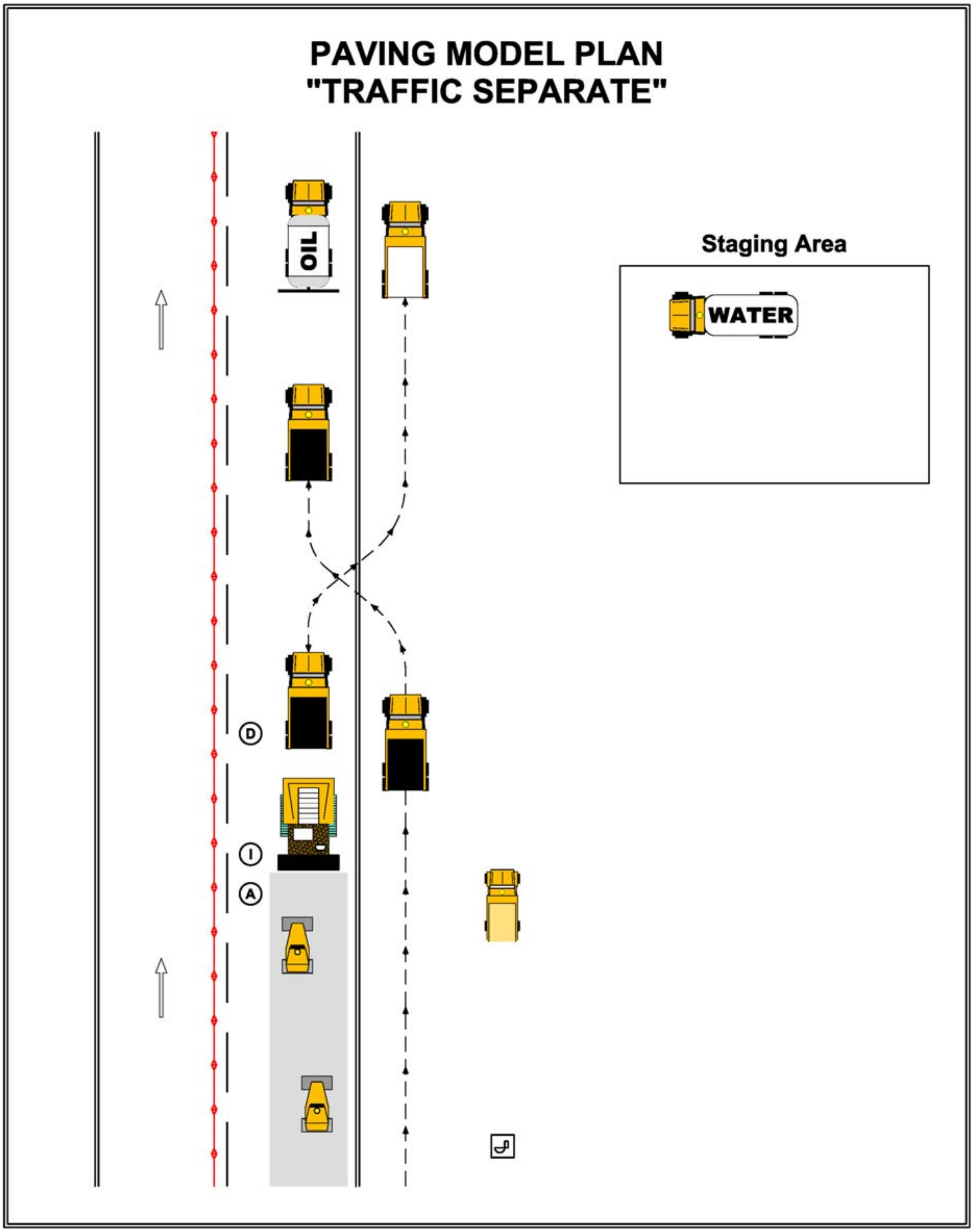

Figure 1. Paving model plan

The remainder of this paper will explain the components of the ITCP, the preparation of the ITCP for a specific site, and the use of the ITCP during paving operations.

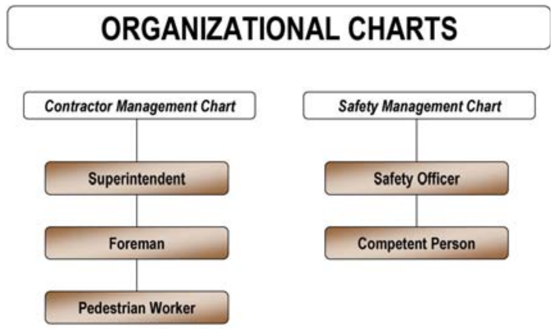

Figure 2. Typical organizational chart for a paving contractor

COMPONENTS OF AN INTERNAL TRAFFIC CONTROL PLAN

TTC plans consist of three basic components: the traffic control layout or diagram, a legend explaining symbols used in the diagram, and notes explaining portions of the diagram. The components of an ITCP are the same as for a TTC plan, but the specifics of each part vary from those of TTC plans.

ITCP Diagrams

The heart of the ITCP is the diagram showing the layout of the work space and the movement of personnel and vehicles within the work space. Since the ITCP will include the access points to the work space, it will also show some parts of the overall work zone. However, there is no need to show all of the work zone and temporary traffic control devices because the TTC plan will cover the entire work zone.

A model plan (similar to typical applications) for a paving operation with traffic separated from the work space by a temporary barrier is shown in Figure 1. An ITCP diagram may be the model plan, a modified model plan, or a separate site-specific plan showing the actual work space. While the diagram does not have to be to scale, it should show critical dimensions related to the injury reduction measures. For example, a 50-foot minimum distance required between the paver and the first roller is shown in the ITCP. The ITCP diagram may be shown on 8.5 x 11-inch sheets or larger, up to plan-size sheets if required. In some cases, a site diagram may be required with the ITCP diagram covering a portion of the site; however, most plan sets will include the site diagram.

Legend

The legend explains the symbols used on the ITCP diagram. A legend used for a paving ITCP is shown in Figure 3. Standard symbols are based on those used in the MUTCD. However, additional details on classes of personnel and vehicle types are needed in developing an ITCP for a paving operation.

ITCP Notes

The ITCP notes contain safety points, injury reduction measures, site-specific provisions, and duties of various contractor personnel. Safety points include pedestrian-free zones and buffer areas for vehicles such as rollers. Duties of the safety officer, plant operator, pedestrian workers, and truck drivers in safety terms are specified. Injury reduction measures specify when project safety meetings should be held, use of the ITCP, communication needs, coordination of dump truck arrivals and departures, and reference to general safety requirements such as 29 CFR. The ITCP notes for Site 1 included provisions for communication between workers, spotters for backing trucks, and site speed limits.

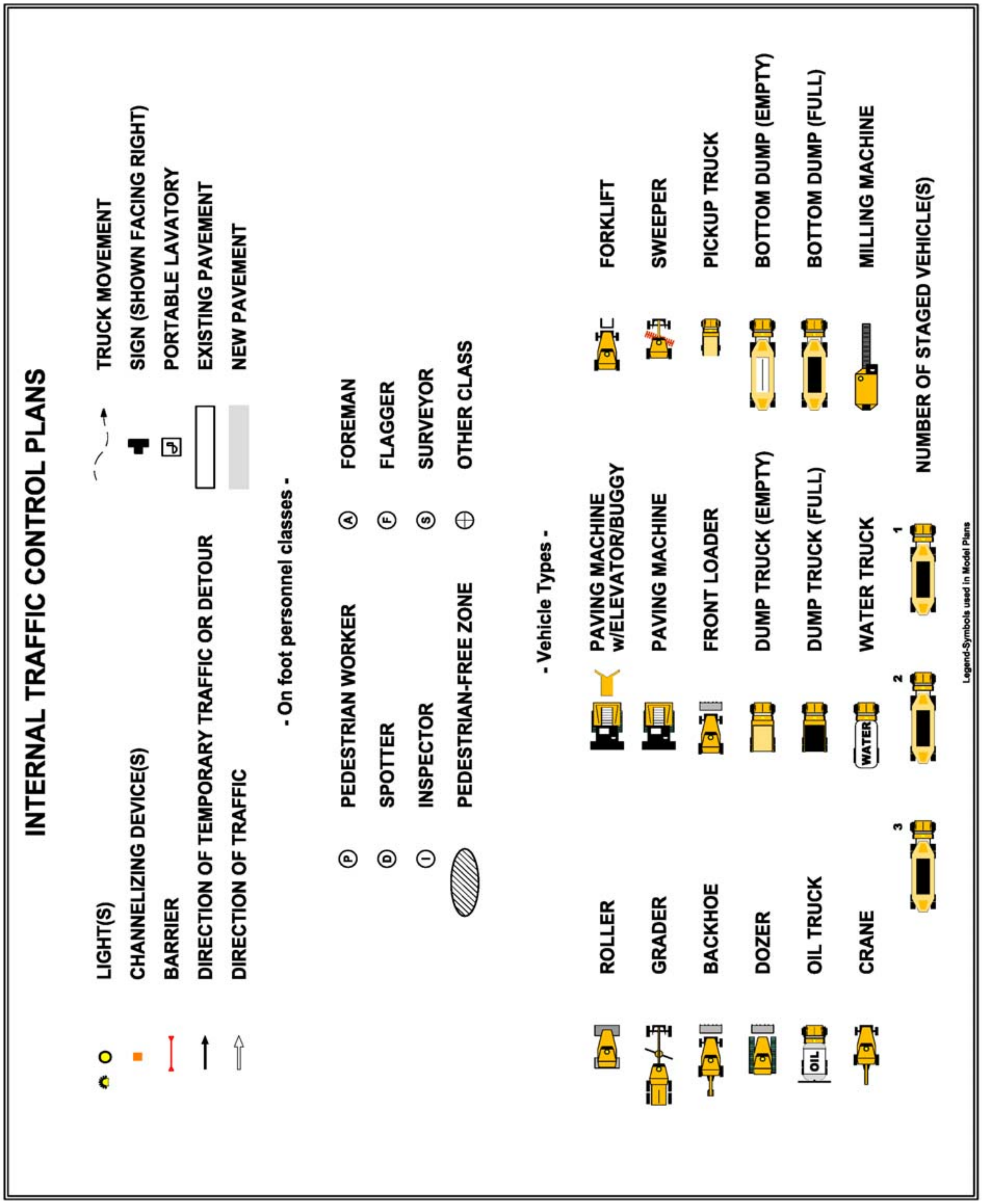

Figure 3. Internal traffic control plan legend

ITCP PREPARATION

The ITCP is part of the contract documents for a construction project. In most cases, it will be prepared by contractor personnel after the contract is awarded. The ITCP is a map of how the contractor chooses to complete the construction project; therefore, it must be done after the contract is awarded. (The model plan for some of the tasks involved in the project may be included in the PS and E package).

A process for developing an ITCP using principles of safe construction traffic control is detailed in the development guide and summarized here. The ITCP is then utilized during the project to reduce worker injuries and fatalities. Application of the ITCP is also discussed later in this paper.

The following outline shows six steps in the preparation of an ITCP. The ITCP must build on the information in the TTC plan and other contract documents. Site-specific ITCPs are completed for the phases of construction that are expected to be the most hazardous due to large numbers of pedestrian workers and their interaction with trucks and other equipment. For paving projects, this will generally be the paving phase, which requires a number of pedestrian workers to work near the dump trucks bringing asphalt to the paving machine.

Each of the steps in preparing an ITCP is discussed below. Examples of an ITCP are taken from the two asphalt paving projects studied.

Step 1: Review Contract Documents and Model Plans

The contract document most relevant to the ITCP preparation is the TTC plan. Also, a plan and cross-section of the site and the sequence of construction is important to review. Model plans are consulted to determine the basic layout of the paving operation.

A work site plan and cross section of the first example site is shown in Figure 4. This site was an urban four-lane freeway between two interchanges. The median of the freeway was to be cleared and new paving would add a travel lane and shoulder to each direction of travel. Other work included milling and overlay of the existing paved section and interchange improvements at Ina Road. Because of heavy traffic volumes, it was expected that some of the work would have to be completed at night. The paving of existing lanes was similar to the traffic-adjacent paving model plan and the paving of the new lanes in the median was similar to the model plan for paving with traffic separate (see Figure 1).

Step 2: Determine Construction Sequence and Choose Phases Having Site-Specific ITCPs

In this step, the personnel and equipment required for each phase of construction are listed, and the phases considered potentially hazardous are chosen for site-specific ITCPs. The sequence of construction, for example, in Site 1 was the following:

- Phase 1: Clear median and set concrete barrier
- Phase 2: Mill existing lanes and repave
- Phase 3: Add fill to median for subbase for new traffic lanes
- Phase 4: Pave new lanes in median

The equipment and personnel required for Phase 4 are shown in Table 1. This phase involved the most equipment and personnel and would be expected to be the most hazardous phase of this project. A site-specific ITCP was drawn for Phase 4.

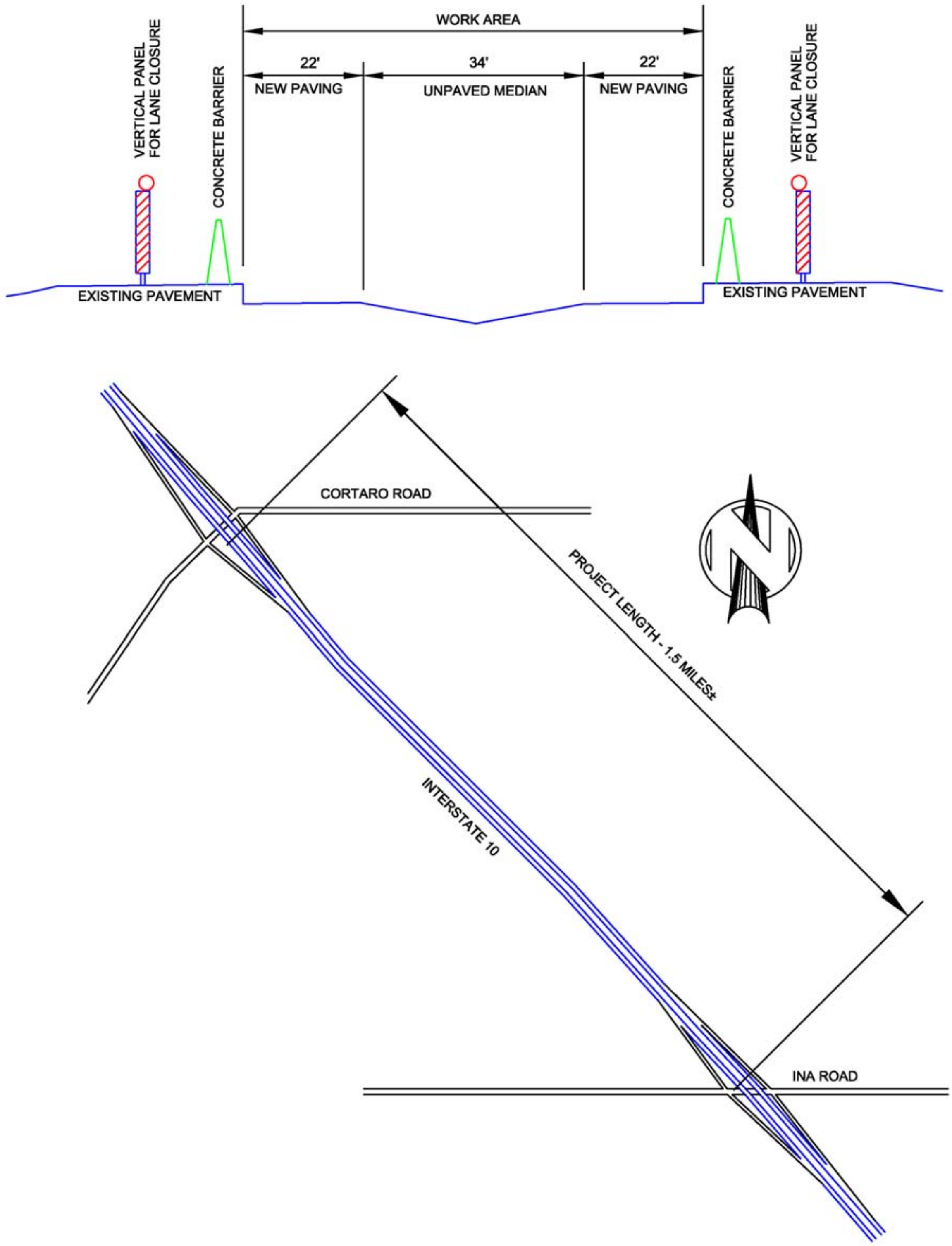

Figure 4. Site plan

Table 1. Equipment and personnel list for Site 2

Personnel	Equipment
Supervisor (1)	Oil (boot) truck (1)
Boot truck operator (1)	Paving machine (1)
Truck driver (8-10)	Bottom (belly) dumps (8-10)
Paving machine operator (1)	Backhoe (1)
Screed operator (1)	Loader (1)
Laborer/raker (2)	Vibratory roller (2)
Spotter/truck guide (2)	Water truck (1)
Roller operator (2)	Sweeper (1)
Mechanic (1)	Mechanic truck (2)
ADOT inspector (1-2)	Miscellaneous pickup truck (6)
Materials inspector (1)	

Step 3: Draw the Basic Work Area Layout

In this step, the configuration of the work area is drawn. The drawing does not need to be scaled, but it should be of sufficient size to allow the addition of personnel and equipment paths to the ITCP. Access and egress points for dump trucks will be shown on the ITCP; therefore, the existing traffic lanes will also be shown in most cases. In many circumstances, the basic layout can be taken from the TTC plan for the phase being shown.

The basic layout for example site 1 included the old median area where new lanes are to be added. As shown in Figure 4 the old median is divided into three areas, including a westbound travel lane and shoulder, a new median, and an eastbound travel lane and shoulder. The provision for concrete barriers was shown in the TTC plan and is therefore part of the basic layout of the ITCP.

Step 4: Plot Pedestrian and Vehicle Paths

The pivotal step of the ITCP development is to plot where pedestrians will normally be located, the types of equipment in the work area, and the path for each piece of equipment in the work area.

For a paving operation, the main activity will be around the paving machine. Dump trucks move into the work area to the front of the paver to deliver asphalt and then exit the work area. Pedestrian workers led by a foreman will be stationed near the paver; a spotter should be guiding trucks and directing the windrowing of asphalt in front of the paver. Other vehicles and pedestrian workers are stationed at various points relative to the paver. For example, rollers will work the new pavement mat behind the paver, and inspectors will move to the paver or mat behind the paver to sample the paving material.

The paths of vehicle movement should be planned in line with the principles of safe construction traffic control. Long backing maneuvers for dump trucks should be avoided and points of access and egress of trucks moving to the paver should be controlled. Pedestrians should be located as far as possible from vehicle paths, and parking, toilet, and break areas should be staged away from the principal conflict points involved with the paving rollers and dump trucks.

Vehicles and pedestrian workers at site 1 paving of new lanes are shown in Figure 5. Note that truck access points are controlled by openings in the barrier along the new travel lanes. An existing travel lane is closed in each direction to allow the trucks to exit and reenter the traffic stream. The truck access points were chosen to minimize the length that trucks traveled in the work area and to minimize the length of round trip from the asphalt plant to the work area. An alternate scheme would be to have trucks exit at the same point as they entered, but this would increase the truck travel in the work area and could cause conflicts between entering and exiting trucks. At site 1, bottom dump trucks were utilized and an elevator was used on the paver. These pieces of equipment allowed material to be windrowed in front of the paver rather than trucks backing up to the paver hopper.

Since this phase of construction was done at night, Figure 5 shows the location of light stations needed to ensure acceptable light levels throughout the work area. Pedestrian free zones should be added when pedestrian workers are in great danger, such as behind backing trucks. Buffer spaces between pedestrians and equipment should be provided. For example, see the 50-foot minimum buffer between the paver and rollers at site 1 in Figure 5.

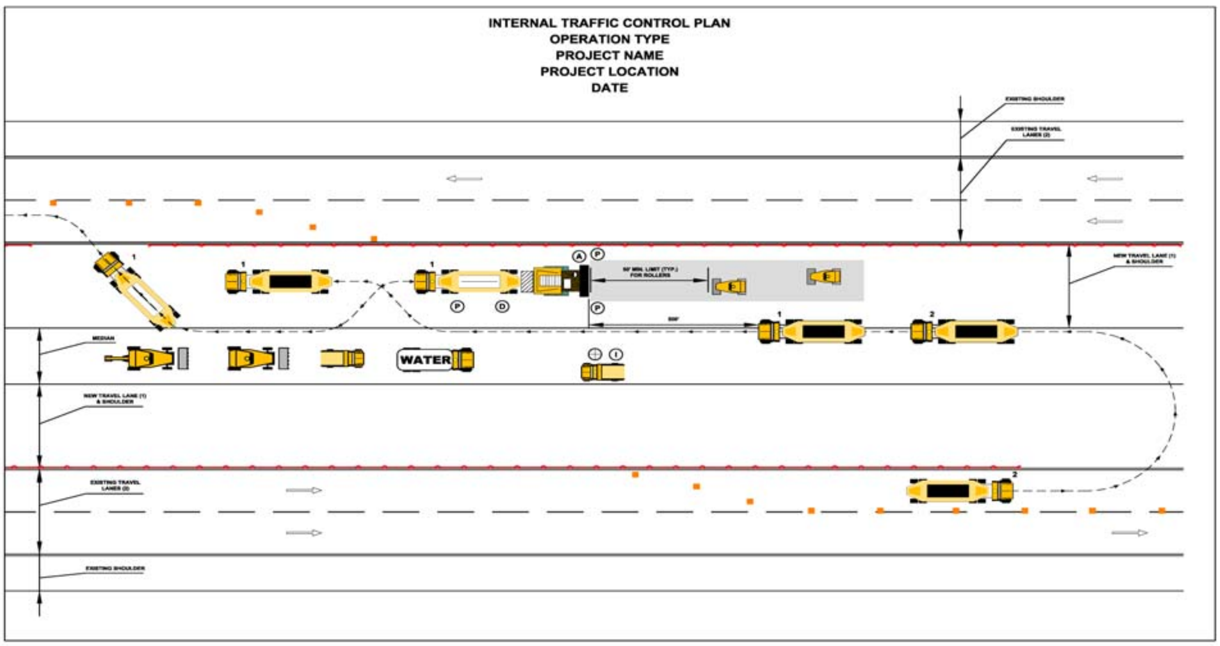

Figure 5. ITCP with all remaining features

Step 5: Locate Utilities, Storage, and Staging Areas

To complete the ITCP diagram, the location of utilities, storage areas for equipment and materials, and staging areas within the work area are added to the diagram. Utilities that would impact the work area operations, such as power lines or catch basins, should be shown on the ITCP. Special traffic control, such as overhead power lines awareness markers, should also be added to the diagram.

Equipment needed periodically should be stored in a safe area. For example, at site 1 the water truck, backhoe, and front-end loader were stored in the median until needed (see Figure 5). At site 2, the equipment storage and restroom facility were stored near the right-of-way line of the interchange area being repaved (see Figure 6).

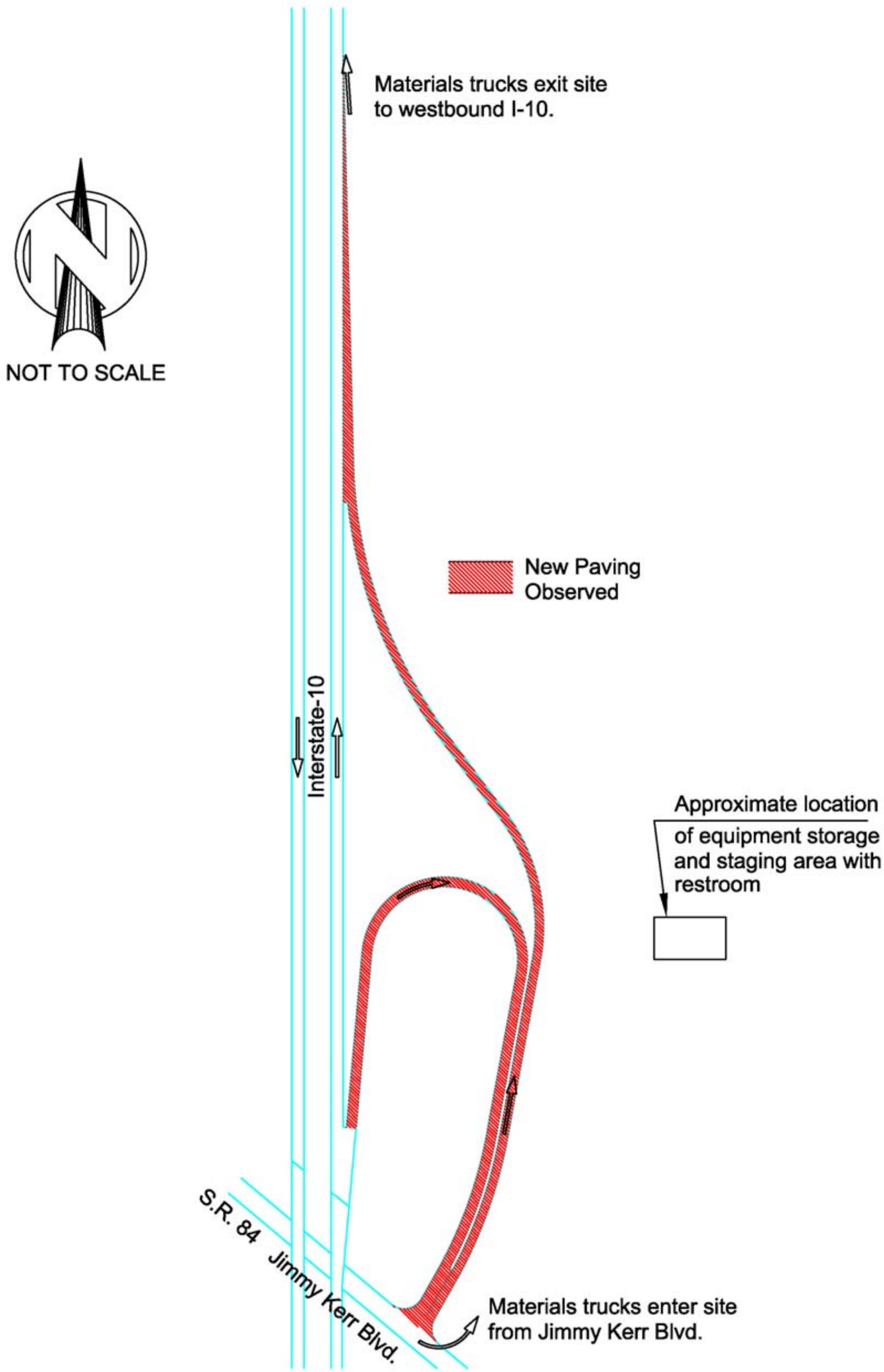

Figure 6. Internal traffic control plan with equipment storage and restroom facility

Step 6: Prepare ITCP Notes

The last step in the preparation of the ITCP is to write notes that explain the diagram and specify the duties of various personnel in the work area. The notes can contain general conditions that are common to most paving operations and specific conditions that are applicable for only that site and phase of construction. The notes for site 1 included references to lighting requirements and use of fog lights for trucks, since this phase of work was done at night. The notes may also reference other safety requirements such as CFR29. The notes of the ITCP should supplement other contract documents but should not conflict with or supercede other contract provisions.

Safety points can include a description of pedestrian-free areas and required buffer areas for vehicles, such as the minimum distance between the paver and rollers.

APPLICATION OF ITCPS

While preparation of an ITCP may benefit a contractor in planning for a safe project, the main benefit of the ITCP will be during pre-construction and project safety meetings. Use of the ITCP in daily safety meetings is necessary to make all project personnel aware of how to perform their jobs safely.

At pre-construction meetings, the ITCP can be used to illustrate the safety plan and the contractor's approach to worker safety. The plan is also useful to assure the contracting agency that worker safety is being considered and planned for in a manner similar to that of road users moving through the work zone.

During the project, the plans are useful to discuss construction strategy and daily changes that are part of any paving project. The plan should be distributed to all personnel working on the project, including inspectors and subcontractors such as independent truckers. The safety officer and competent person on the project should make changes in the ITCP as conditions warrant during the project.

Using ITCPs at Pre-Construction Meetings

The ITCP should be a discussion item at the pre-construction meeting along with the overall safety plan. Common worker injury and fatalities for paving operations should be discussed, along with injury reduction measures contained in the ITCP Notes.

Critical parts of the ITCP, such as truck access points and staging areas, should be discussed and approved by the contracting agency. Protection of vehicle operators from hazards, such as overhead power lines or steep slopes, should be discussed. A plan for communicating the provisions of the ITCP and the overall safety plan to each worker should be discussed at the pre-construction meeting. Communication methods should include daily safety meetings and required attendance of subcontractors at ITCP briefings. Training for inspectors and tare collectors in safely performing their jobs should be provided utilizing the ITCP.

Using ITCPs During Construction

The safety officer and designated competent person for each shift should use the ITCP to illustrate the safety plan for the paving operation. If changes to the ITCP are necessary as the project progresses, then the competent person should be in charge of getting the changes approved and communicating the changed plan to all project personnel.

The competent person should also be responsible for warning pedestrian workers or vehicle operators of violations of the ITCP. These violations could include workers out of position or working in pedestrian-free zones, or truck drivers operating at speeds above the designated site speed limit. The safety officer or competent person should take photographs or video of the paving operation to check compliance with the ITCP or to check areas where changes are necessary.

Truck drivers should be briefed on how to access the project site, the path to follow to the paver, where to stop for staging, and how the spotter will instruct them once they are near the paver. The plant operator will also be briefed on holding trucks at the plant site to control the number of trucks on the site at any time. Truck drivers should also be briefed on procedures for leaving the project area and re-entering the traffic stream.

A method for handling visitors to the project should also be discussed. Visitors should park at an off-site staging area and then be briefed on the ITCP. If visitors drive to the site, they should access at a known point and park and walk in approved areas.

At the conclusion of each phase of construction, the ITCP should be critiqued and critical points of upcoming phases should be discussed.

OBSERVATION OF PAVING OPERATIONS

Two asphalt paving sites and two PCC paving sites were observed. ITCPs were prepared for these sites, but were not implemented. The results of the observations and recommendations of the research team are contained in a number of project reports.

The paving operations were videotaped with two cameras. One of the cameras was stationary and filmed the overall operation of each site. The second camera was a hand-held digital camera that was used to film from the ground level and also filmed from inside construction vehicles and filmed interviews with workers and vehicle operators. Sites were observed for approximately 40 hours at each and site 1 included night observation.

Observation Results

1. At most sites, safety officers were only on-site once per day for 15-30 minutes.
2. No accidents or near accidents were observed.
3. At one of the PCCP sites, truck drivers were confused about where to go and where to start backing to the paver. At an asphalt site, truck drivers were not instructed on how to enter and exit the work space.
4. A loader was operated between the paver and backing trucks at one PCCP site. The movement of the loader was seen as a potential conflict, and if the loader struck the paver an unbelted rebar setter on the paver could have been thrown into the paver’s auger.
5. At the first asphalt paving site, truck drivers tried to complete the cycle from the plant to the paving site as quickly as possible. No desirable speeds were required or mentioned for the public travel portion nor the work space portion of the trip. Other vehicles were observed operating at relatively high speeds in the work space.
6. At all sites, if paving operations were halted or stopped, several trucks backed up on-site, creating additional unnecessary hazards.
7. While all of the operation had designated spotters, there was no method of communication between trucks and spotters other than hand signals.
8. Trucks and pavers had blind spots where a person could not be seen.
9. At one site, a backhoe placed paving material in paving gaps. The backhoe moved in and out of the area between trucks and the dump area. No one directed the backhoe’s movements and workers were not warned of his movements.

10. For night paving operations, sufficient light was available near the paver; however, there was little light available in other areas, such as where inspectors sampled the placed asphalt mat.
11. One of the most serious violations of safety procedures was observed when an employee entered the front of the paving machine while the concrete auger was still rotating. The auger should be de-energized as required in lock-out, tag-out procedures.

CONCLUSIONS

This research yielded the following conclusions:

1. The ITCP is a graphical method to inform vehicle operators and pedestrian workers of hazards inside the work area. The provision of an ITCP would have reduced hazards and observed conflicts at all four paving sites observed.
2. A competent person was not available during all paving operations. The safety officer was either absent or visited the site for a very short time.
3. Safety plans were generic and not specific to any of the sites.
4. Truck drivers were often confused about how to access the site and most could not communicate with spotters, forepersons, or plant operators.
5. At one site, material trucks and other service vehicles operated at relatively high speeds, even at night with little illumination.
6. There was no reliable method of controlling the rate of truck arrivals at the work site.
7. Lock-out, tag-out procedures were not always observed.

RECOMMENDATIONS

The following are the recommendations based on this research:

1. Ensure that there is a detailed safety plan that meets 29 CFR requirements with specific documented training for all employees. Require a competent person who meets 29 CFR standards to be on site during the work.
2. Daily safety meetings should be conducted with all personnel, including truck drivers, inspectors, etc. The ITCP should be discussed, along with updates in operations.
3. Spotters should have direct communication with the material truck drivers through the use of radios or other communication devices.
4. A crew member, most logically the screed operator, should be designated to communicate with the rest of the crew when the paving machine is backing up.
5. A crew member should be designated to communicate with the rest of the crew when other equipment is operating in the work area.
6. Truck drivers need instructions on how to enter and exit the work zone and how to maneuver within the work zone. This could be accomplished by the designated safety officer going over the ITCP with them prior to the work commencing.
7. All other equipment operators or passenger truck drivers on site should also be made familiar with the ITCP so that they can more safely and efficiently operate within the work area.
8. For night work, light standards need to be placed so that lighting is consistent along the work site.
9. All safety vests should be checked for reflectivity for night operations.
10. Establish desirable operating speeds for vehicles on public roads and in the work space.
11. Seat belts should be required for all vehicles and a seat belt or harness should be required for the rebar setter on PCC paving machines.
12. Establish a specific lock-out, tag-out program for use when servicing the machinery.

SUMMARY

Safety of pedestrian workers and construction vehicle operators can be enhanced by a carefully prepared internal traffic control plan administered by a competent person at the paving site. Time and effort spent in preparing and using an ITCP should lower the rate of occupational injuries and fatalities experienced by construction personnel at asphalt paving projects.

The full development guide, while aimed specifically at asphalt paving projects, can also be useful in preparation of ITCPs for other common types of construction. It is hoped that, in the future, additional model plans will be prepared that can aid in development of ITCPs for other construction operations.

ACKNOWLEDGMENTS

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Sensitivity Analysis of Rigid Pavement Systems Using Mechanistic-Empirical Pavement Design Guide

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ABSTRACT

Pavement design procedures available in the literature do not fully take advantage of mechanistic design concepts, and as a result, heavily rely on empirical approaches. Because of their heavy dependence on empirical procedures, the existing rigid pavement design methodologies do not capture the actual behavior of Portland Cement Concrete (PCC) pavements. However, reliance on empirical solutions can be reduced by introducing mechanistic–empirical methods, now adopted in the newly released Mechanistic-Empirical Pavement Design Guide (MEPDG). This new design procedure incorporates a wide range of input parameters associated with the mechanics of rigid pavements. A study was undertaken to compare the sensitivity of these various input parameters on the performance of concrete pavements. Two Jointed Plain Concrete Pavement (JPCP) sites were selected in Iowa. These two sections are also part of the Long Term Pavement Performance (LTPP) program, where a long history of pavement performance data exists. Data obtained from the Iowa Department of Transportation (Iowa DOT) Pavement Management Information System (PMIS) and LTPP database were used to form two standard pavement sections for the comprehensive sensitivity analyses. The sensitivity analyses were conducted using the MEPDG software to study the effects of design input parameters on pavement performance, specifically faulting, transverse cracking, and smoothness. Based on the sensitivity results, the rigid pavement input parameters were ranked and categorized from most sensitive to insensitive to help pavement design engineers to identify the level of importance for each input parameter. The curl/warp effective temperature difference (built-in curling and warping of the slabs) and PCC thermal properties are found to be the most sensitive input parameters. Based on the comprehensive sensitivity analyses, the idea of developing an expert system is introduced to help the designer identify the input parameters that can be modified to satisfy the predetermined pavement performance criteria.

Keywords: jointed plain concrete pavements—mechanistic-empirical pavement design guide (MEPDG)—pavement performance—rigid pavement design—sensitivity analysis

INTRODUCTION

Background

The historical development of Mechanistic-Empirical (M-E) pavement design procedures in the American Association of State Highway and Transportation Officials (AASHTO) guides goes back to the 1986. In the 1986 AASHTO guide for pavement structures, M-E design procedure was firstly defined as the calibration of mechanistic models with observations of performance, i.e., empirical correlations. It was also stated that in a multi-layered pavement system, analytical methods were the numerical calculations of the pavement responses when subjected to external loads or the effects of temperature or moisture. Then, assuming that pavements can be modeled as a multi-layered elastic or visco-elastic structure on an elastic or visco-elastic foundation, the stress, strain, or deflection could be calculated at any point within or below the pavement structure. Mechanistic procedures are referred to for their ability to translate the analytical calculations of the pavement responses to physical distress such as cracking or rutting (pavement performance). However, pavement performance is subject to a number of factors that cannot be exactly modeled by mechanistic methods. It is, therefore, necessary to incorporate empirical pavement performance models with mechanistic models. Thus, in the 1986 AASHTO guide, the procedure is defined conceptually as a mechanistic-empirical pavement design procedure (AASHTO 1972).

The AASHTO pavement design guides (AASHTO 1972, 1986, 1993) used empirical methods, which are valid for specific environmental, material, and loading conditions. In order to develop a design procedure without these limitations, the development of M-E design procedures was promoted by the AASHTO Joint Task Force on Pavements (JTFP). AASHTO JTFP recommended the research should be initiated for the later versions of the AASHTO design guides. The National Cooperative Highway Research Project (NCHRP) Project 1-26 was the first NCHRP project to be sponsored (NCHRP 1990a, 1990b, 1992a, 1992b). Later, the second phase of NCHRP 1-26 was completed in 1992 with its two volumes of final reports detailing the guidelines for the data input stage of the procedures (Masada et al. 2004) Finally, at the conclusion of a workshop held in March 1996 in Irvine, California, JTFP concluded a long-term project for the development of a design guide based as fully as possible on mechanistic principles. This guide is titled *The NCHRP Project 1-37A mechanistic-empirical design guide for design of new and rehabilitated pavement structures* (NCHRP 2004).

Project Scope

The main focus of this paper is to identify the sensitivity of input parameters needed for designing jointed plain concrete pavements used in the mechanistic-empirical pavement design guide. This paper identifies input parameters ranging from “most sensitive” to “insensitive” for three critical rigid pavement performance measures of faulting, transverse cracking, and smoothness.

SENSITIVITY ANALYSIS USING MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE

Collecting MEPDG data

The very first part of this project involved extensive data collection. Two rigid pavement sections were selected from the Iowa Department of Transportation (Iowa DOT) Pavement Management Information System (PMIS) which were also part of the Long Term Pavement Performance (LTPP) program. A history of pavement deflection testing, material testing, traffic, and other related data were available in the LTPP database. These two sections were named as PCC-1 and PCC-2.

PCC-1, located on US-218 near Johnson County, Iowa, was constructed in 1983. This section of US-218 is located in the wet-freeze environmental region. This area has a freezing index of 466.88 and receives 930.58 mm of rainfall annually. The latitude and longitudes are given as 41.57 degrees and 91.55 degrees, respectively. The pavement is a 9.6-inch JPCP with 15 ft joints and Class II type aggregates. The slab rests on a 4-inch Class A sub-base course. The subgrade for this site consists of AASHTO A-7-6 material and it is noted that there exists silty clay of Loess material with some glacial till treatments.

PCC-2, located on US-20 near Hamilton County, Iowa, was constructed in 1968. The test section was westbound in the north central LTPP SHRP region, and designated between 149.5 and 153.47 miles of US-20. This section of US-20 is also located in the wet-freeze environmental region. This area has a freezing index of 763.69 and receives 861.74 mm (34 in) of rainfall annually. The latitude and longitudes are 42.46 degrees and 93.59 degrees, respectively. The pavement is a 10-inch JPCP with 15 ft joints. The slab rests on a 4-inch granular sub-base course. The subgrade is made of AASHTO A-6 (7) to A-6 (10) material, a glacial till soil.

Generating a Representative Pavement Section for MEPDG

In order to conduct mechanistic-empirical analyses using the MEPRG, a pavement section representative of the Iowa highway system was generated. Sensitivity analyses were carried out on this representative pavement section to examine the effect of each input or inputs groups of two on pavement performance by using the MEPDG software and the design inputs. The standard input parameters for the representative pavement section were determined using the inputs from two PCC sections and were adjusted considering Iowa conditions.

Sensitivity Analysis Using MEPDG

The representative pavement section was analyzed using the MEPDG software by varying one input parameter within its ranges while holding other parameters constant in the model. Several analyses were carried out. Pavement distresses throughout the design life for each input file were plotted. The goal of these analyses was to determine the individual effects of each input parameter on the critical pavement performance using the MEPDG software. It should be noted that the climate input variable in the MEPDG sensitivity analyses reflects Iowa’s climate data, in or around Iowa. The chosen weather stations are located in Figure 1.

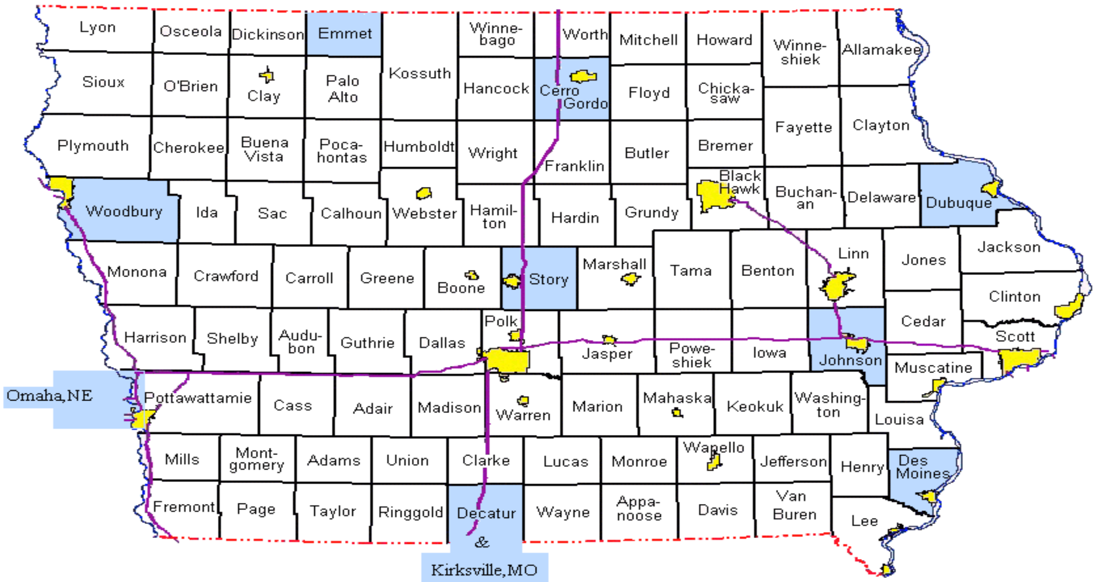

Figure 1. The selected climatic locations for sensitivity analysis

The second step was to find the interaction of input parameters between each other and their interaction with pavement performance values. The results of the first test (varying one variable) revealed that the standard input parameters established for the representative pavement section were corresponding beyond the capacity of pavement performance. Therefore, in some cases the standard input variables were modified to reflect the capacity of pavement performance.

For each input variable, range was defined according to their maximum and minimum values. Moreover, additional values in between minimum and maximum values were considered in order to observe the trend of their impact on pavement performance. Several hundreds of graphs were created using the results of MEPDG software (Guclu 2005). The different curl/warp effective temperature difference input vs. pavement responses graphs are shown as examples of those graphs in Figures 2 through 4.

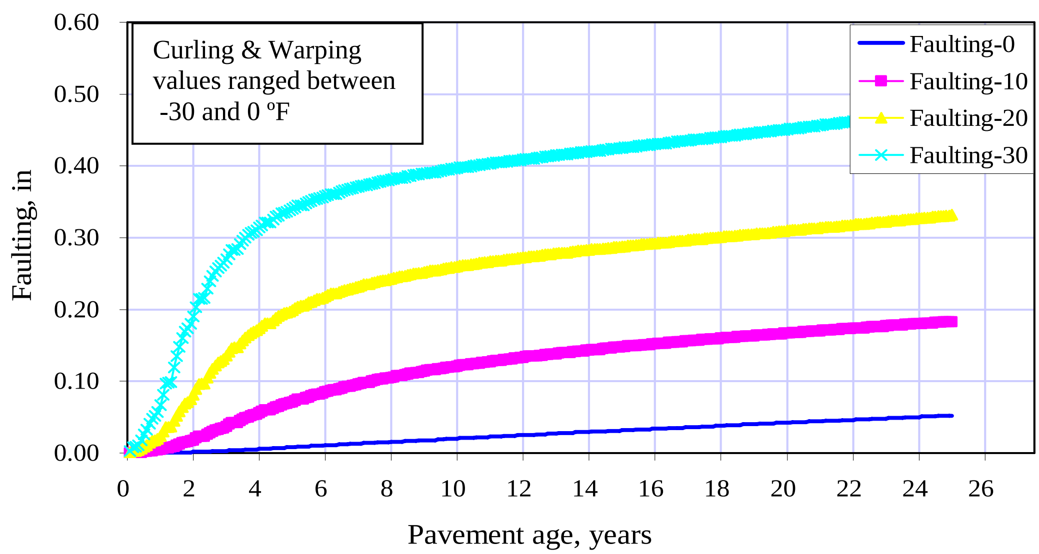

Figure 2. Faulting for different curl/warp effective temperature difference (built-in)

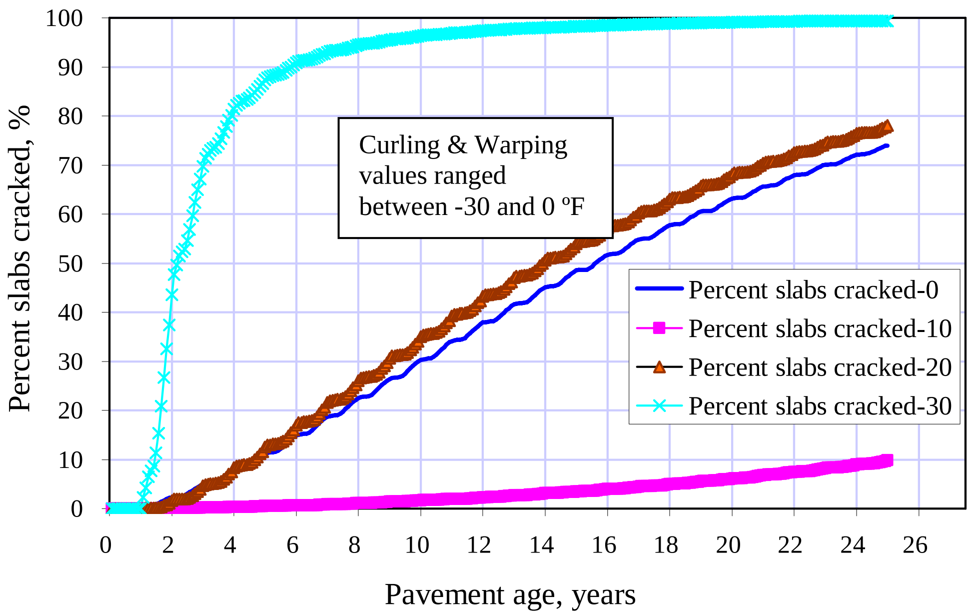

Figure 3. Cracking for different curl/warp effective temperature difference (built-in)

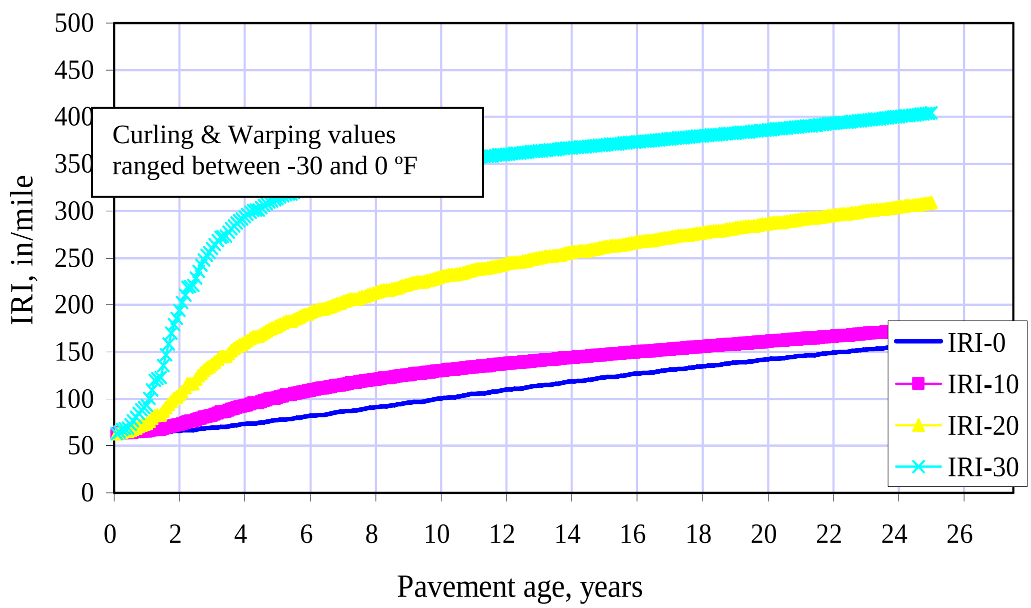

Figure 4. IRI for different curl/warp effective temperature difference (built-in)

The obtained sensitivity graphs were visually inspected. The evaluation was made according to the pavement performance value and the amount of change in the pavement performance value due to changing input variable. The results obtained were sensitive in different scales, so the scales from insensitive to the most sensitive were developed for a better understanding of the effects of each input parameters.

The sensitivity values extracted from all of the sensitivity analyses are summarized according to their sensitivities for each pavement response and presented in Tables 1-3. In the tables, the sensitivities of inputs are given under three columns–extreme sensitivity, sensitive to very sensitive, and low sensitive to insensitive for each pavement performance models of faulting, transverse cracking, and smoothness.

Table 1. Summary of sensitivity level of input parameters for faulting of JPCP

<table>
<tr><th rowspan="2">Performance Models</th><th colspan="3">Inputs</th></tr>
<tr><th>Extreme Sensitivity</th><th>Sensitive to Very Sensitive</th><th>Low Sensitive to Insensitive</th></tr>
<tr><td>Faulting</td><td>• Curl/Warp Effective Temperature Difference
• Doweled Transverse Joints</td><td>• AADTT
• Mean Wheel Location
• Unbound Layer Modulus
• Cement Content
• Water/Cement Ratio
• Coefficient of Thermal Expansion
• Thermal Conductivity</td><td>• Sealant Type
• Dowel Diameter
• Dowel Spacing
• PCC-Base Interface
• Erodibility Index
• Traffic Wander
• Design Lane Width
• Infiltration of Surface Water
• Drainage Path Length
• Pavement Cross Slope
• Cement Type
• Aggregate Type
• PCC Set (Zero Stress) Temperature
• Ultimate Shrinkage at 40% R.H.
• Reversible Shrinkage
• Time to Develop 50% of Ultimate Shrinkage
• Curing Method
• Edge Support
• Surface Shortwave Absortivity
• Unit Weight
• Poisson’s Ratio
• Climate
• PCC Strength
• Joint Spacing</td></tr>
</table>

Table 2. Summary of sensitivity level of input parameters for transverse cracking of JPCP

Performance Models	Inputs		
	Extreme Sensitivity	**Sensitive to Very Sensitive**	**Low Sensitive to Insensitive**
Transverse Cracking	• Curl/Warp Effective Temperature Difference • PCC Thermal Properties (Coefficient of Thermal Expansion, Thermal Conductivity) • PCC Layer Thickness • PCC Strength Properties • Joint Spacing	• Edge Support • Mean Wheel Location • Unit Weight • Poisson's Ratio • Climate • Surface Shortwave Absortivity • AADTT	• Sealant Type • Dowel Diameter • Doweled Transverse Joints • Dowel Spacing • PCC-Base Interface • Erodibility Index • Traffic Wander • Design Lane Width • Infiltration of Surface Water • Drainage Path Length • Pavement Cross Slope • Cement Type • Cement Content • Water/Cement Ratio • Aggregate Type • PCC Set (Zero Stress) Temperature • Ultimate Shrinkage at 40% R.H. • Reversible Shrinkage • Time to Develop 50% of Ultimate Shrinkage • Curing Method • Unbound Layer Modulus • Heat Capacity

Table 3. Summary of sensitivity level of input parameters for smoothness of JPCP

Performance Models	Inputs		
	Extreme Sensitivity	**Sensitive to Very Sensitive**	**Low Sensitive to Insensitive**
Smoothness	• Curl/Warp Effective Temperature Difference • PCC Thermal Properties (Coefficient of Thermal Expansion, Thermal Conductivity)	• Doweled Transverse Joints • AADTT • Mean Wheel Location • Joint Spacing • PCC Layer Thickness • PCC Strength Properties • Poisson's Ratio • Surface Shortwave Absortivity • Unbound Layer Modulus • Cement Content • Water/Cement Ratio	• Sealant Type • Dowel Diameter • Dowel Spacing • PCC-Base Interface • Erodibility Index • Traffic Wander • Design Lane Width • Infiltration of Surface Water • Drainage Path Length • Pavement Cross Slope • Cement Type • Aggregate Type • PCC Set (Zero Stress) Temperature • Ultimate Shrinkage at 40% R.H. • Reversible Shrinkage • Time to Develop 50% of Ultimate Shrinkage • Curing Method • Edge Support • Climate • Unit Weight

SUMMARY AND CONCLUSIONS

A number of conclusions were drawn as a result of the sensitivity analyses.

Transverse cracking

- The extremely sensitive input parameters for transverse cracking are the following:
    - Curl/warp effective temperature difference (built-in)
    - Coefficient of thermal expansion
    - Thermal conductivity
    - PCC layer thickness
    - PCC strength properties
    - Joint spacing
- The sensitive to very sensitive input parameters for transverse cracking are the following:
    - Edge support
    - Mean wheel location (traffic wander)
    - Unit weight
    - Poisson's ratio
    - Climate
    - Surface shortwave absorptivity
    - Annual average daily truck traffic (AADTT)
- Other examined parameters are found as less sensitive to insensitive.

Faulting

- The extremely sensitive input parameters for faulting are the following:
    - Curl/warp effective temperature difference (built-in)
    - Doweled transverse joints (load transfer mechanism, doweled or un-doweled)
- The sensitive to very sensitive input parameters for faulting are the following:
    - Coefficient of thermal expansion
    - Thermal conductivity
    - Annual average daily truck traffic (AADTT)
    - Mean wheel location (traffic wander)
    - Unbound layer modulus
    - Cement content
    - Water to cement ratio
- Other examined parameters are found as less sensitive to insensitive.

Smoothness

- The extremely sensitive input parameters for smoothness are the following:
    - Curl/warp effective temperature difference
    - Coefficient of thermal expansion
    - Thermal conductivity
- The sensitive to very sensitive input parameters for smoothness are the following:
    - Annual average daily truck traffic (AADTT)
    - Doweled transverse joints (load transfer mechanism, doweled or un-doweled)
    - Mean wheel location (traffic wander)
    - Joint spacing

- PCC layer thickness
- PCC strength properties
- Poisson's ratio
- Surface shortwave absorptivity
- Unbound layer modulus
- Cement content
- Water to cement ratio

- Other examined parameters are found as less sensitive to insensitive.

Among the extremely sensitive and sensitive to very sensitive parameters, the pavement design engineer can only modify PCC layer thickness, doweled transverse joints, and joint spacing. PCC strength properties are also modifiable provided that pavement design specifications are met.

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Automated Extraction of Weather Variables from Camera Imagery

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ABSTRACT

Thousands of traffic and safety monitoring cameras are deployed all across the country and throughout the world to serve a wide range of uses, from monitoring building access to adjusting timing cycles of traffic lights at clogged intersections. Currently, these images are typically viewed on a wall of monitors in a traffic operations or security center where observers manually monitor potentially hazardous or congested conditions. However, the proliferation of camera imagery taxes the ability of the manual observer to track and respond to all incidents and the images contain a wealth of information that often goes unreported or undetected. Camera deployments continue to expand and the corresponding rapid increases in both the volume and complexity of camera imagery demand that automated algorithms be developed to condense the discernable information into a form that can be used operationally.

MIT Lincoln Laboratory, under funding from the Federal Highway Administration (FHWA), is investigating new techniques to extract weather and road condition parameters from standard traffic camera imagery. To date, work has focused on developing an algorithm to measure atmospheric visibility and proving the algorithm concept. The initial algorithm examines the natural edges within the image (e.g., the horizon, tree lines, roadways) and compares each image with a historical composite image. This comparison enables the system to determine the visibility in the direction of the sensor by detecting which edges are visible and which are not. A primary goal of the automated camera imagery feature extraction system is to ingest digital imagery with limited site-specific information, such as location, height, angle, and visual extent, thereby making the system easier for users to implement. Many challenges exist for providing a reliable automated visibility estimate under all conditions (e.g., camera blockage/movement, dirt/raindrops on lens) and the system attempts to compensate for these situations. This paper details the work to date on the visibility algorithm and defines a path for further development of the system.

Key words: camera— extraction—visibility—weather

INTRODUCTION

The first video surveillance cameras used for surveying traffic congestion were deployed in England in the mid-1970s and in the United States in the 1980s by the New York City Police Department. These early analog camera images were useful for general viewing, but the images were often too washed out to even determine general traffic conditions. The advent of digital camera technology, high-bandwidth wired and wireless communications, and dramatic reductions in camera costs have combined with increased funding of intelligent transportation systems (ITS) infrastructure and a heightened need for security surveillance of wide areas due to post-9/11 terrorist concerns to fuel an explosion in available camera assets. Today, thousands of traffic and safety monitoring cameras are deployed across the country. Video cameras are being employed for a wide range of uses, from monitoring building access to adjusting timing cycles of traffic lights at clogged intersections. Many of these traffic-related cameras are available on the web with near real-time access and are located at key traffic bottleneck or hazard points across the country. There are currently over 4,000 state/city DOT-owned and publicly available traffic cameras across the United States, and many state, county, and municipal centers are planning more installations (U.S. DOT 2004). Figure 1, based on a combination of web survey and U.S. DOT research (2004), shows current traffic camera deployment by state.

The DOT-owned cameras are located along major arterial roadways, key traffic intersections, remote intersections in mountain passes, and alongside Road Weather Information System (RWIS) sites. The cameras are used to monitor road conditions for snow and blowing snow; monitor and adjust, where possible, traffic flows; and verify RWIS observations and external road condition reporting. Currently, much of this monitoring is done manually. Images are typically viewed on a wall of monitors in a traffic operations or security center. Image processing applications for traffic management are mainly focused on arterial management to detect the presence of stopped vehicles at signalized intersections. Often the imagery is also displayable from a web-based map, allowing commuters and commercial vehicle operators to view current traffic and weather conditions. However, it is generally left to observers to monitor potentially hazardous or congested conditions and to notify the appropriate agency.

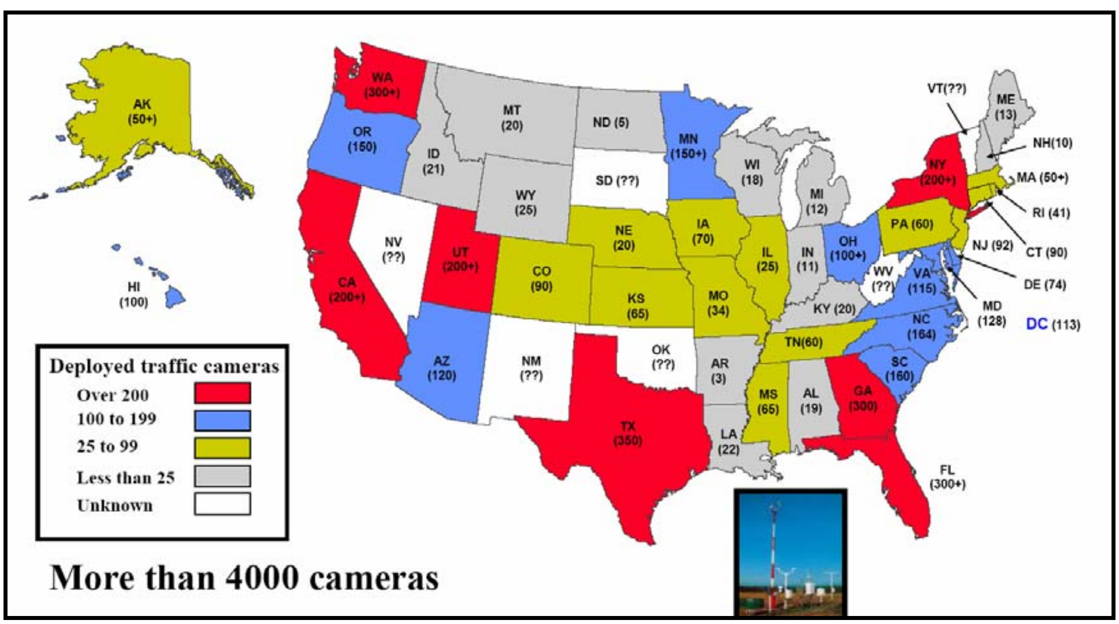

Figure 1. Survey of state DOT traffic cameras

PROBLEM STATEMENT

The proliferation of camera imagery taxes the ability of the manual observer to track and respond to all incidents. In addition, the images contain a wealth of information, including visibility, precipitation type, road surface conditions, that often goes unreported because these variables are not always critical to operations or because the variables go undetected by the observer. Camera deployments continue to expand, and the corresponding rapid increases in both the volume and complexity of camera imagery demand that automated algorithms be developed to condense the discernable information into a form that can be easily used operationally. Recently, a number of companies have stepped forward to examine ways for automated image processing to assist security and safety officials. Several companies offer automated license plate detection and reading for automated toll way and red light enforcement, while others analyze images for security breaches, and some have begun to use video for traffic incident and flow monitoring. However, the area of weather and road condition analysis from video imagery is relatively new. The Japanese meteorological institute has used image analysis to determine road conditions (Yamada 2001). Similar road condition studies using neural networks and infrared cameras have been performed by the Swedish National Road Administration (SNRA 2002). The University of Minnesota has performed visibility tests using fixed distance targets (Kwon 2004). However, most of these programs require either new hardware (e.g., infrared cameras and fixed sign placements) or extensive site surveys to determine object distances and land types.

MIT Lincoln Laboratory (MIT/LL), under previous funding from Department of Defense and under new funding from the Federal Highway Administration (FHWA), is investigating new techniques to extract weather and road condition parameters from standard traffic camera imagery without additional hardware, signage, or site surveys. This paper examines the work accomplished to date, including results from an early prototype algorithm, and discusses future efforts under the FHWA Clarus Initiative, discussed later, to develop the algorithm further.

Background

The initial work on camera imagery for weather sensing was performed for the U.S. Army as part of a program to gather real-time weather data on the battlefield to support ground operations. The Weather Web (WxWeb) program was part of the overall Smart Sensor Web program, which also had a component called Image Web. Image Web envisioned deploying digital cameras to use in a military setting to monitor movement of enemy forces. MIT/LL was tasked with evaluating the use of camera images for weather surveillance. As a first step, MIT/LL deployed two fixed digital cameras at different times to two field site locations. The cameras were co-located with meteorological sensors for measuring, temperature, dew point, pressure, wind speed and direction, and visibility. The primary focus of WxWeb was mountainous terrain and fog conditions; thus, initial algorithm development focused on visibility restrictions due to fog. From this initial work, a conceptual algorithm was created for automatically processing incoming images to derive a variety of weather variables (a patent is pending for the technique described below [MIT 2002]). The images and data collected during the WxWeb program provide the basis for the analysis presented here. Future funding from the FHWA under the Clarus Initiative will be used to generalize and extend the current algorithm.

RESEARCH OBJECTIVES

The goal of this research is to develop an automated weather variable extraction system that utilizes existing visible camera technology. In addition, the user should only be required to enter rudimentary location (latitude, longitude, elevation) and viewing information (minimum/maximum viewing distance).

The initial focus is on daylight imagery, as nighttime imagery requires some ambient or fixed-point source lighting and more extensive analyses that are outside the scope of this initial research.

The initial goal is to estimate the overall visibility automatically based on the characteristics of the image. Meteorological visibility is defined in several ways in the Glossary of Meteorology (AMS 2005). The general definition is the furthest distance that a manual observer can see a defined reference object with the naked eye in a given direction. The manual observation reported is the prevailing visibility, which refers to the median visibility gathered from around the horizon. Automated visibility sensors used on Automated Surface Observing Systems (ASOS) are designed to measure the prevailing visibility by assuming that the conditions between the sensor's receiver and transmitter represent the nominal conditions around the horizon. Since the actual visibility may not be homogeneous over the entire domain, it is quite possible that the visibility estimate of the laser sensor could differ from a manual observation. Similarly, visibility measured by a fixed camera and viewing angle may not be the same as either the manual or laser sensor. The camera captures prevailing visibility in the direction the camera is pointed, which is called directional visibility. Directional visibility may differ significantly from the local laser sensor when the contaminant causing the visibility reduction is not present at the point of measurement. These situations might occur in the case of approaching rain/snow showers or cloud decks, fog banks growing or decaying, and other more localized atmospheric phenomena.

In addition to visibility, the algorithm is designed to incorporate other weather variable algorithms in the future (e.g., fog or precipitation detection and trends, road condition detection and trends). Of equal importance is the need to recognize when the camera image is impaired either due to hardware failure or external objects (dirt/precipitation on the lens or unexpected blockages from vehicles).

DATA GATHERING

Camera Locations

Two site locations were fitted with a mid-resolution (320x240) digital camera. The first site was located at MIT/LL's Annex-2 facilities atop a 25-meter tower on Katahdin hill at Hanscom Air Force Base in Lexington, MA. During testing, the camera collected images on a five-minute interval from February 2000 through March 2000. The primary camera view for Annex-2 was northwest over the airfield. Figure 2(a) is an image collected on a clear day, with visibility in excess of 50 kilometers. There are several distinctive features in the camera image. First, the distant horizon is the small mountains located in southwest New Hampshire, all at distances greater than 50 kilometers. Second, Hanscom airfield is located in the center of the image, at a distance of 3.2 kilometers to the far side and 1.5 kilometers on the near side. The smokestacks visible on the left side of the image are at a distance of 400 meters. Finally, the ASR radar located in the foreground is 60 meters from the camera. The image has significant clutter in the foreground from several trailers and other vehicles. The camera housing is also visible in the upper-left corner of the image.

The second camera was located atop an instrument shelter at the Harriman and West airport in North Adams, MA and operated by capturing images at one-minute intervals from March 2000 to March 2001. Although the camera was remotely controllable, the primary camera view is to the west of the airport over the Williamstown valley, located along the Taconic Mountain Range. Figure 2(b) is an image collected from the Harriman camera on a clear day. There are several distinctive features in the camera image. First, the most distant horizon is the Taconic Range, located 6.7 kilometers to the west of the instrument shelter. Second, the airport's fuel storage facilities are located in the foreground at a distance of 42 meters. Third, the airport's hangar facilities are located in the center and the north (right) side of image. Two aircraft hangars are also visible, one behind the fuel storage facility (400 meters) and one to the right and behind the facility (200 meters). On the east (left) side of the image are the runways, taxiways, and parked aircraft. Also visible on the left side of the image is a near ridgeline, located approximately 2.1 kilometers from the camera. Finally, the camera's housing bracket is seen at the top of the image.

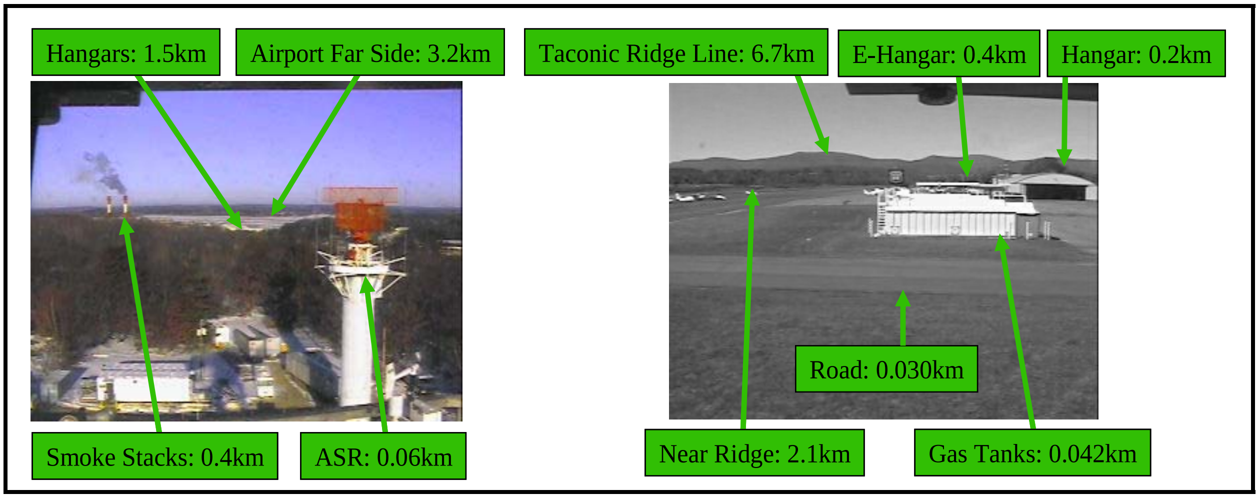

Figure 2. (a) Hanscom AFB camera view, left, and (b) Harriman and West camera view, right

Meteorological Data

Weather data was gathered from sensors co-located with each camera. Standard measurements of temperature, dew point, pressure, wind speed and direction, and visibility were gathered continuously over the test period. Of key interest for verification in this study were visibility measurements gathered using a Vaisala FD12-P laser to estimate visibility by analyzing the scatter of the laser beam. The FD12-P makes for an excellent automated method of generating standard meteorological visibility estimates. However, as mentioned above, the FD12-P produces an estimate of the visibility using a small spatial sample (Figure 3), because the distance from the laser transmitter to the receiver is only three feet.

Figure 3. Vaisala FD12-P laser visibility sensor

Example Test Site Images

Figure 4 depicts several examples of images from Harriman with fog at the airport. Figure 4(a) is from August 31, 2000 at 11:00Z, visibility at this time reported at 130 meters. Figure 4(b) is from August 21, 2000 at 12:00Z, visibility estimated at 450 meters. Figure 4(c) is from August 7, 2000 at 12:00Z, visibility estimated at 3,500 meters. Figure 4(d) is from August 16, 2000 at 12Z, visibility estimated at 9,200 meters. Several important things can be observed in this series of images. First, the clarity of the foreground objects (most notably the fuel storage facility) improves as the visibility increases. In Figure 4(a) the fuel facility is visible; however, it appears fuzzy. In Figure 4(b) the fuel facility is visible and the edges are more distinct than in 4(a). In Figure 4(c) the image of the fuel facility appears sharp, but the more distant Taconic ridge line is missing. Conversely, in Figure 4(d) the foreground buildings and the distant ridge line can be seen clearly. The background image also changes from the lowest to highest visibility images. In the first image, the background (grassy areas and pavement) has very little texture, but by Figure 4(d) the background has more texture, and is of a lower grayscale value. We can also observe in the last image the presence of rain drops on the camera enclosure. This data quality issue presents a problem for weather algorithms and is addressed in the algorithm development section.

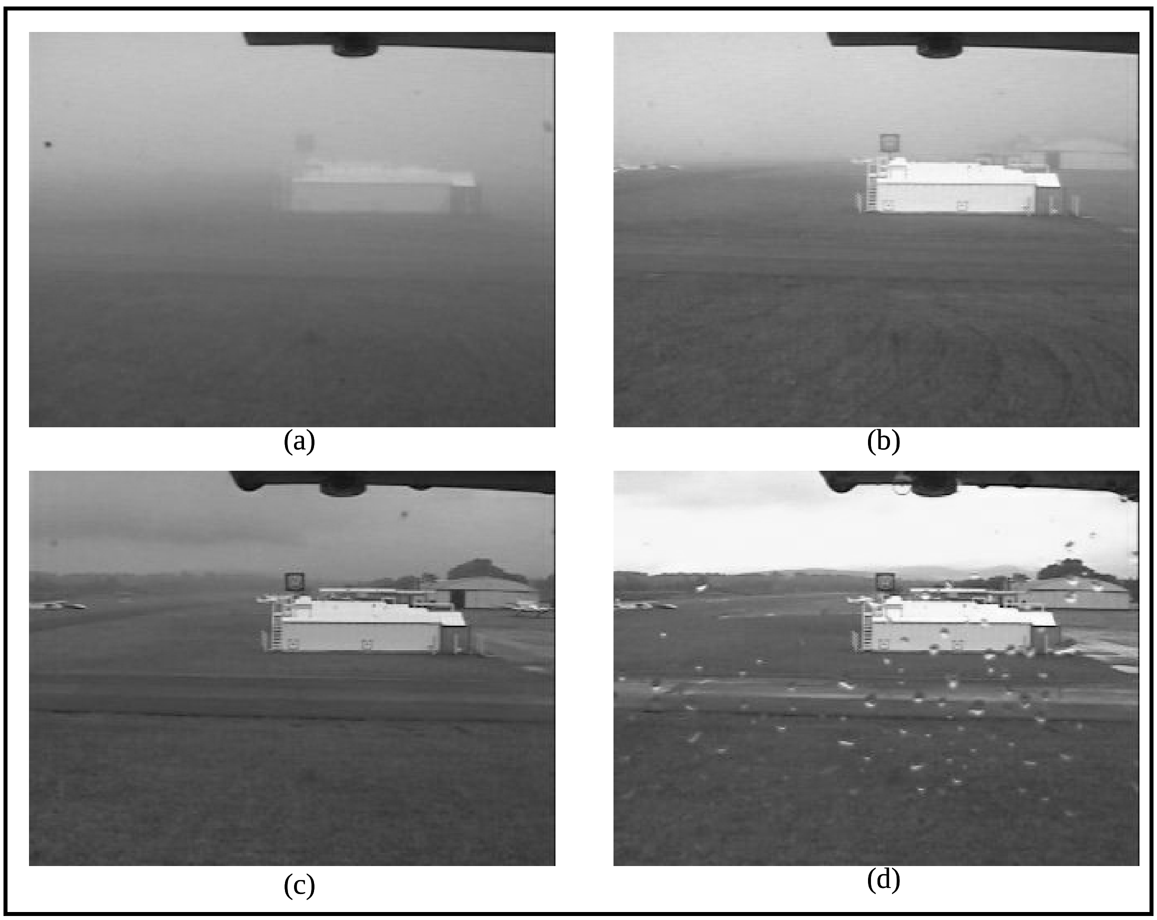

Figure 4. (a) August 31, 2000 at 11Z, visibility 130 m; (b) August 21, 2000 at 12Z, visibility 450 m; (c) August 7, 2000 at 12Z, visibility 3,500 m; (d) August 16, 2000 at 12Z, visibility 9,200 m

Figure 5 depicts several examples of images from Annex-2 with fog present. In the first image, Figure 5(a), the visibility is less than 100 meters. This image is from February 11, 2000 at 20:00Z. The ASR radar located 60 meters from the camera is faintly visible, almost blending in with the fog. The second image, Figure 5(b), is from February 18, 2000 at 20:00Z. The visibility at this time was just over 1,000 meters. Most notable in this image is that the smokestacks are now visible and a ridgeline is now visible that was not detectable in the lower visibility images. Figure 5(c) is from February 27, 2000 at 17:00Z. The visibility at that time was just over five kilometers. Now the airfield is visible and the far side of the airfield is faintly visible. The final image, Figure 5(d), was taken on February 20, 2000 at 14:00Z. The visibility is greater than 10 kilometers at this time. Snow cover is plainly visible in Figure 5(b) and 5(d), while in Figure 5(b) snow is actually falling.

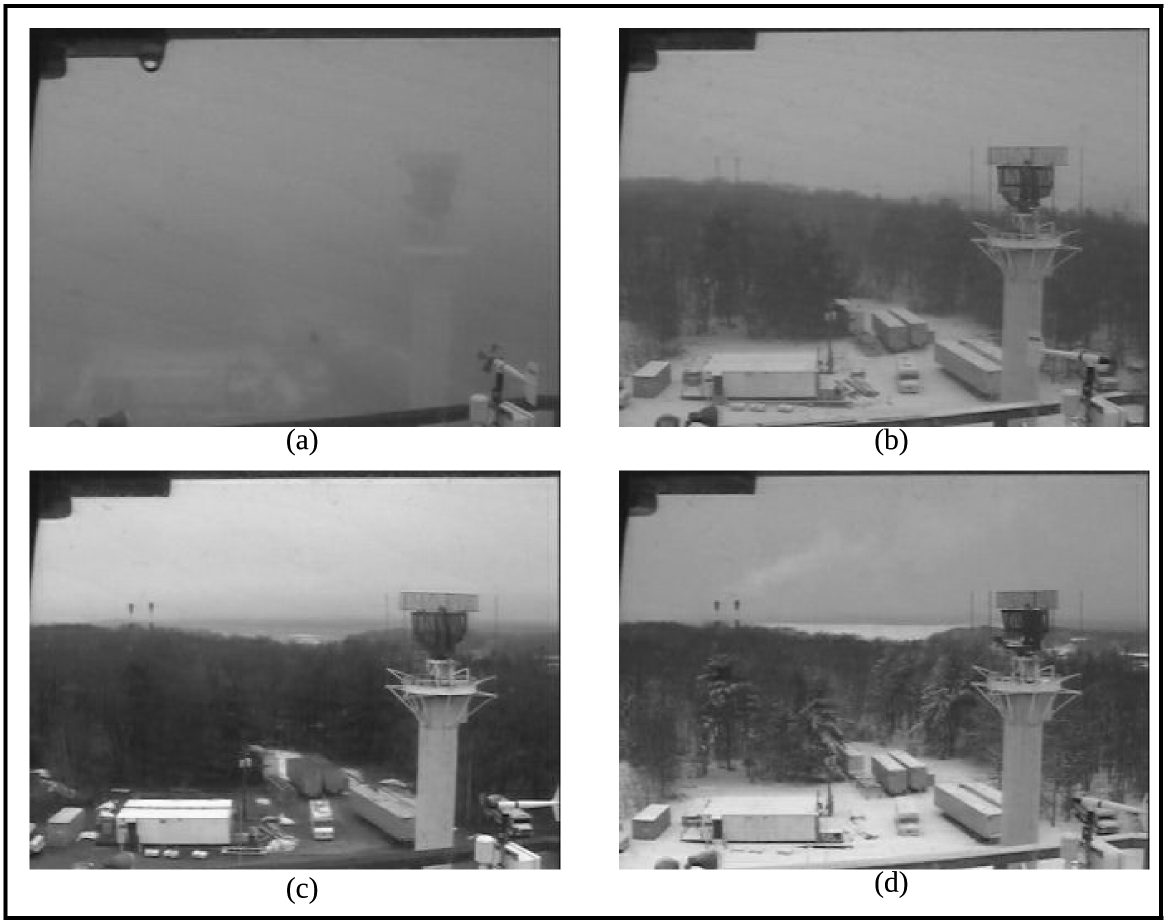

Figure 5. (a) February 11, 2000 at 20Z, visibility less than 100 m; (b) February 18, 2000 at 20Z, visibility 1,000 m; (c) February 27, 2000 at 17Z, visibility 5 km; (d) February 20, 2000 at 14Z, visibility 10 km

ALGORITHM DEVELOPMENT

Defining Features

One of the primary goals of the envisioned algorithm is that it can be easily deployed in a variety of environments with little manual site setup. As such, it is better for the algorithm to rely on overall features in a subject image rather than explicit knowledge of the distance to various objects. Therefore, the core of the algorithm is based on analyzing the entire image itself and edge features within the entire image. The images in Figure 6 show the clear image from Figure 2(b) and the heavy fog image from Figure 4(a), but with an edge extraction technique applied. In the high visibility case on the left, edges both near and far can be seen quite clearly. In the image on the right, however, the furthest edges that can be seen are those of the gas tanks and associated building located some 42 meters from the camera. The hangar sitting at 200 meters is completely obscured. Indeed, the laser-measured visibility for this image is 130 meters. Similarly, Figure 7 shows the same edge losses in the Hansom AFB images from Figures 2(a) and 5(a). The nearest edges in these images are those from the radar tower some 60 meters from the camera. No other edges can be seen in the low visibility image on the right and the laser-measured visibility in this case is approximately 100 meters.

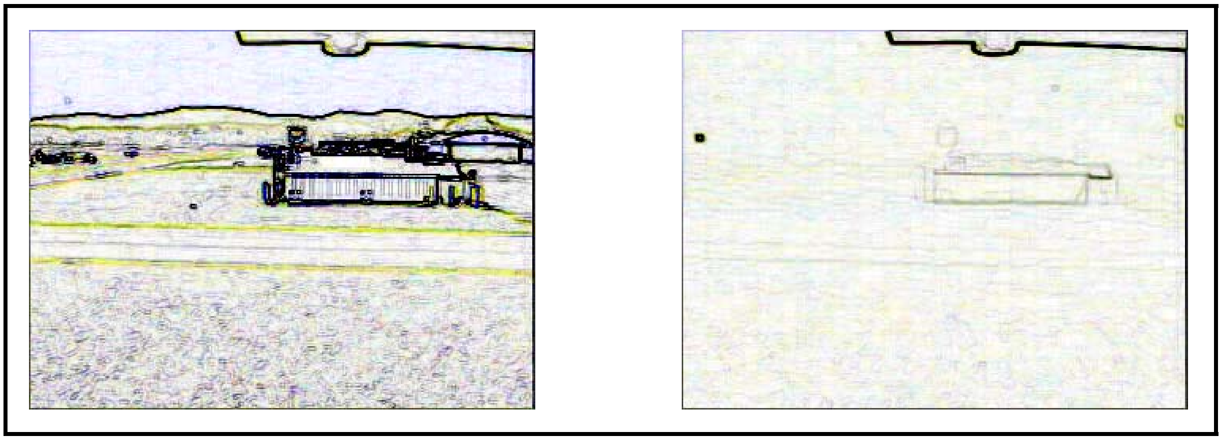

Figure 6. Edge-extracted images for the Harriman airport from a clear day (>20km visibility), left, and a low visibility day (130 m), right.

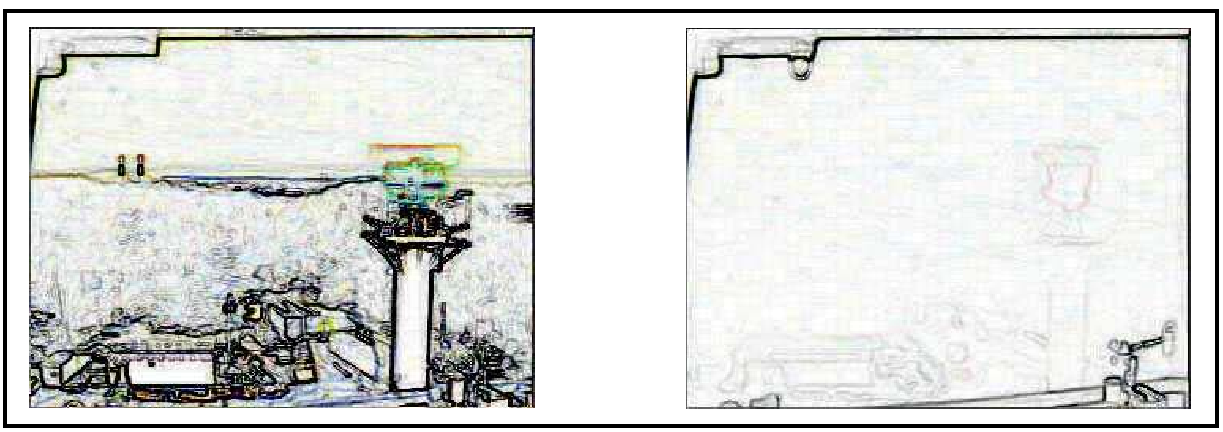

Figure 7. Edge-extracted images for the Hanscom AFB camera from a clear day (>40km visibility), left, and a low visibility day (< 100 m), right.

Based on reviewing dozens of low visibility events, it was clear that finding a way to correlate edge loss with visibility was a concept worth exploring. A clear image contains a full set of expected edges; these are the strong edges associated with, for example, buildings, trees, the horizon, and roads. As visibility decreases, fewer and fewer expected images are visible, and the loss of these edges occurs from the furthest edge to the closest as visibility approaches zero. Determining expected edges is accomplished by maintaining a composite image compiled from a historical average of daylight imagery within the system. In addition, constant but weak edges are also removed from the composite image, leaving only high-signal edges that should be found in any clear image. In each image, of course, there are unexpected edges; these are edges associated with traffic, dust/water on the camera lens, snow piles, and other varying phenomena. Figure 8 illustrates the concept of separating expected and unexpected edges within the system. Composite edges are shown in the upper left, a building-shaped edge near the bottom with an average weighting of 0.8 (on a hypothetical scale of 0.0–1.0 edge strength, with 1.0 being a strong edge) and a horizon edge with an average weighting of 0.5. Weaker edges (below some threshold, in this example 0.5) are removed from the composite image. The current edges in the lower left represent the edges from an incoming image. In addition to the expected edges seen in the composite image, there are unexpected edges from transient objects (in this case, rain drops on the camera shield). Expected edges are extracted from the current edge field by finding matching edges within the composite edge field. The relative strength or weakness of expected edges as compared to the composite field is directly proportional to the reduction in visibility. Unexpected edges are strong edges (>0.5) that are not associated with a corresponding composite edge. This illustration is conceptual, but the system examines each pixel within an image to determine its edge strength. While those strong pixels will make lines similar to the ones shown, the signal strength may vary significantly.

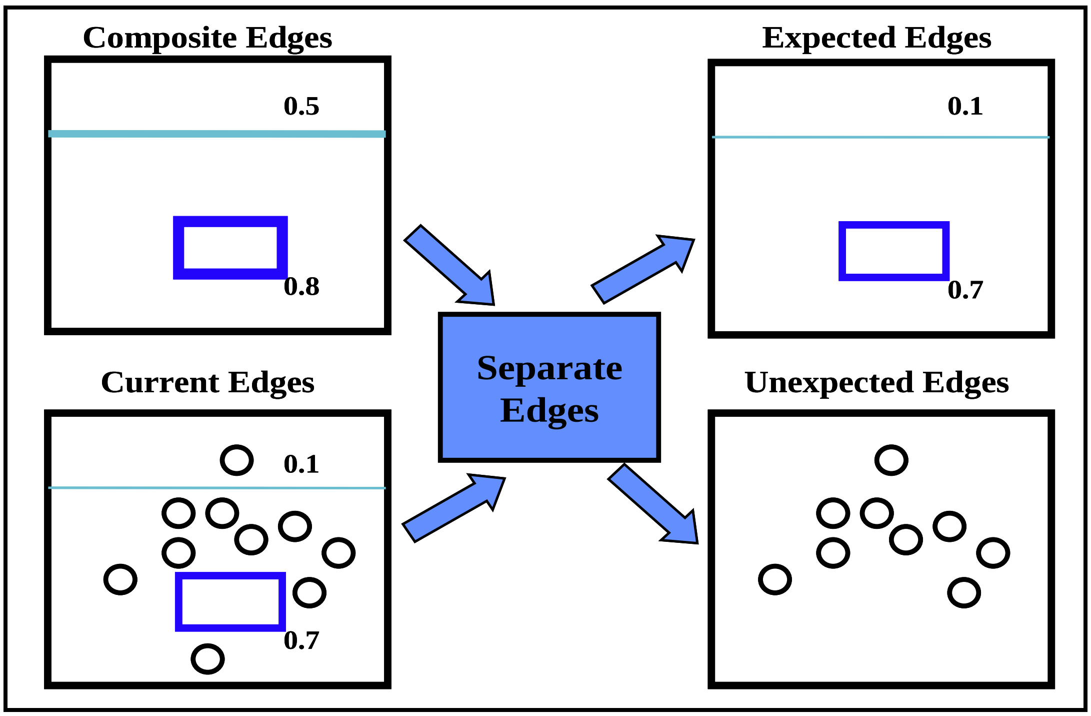

Figure 8. Illustration of edge analysis to separate strong, long-term expected edges from strong but transient unexpected edges

Removing unexpected edges is crucial to calculating an accurate estimate of the true visibility. The example in Figure 9 shows how the system can effectively eliminate these unexpected edges. Both the top (Feb 28th) and bottom (Feb 21st) images on the left are days with visibilities greater than 10 km. However, the top image has a large number of raindrops on the camera shield. Edge detection (middle row) and expected edge extraction (right) relative to the composite image edges are performed on both input images. As can be seen, the images on the right are quite similar, the strong expected edges have been preserved, and both images yield algorithm visibility estimates greater than 10 km.

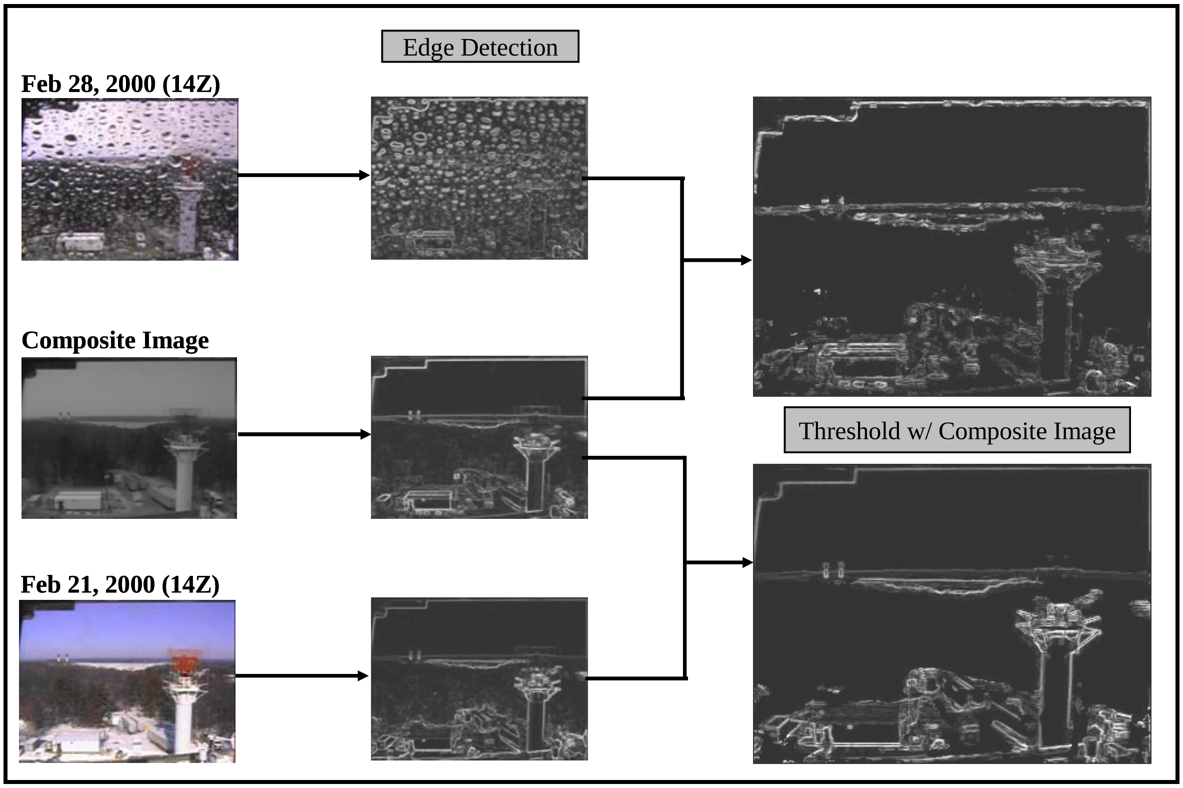

Figure 9. Example applications of data quality algorithms to remove transient edges

Analyzing Images

There are a multitude of edge detection algorithms and a variety of ways to quantify the strength of the edges found. The processing used for this analysis is the Sobel edge detection algorithm (Parker 1997). This algorithm looks for discontinuities in the image and generates a scaling (0–255) that represents the intensity of the edge. The Sobel algorithm yields images like the ones shown in Figures 6 and 7. Another approach to image analysis is the Fast-Fourier Transform (FFT), also referred to as the power of an image. The FFT is an efficient algorithm to determine the frequency spectra of digital signals. The FFT will produce an image with the same dimensions as the original image. However, the FFT image would not be visually informative to the analyst. Once the FFT is computed, the image magnitude is computed by performing a summation of all relevant pixels in the image. This summation provides a single measure of the relative frequency amplitudes in the input and composite image. Typical low visibility images, e.g., those caused by fog, tend to wash out high-frequency edges and therefore yield lower overall magnitudes than images on high visibility days.

For each camera image used in this analysis, the Interactive Development Library (IDL) was used to generate the Sobel edge detection. The IDL was then used to compute the image grayscale mean, standard deviation, and power from both the original image and the expected edge detection image. For each image, five measures of the relative strength of the input image were calculated. The first measure, normalized image magnitude, was calculated by calculating the magnitude of the input raw image and dividing it by the magnitude of the composite image. The next four measures were based on measures of edge strength as opposed to raw image pixel values. A normalization of each pixel's edge strength was performed first by dividing the input image by the composite image edge strength. Three of the measures, the edge mean, edge median, and edge magnitude, were based only on expected edges. The final determinant, total edge magnitude, was based on all strong edges (expected and unexpected) within the image. Figure 10 shows each of these measures from the Harriman airport camera for all daylight hours as compared to the true visibility measured by the FD12P visibility sensor (from July 1 to October 31, 2000). There is good agreement between the normalized values for the mean of the edges (r=0.70), the median of the edges (r=0.65), the magnitude of the entire image (r=0.62), the expected edges (r=0.63), and all edges (r=0.77). There are a few outliers, and closer scrutiny of each of these cases reveals that, as noted earlier, they are due to valid differences from the visibility sensor in the FD12P.

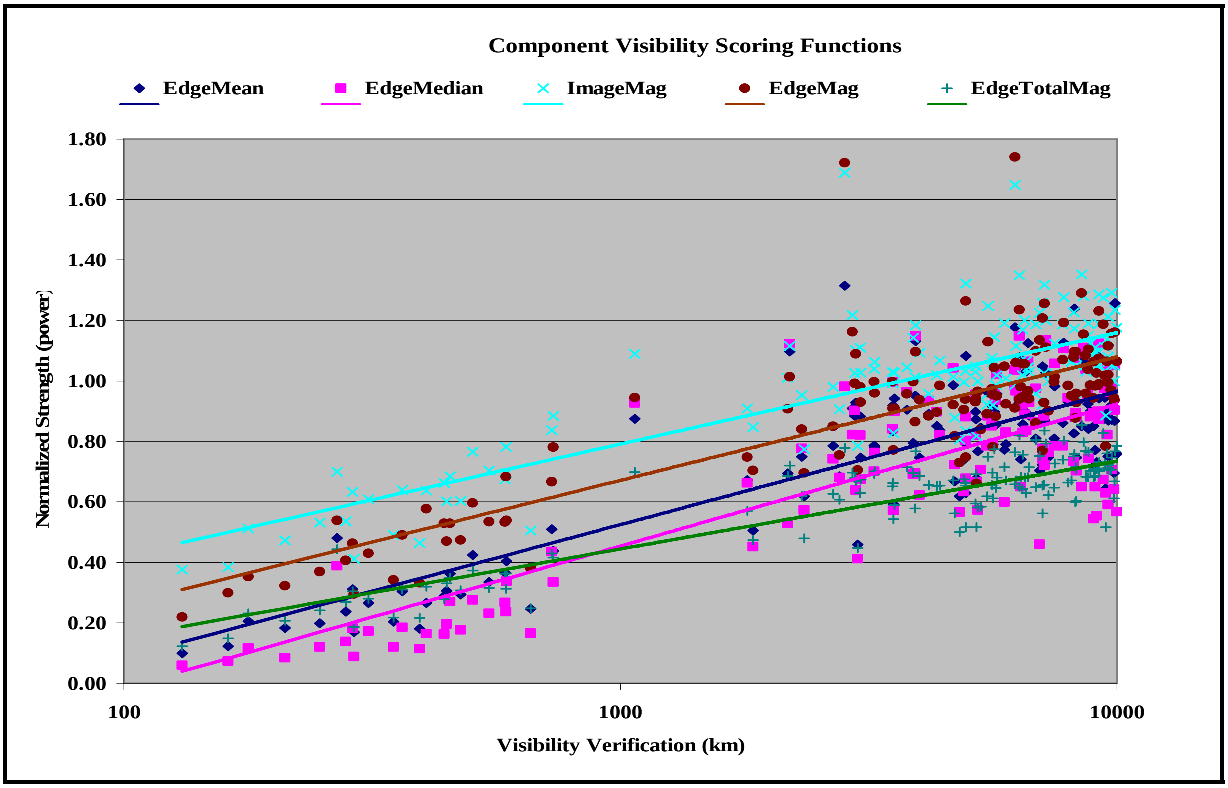

Figure 10. Comparison of edge and image normalized strength ratios to true visibility as measured by the FD12P lidar for all daylight images at the Harriman airport camera site

Initial Results

Rather than choose only one predictor for visibility, the algorithm uses an average visibility value as predicted by each of the predictors. This process, sometimes called fuzzy logic, often yields better results than any single predictor because the consensus reduces the impact of outliers. Figure 11 shows the algorithm results by using the fitted lines shown in Figure 10 as predictor functions. The visibility values are broken down into categories: less than 1 km, 1 to 5 km, and greater than 10 km. The overall probability of a correct visibility categorization was 73.4%; however, the crucial less-than-1 km category was correctly predicted 90.3% of the time. The worst performing category was the greater-than-10 km category, but this range is often the area where the camera might see an incoming front, whereas the lidar sensor can only be estimated based on the local conditions. While these results are promising, they are based on a single camera, and the low visibility comparisons are based on a small set of data.

Lidar (FD12P) Visibility (km) \ Video Estimated Visibility (km)	<1	1-5	5-10	≥ 10
<1	28	2	0	0
1-5	1	24	11	6
5-10	0	10	64	32
≥ 10	0	46	174	663

Figure 11. Prototype visibility algorithm scoring results on daylight imagery for the Harriman airport camera test site, July 1 to October 31, 2000

FURTHER RESEARCH

Clarus Research Initiative

Clarus (Latin for "clear") is an initiative to develop and demonstrate an integrated surface transportation weather observing and forecasting and data management system, and to establish a partnership to create a nationwide surface transportation weather observing and forecasting system (www.clarusinitiative.org). Part of the Clarus charter is to investigate new technologies that have potential application to surface transportation weather. As such, the FHWA will be funding a one-year effort to develop the automated camera visibility algorithm further. While the algorithm detailed above shows promise, there are many technical difficulties to overcome. The Clarus research effort will have three main components: (1) survey the current camera usage by state DOTs; (2) analyze and enhance the current algorithm utilizing MIT/LL-controlled cameras, with particular emphasis on snowplow cab-height visibility; and 3) extend the prototype to operate on a set of existing state DOT camera images.

The initial results were created from one camera location, using correlation curves defined from that same camera. A first step under the Clarus research will be to perform the same algorithm analysis shown above on the Hanscom AFB test images. This will provide valuable information on the types of modifications that may be needed to make the algorithm more generic. In addition, FHWA funding will be used to install a set of two cameras at MIT/LL facilities. A survey will be performed to determine the characteristics of the various state DOT camera installations. The MIT/LL cameras will be similar to those used by DOTs across the country. The heights of the two installations will be near-ground (5–15 meters) and above-ground (25–50 meters) to mimic typical DOT installations. This tiered camera installation will allow the researchers to analyze the correlation between standard-height cameras and the visibility at cab-height. Cab-height visibility is critical for snowplow operators so that they can be warned when road conditions are reaching zero visibility conditions, making operations too risky. Additionally, a lidar visibility sensor (the Vaisala FD12P) and solar radiometer will be co-located with the cameras, along with standard weather data sensors. These test cameras will be used to test and tune the prototype camera visibility algorithm. The biggest challenge will be to develop a system that is transportable to a wide variety of imagery but with as little tuning and site surveying as possible. Once the system is improved and tuned, we will access and process standard DOT cameras available in a variety of cities and states.

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Application of X-Ray CT Scanning to Characterize Geomaterials Used in Transportation Construction

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ABSTRACT

Conventional visual evaluation techniques (i.e., microscopy) of geomaterials are limited to the inspection of two dimensional surfaces, leaving internal three-dimensional characteristics ambiguous or resulting in costly slice-by-slice assembly to generate three-dimensional representations. Improved X-ray–computed tomography (X-ray CT) technology now allows a sample to be analyzed nondestructively and viewed in three dimensions. The outcome is the unique ability to obtain quantitative results from volumetrically imaging a sample's complete structure. Application of this technology is enabling a new area of research in the characterization of geomaterials used in transportation construction (e.g., concrete and soils).

Much valuable information can be gathered from CT scanning technology. Cracks can be mapped in soil samples to quantify spatially the planes and angles of failure. Porosity can be determined throughout sand and gravel samples. Air voids and aggregate segregation can be mapped in portland cement concrete. In pervious concrete, void matrixes can be analyzed for clogging, connectivity, and size.

A state-of-the-art CT scanning chamber was used to develop the images, and post-processing computer programs were implemented to interpret data for evaluation of geomaterials. This paper summarizes research techniques and describes the hardware and procedures required to conduct CT scanning for geomaterial applications. Finally, example images illustrating volumetric files are presented for various geomaterials.

Key words: geomaterials—nondestructive testing—X-ray–computed tomography

INTRODUCTION

X-ray computed tomography (X-ray CT) offers the field of material engineering the unique ability to view internal characteristics of specimens nondestructively and is applied in this paper to study various geomaterials (e.g., concretes and soils). This technology is made possible by using measurements of x-ray attenuation, which is a function of material density. In short, the product of this type of analysis is the creation of volumetric maps of variations in material density. The process works by positioning a sample inside an x-ray fan beam and projecting its shadow onto a special camera or detector that translates x-ray energy into electrical current (see Figure 1). As the sample is rotated inside the x-ray fan beam, this shadow is translated into a two-dimensional cross-section. By measuring several of these cross-sections at small intervals, the cross-sections can be stacked one upon another to form a three-dimensional digital representation of variations in sample density. True microfocus X-ray CT utilizes x-ray beam sources with spot sizes of only two to five microns and can produce volumetric digital files with effective pixel sizes of a few microns (Zhang et al. 2003).

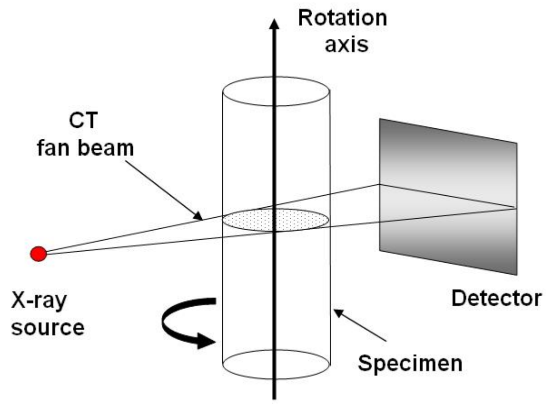


Figure 1. X-ray CT scanning setup

X-ray CT has been used recently in such applications as wetting phase displacement characteristics in porous media (Ham and Willson 2005); porosity distribution in rocks (Bashar et al. 2005); detection of large (over 40 cm^3), very dense (metal), and very low density (foam) inclusions in concrete (Diagle et al. 2005); spatial distribution of water, air, and soil solid phases throughout a soil sample (Rogasik et al. 1999); detection of disturbance zones in fine sand resulting from insertion of a cone penetration (Ngan 2005); characterization of fine-grained sand (0.22-mm mean diameter) after traditional axisymetric (triaxial) compression failure (Alshibli et al. 2000); and analysis of fracture geometry in rock that has failed under tension (Walters et al. 2005).

At Iowa State University's Center for Nondestructive Evaluation (CNDE) complete microfocus X-ray CT systems have been created, including the development of customized software for data acquisition, volumetric file reconstruction, and visualization. A 64-node Linux is used in the CT reconstruction. The chamber used for these scans utilizes a 130-kilovolt microfocus X-ray tube capable of 2.5-micron resolution and 1400x1400x500-voxel (3D resolution unit) data volumes. Because all software was designed by developers at the CNDE, the unique ability to alter and add to software is available (Zhang et al. 2005).

PROBLEM STATEMENT

The Department of Civil, Construction and Environmental Engineering at Iowa State University conducts research on various construction materials used in transportation engineering. In the evaluation of these materials, both quantitative and qualitative characterization of internal structure is of significant importance. This paper illustrates images from 3D volumetric files, explains the importance and implications of such images, and describes future analysis techniques planned for the evaluation of various construction materials. It also outlines a post-processing technique used to improve image quality of computer-processed X-ray CT data.

MATERIALS AND METHODS

Results from four different materials are described in this paper: silt subject to compression loading, geogrid-reinforced quartz sand, self consolidating concrete (SCC), and pervious concrete.

Silt

The silt used in this study has a mean particle diameter of 24 microns. Samples prepared for this analysis were consolidated using a computer-controlled pressure chamber and then failed in triaxial compression. Samples were compacted into 2-in. x 2-in. x 4-in. (5.1-cm x 5.1-cm x 10.1-cm) cylindrical molds using three lifts of drop hammer compaction. The specific example illustrated in this paper was consolidated to 250 kilopascals (36.3 psi) from lateral water pressure while disallowing vertical expansion. After consolidation equilibrium was reached, vertical pressure was introduced until sample failure occurred.

After sample creation and triaxially induced failure, samples were allowed to dry. X-ray CT analysis consisted of obtaining a three-dimensional volumetric file of 640x640x340 voxels in size, resulting in a cross-sectional resolution of 0.010 mm x 0.010 mm and a vertical resolution of 0.25 mm (the distance between slices). The scan was conducted at 130 kV and 0.1 mA.

Self-Consolidating Concrete

SCC is developed to provide good workability without causing problematic settlement or requiring vibration for consolidation. Through the development process of new mixes, it is important to monitor aggregate settlement and void distribution throughout a sample to ascertain a homogenous distribution of all material phases. That need is met nondestructively by using X-ray CT, allowing the samples to be used for subsequent testing (e.g., compressive strength, freeze-thaw).

SCC samples were created by simply pouring cement into a 3-in. x 3-in. x 6-in. (7.6-cm x 7.6-cm x 15.2-cm) cylindrical mold and allowing cement to cure. In the design of SCC, it is important to keep the densities of aggregate and cement similar so aggregate settlement does not occur (Wang 2003). Since objects with equivalent densities exhibit equivalent x-ray attenuation throughout their volumes, mapping variations in densities can be challenging. Also, cement cylinders have a high density (specific gravity of 2.3 +/-) and a relatively thick diameter so samples attenuate X-rays easily, making imaging difficult, especially towards the center of the cylinders. It was found that by sending higher voltage and current through the X-ray tube, the increase in electrical power increased the number and energy of X-rays passing through the sample, improving the crispness of aggregate edges and allowing data from the center of the samples to be gathered.

The SCC sample illustrated in this paper was scanned to obtain a three-dimensional volumetric file of 640x640x310 voxels in size, resulting in a cross-sectional resolution of 0.16 mm x 0.16 mm and a vertical resolution of 0.49 mm. Scanning was conducted at 130 kV and 0.45 mA. Also, a copper filter consisting of 0.005 inches of metal was placed over the end of the X-ray tube to attenuate low-energy X-rays that absorb easily in the exterior of a sample’s volume and result in erroneously high density values towards sample edges.

Geogrid-Reinforced Sand

Clean sand is a common foundation material in construction because of its high permeability. Sand has high strength when laterally supported, but has very little strength in tension or without lateral support due to its lack of inter-particle cohesion. Therefore, mats composed of interwoven synthetic polymer-coated fibers (geogrids) can be implemented to provide tensional strength to a sand structure and disallow expansion, thwarting failure zone development. X-ray CT can be used to characterize spatially the dead, dilated, and shear domains (Oda 1972) present in compressed sand samples, both with and without geogrid reinforcement.

Sand samples were prepared using ottawa silica sand, a clean subrounded to rounded sand. Sand was sieved and only sand retained on #8 and #10 sieves was used, resulting in a mean particle diameter of 2.42 mm.

In order to produce cohesionless samples for triaxial compression, sand was compacted into its 2.8-in. x 2.8-in. x 5.6-in. (7.1-cm x 7.1-cm x 14.2-cm) cylindrical mold fitted with a flexible latex membrane. Compaction was completed by hand tapping the mold's sides until 860 grams of sand settled inside with a porous stone at both ends, resulting in an initial void ratio of 0.65, which was found to be the maximal degree of compaction achievable by hand tapping. If geogrid reinforcement was added, the quantity of sand required was reduced accordingly to allow consistent void ratio. Samples were then confined using an internal vacuum until liquid pressure could be provided from the triaxial testing apparatus. Once samples were failed under triaxial compression, the experiment was halted and all instruments remained static while the sample was impregnated with EPO-TEK 301 resin from an impregnation chamber. EPO-TEK 301 has been shown to be optimal for freezing samples in place because it exhibits low viscosity (100 cps at 25^{o}C), cures at room temperature, and has minimal shrinkage during curing (linear shrinkage of about 1.5%) (Jang et al. 1999). Samples were allowed to cure for 24 hours prior to removal.

CT scan of the epoxy sand sample illustrated in this paper consists of creating a volumetric file of 640x640x380 voxels in size, resulting in a cross-sectional resolution of 0.16 mm x 0.16 mm and a vertical resolution of 0.35 mm. Scan was conducted at 130 kV and 0.11 mA. Also, a 0.005-inch copper filter was used to attenuate problematic low-energy X-rays. Post processing was then implemented, as described in the section entitled "Methodology for Post-Processing Three Phase Media."

Pervious Concrete

There are many uses for pervious concrete, such as in water retention. In the development of pervious concrete, it is important to understand liquid transport through its interconnected void spaces. X-ray CT provides a means to analyze pore continuity and flow channel dimensions to gain an understanding of how to achieve an optimal void network.

Samples of pervious portland cement concrete were created by pressing the concrete mix into a 3-inch (7.6-cm) diameter cylindrical mold so that sample weights consistent with that created by slip form paving were achieved. Samples were extruded after curing and the top and bottom sawed off to obtain a 3-inch–tall (7.6 cm) cylinder of pervious concrete.

The pervious concrete sample illustrated in this paper was scanned to produce a volumetric file dimensioned as 640x640x310 voxels, resulting in a cross-sectional resolution of 0.15 mm x 0.15 mm and a vertical resolution of 0.25 mm. Scans were conducted at 130 kV and 0.11 mA. Also, a 0.005-inch copper filter was used to attenuate problematic low-energy X-rays.

METHODOLOGY FOR POST-PROCESSING THREE PHASE MEDIA

Cross-sections of samples digitally imaged using X-ray CT are grids composed of values that represent densities. These values are presented using voxels. It is useful in many instances to be able to post-process these voxel arrangements to filter out unwanted effects and to organize data in a way to simplify further analysis. Therefore, a method of post-processing volumetric files has been devised to satisfy the following three criteria:

1. Remove beam hardening effects caused by using a white X-ray spectrum of wave energies
2. Remove unwanted erroneous patterns (artifacts)
3. Identify different phases within a sample volume

Beam Hardening and Artifacts

Beam hardening effects are caused by lower energy X-rays being preferentially absorbed in the exterior regions of a sample. During CT reconstruction, it is assumed that a constituent of a sample, say aggregate found in a concrete sample matrix, has consistent ability to attenuate X-ray photons (absorption coefficient) regardless of location inside a sample volume. This assumption is invalid if there are different photon energies present, such as those produced by a white bremsstrahlung source, in which case lower energy photons are absorbed more readily than higher energy photons. The lower energy X-rays therefore tend to become attenuated close to a sample's edge before reaching a sample's deeper interior. The effect is that regions located closer to the edge are depicted during reconstruction as having erroneously higher densities than centrally located regions. This effect can be seen by viewing voxel intensities located along a line drawn across a cross-section of a volumetric data file (Figure 2a) and looking at the trend in voxel values along that line (Figure 2b). It can be seen that at the edge of the sample, voxel intensities tend to have a higher value than those in the central region. An artifact can also be seen where a ring of higher intensity has been depicted outside the sample edge. This artifact is caused by overloaded X-ray receptors on the detector.

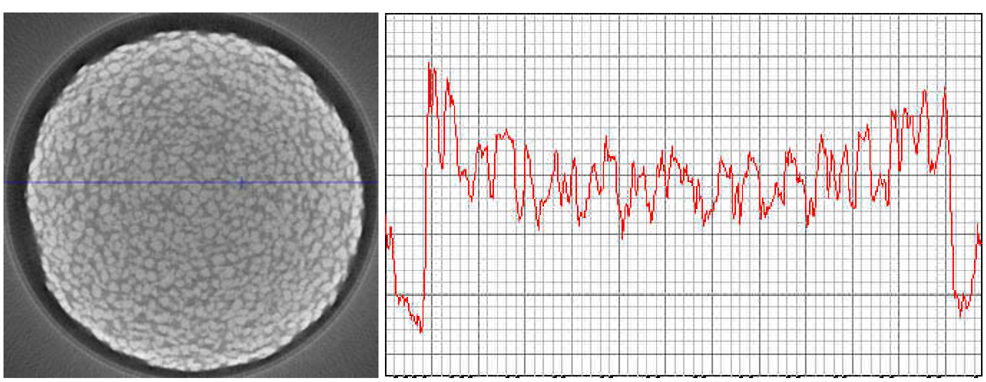


Figure 2. (a) Original sand slice and (b) histogram of intensities along the horizontal line

Identification of Phases

By identifying different phases within an X-ray CT–produced digital volume, it is possible to run computer algorithms based on voxel intensity values that could not be successful otherwise. In the case of the sample illustrated in this paper, an epoxy-impregnated sand sample is processed. It has the following three phases: air, sand, and epoxy. If air had a voxel value of zero, epoxy had a voxel value of no less than one, and sand particles had corresponding voxel values greater than those of epoxy, then the three phases could be easily identified. It would be possible to use the presence of air, now identified as having a voxel value of zero, as a means for a computer program to locate sample edges. By providing a threshold between the voxel intensity that splits epoxy regions from sand regions, a program could map the areas corresponding to those phases. Note that this may originally be impossible because epoxy voxel values (the height of the histogram troughs) at the sample edge can be higher than sand voxel values (the height of the histogram peaks) in the center of the sample (Figure 2b).

Process Steps

The first step proposed for processing the digital volume files is to split the volume into cross-sections to allow two-dimensional analysis. Once this is done, there are a slew of cross-sections that may be stacked to form a volumetric file (Figure 2a).

The second step is to use a program to identify the trend of the beam hardening artifact. This can be done by finding the maximum values found in different sections of a grid set throughout the image area and smoothing those values together to make a two-dimensional picture of the peak value trends (Figures 3a, 3b). This process can be repeated several times and averaged together to obtain a more accurate representation of the beam hardening trend.

Once the beam hardening artifact trend has been found and presented in an image (Figure 3a), its effect can be removed by subtracting the trend from the original image. In doing this, simultaneously the abnormal ring-shaped artifact will be lowered in voxel intensity.

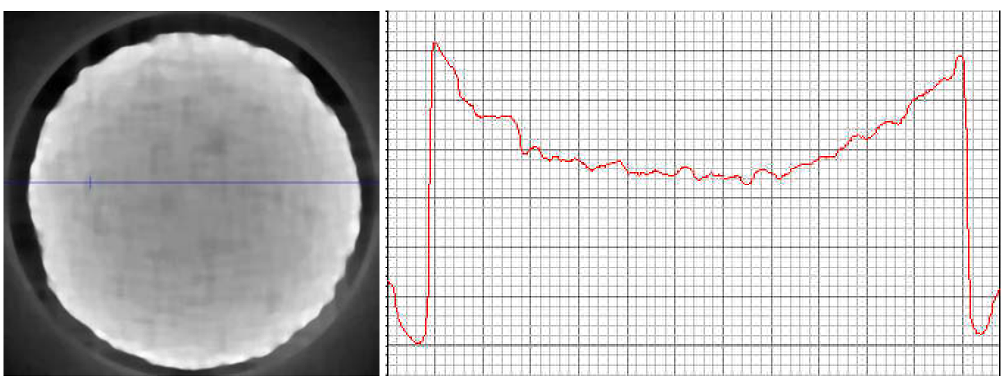


Figure 3. (a) Result of smoothing technique and (b) histogram of intensities

The ring artifact should now be of low enough intensity to be removed by assigning all voxel values below a certain intensity (an intensity below the lowest voxel value inside the sample area but above the highest voxel value of the artifact) a new voxel intensity of 0. This cutoff can also be useful, as it can define the edge of the sample volume. Now by assigning all areas without voxel values of 0 a voxel value of 1, you have effectively separated your air phase from your epoxy and sand phases (see Figure 10).

Now that the trend line of the beam hardening artifact has been identified, the ring artifact outside the sample area has been removed, and the air phase has been identified, it is just a means of taking the original image (Figure 2a), subtracting from it the trend of the beam hardening artifact (Figure 3a), and assigning all areas depicted as air an intensity value of 0; a final image is thus produced (Figure 4a). Note that troughs in the voxel intensity histogram near the edge of the sample volume are now well below the peak values near the sample's central region (Figure 4b).

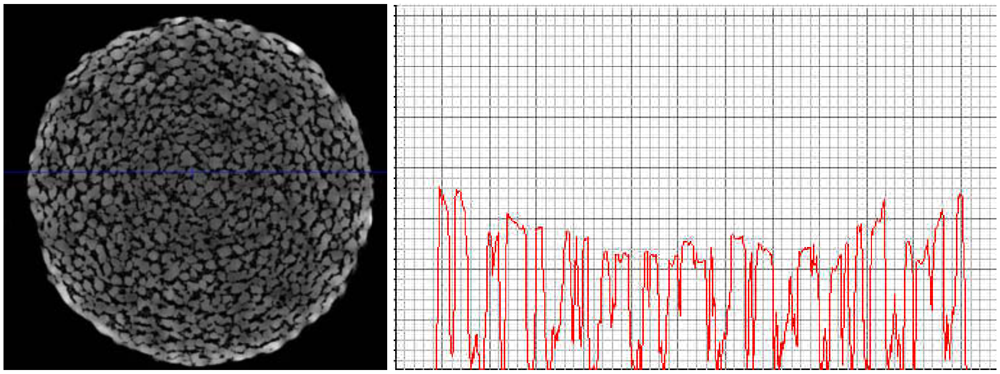


Figure 4. (a) Resulting image of post-processing and (b) histogram of intensities

Now that an image that can be easily processed has been produced, the same process can be reiterated for each slice of a volumetric file, and the slices may be combined into a new volumetric file. In order to decipher between sand and epoxy phases, a threshold voxel value can be assumed to be that which separates the two phases. By using a program to count the number of voxels below and above this threshold (excluding the value of 0, since it indicates air), it is possible to achieve a representation of how much sand and epoxy volume resides inside the sample, assuming the utilized threshold is correct. These phase volumes can be compared to actual measured volumes, and the threshold value may be adjusted accordingly until actual and measured volumes agree. Now that the threshold voxel value separating epoxy and sand has been identified, computer programs may readily process volumetric files and identify the three phases found inside the digital sample volume.

It should be noted that while the processing explanation is lengthy, once a program has been developed to process the X-ray CT–generated slices, it should be rather simple to operate.

KEY FINDINGS

Silt

A vertical profile of the failed silt sample (Figures 5 and 6) shows nearly vertical cracks in the center of the sample and very little cracking towards sample edges. Obvious expansion during compression can be seen by the bulging of the sample sides. There is a large, nearly horizontal shear band formed through the sample center. This is very different than the shear planes typical of failing symmetrically consolidated samples.

Figure 5. (a) Silt sample and (b) X-ray CT cross-sections

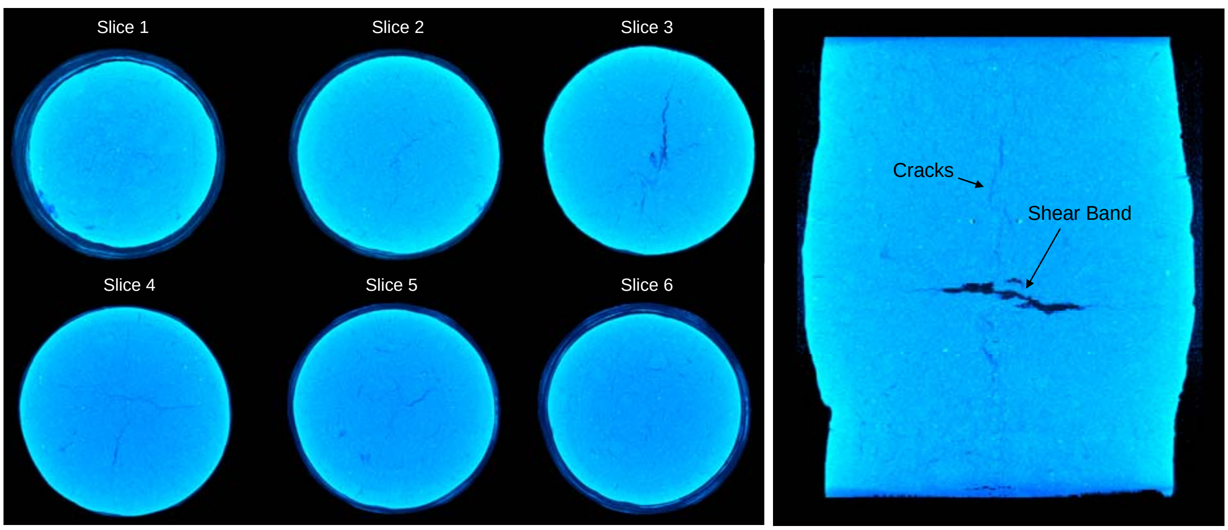


Figure 6. Silt sample vertical profile

Self-Consolidating Concrete

A profile of a SCC sample (Figure 7) shows uniform distribution of large aggregate except near the top center, where only smaller aggregate resides. This condition was present in two out of three scanned samples. As a result, it became apparent that there are edge effects thwarting aggregate settlement from concrete molds. Voids are distributed randomly throughout samples. Large voids tend to align themselves adjacent to aggregate edges, bending around their curvature.

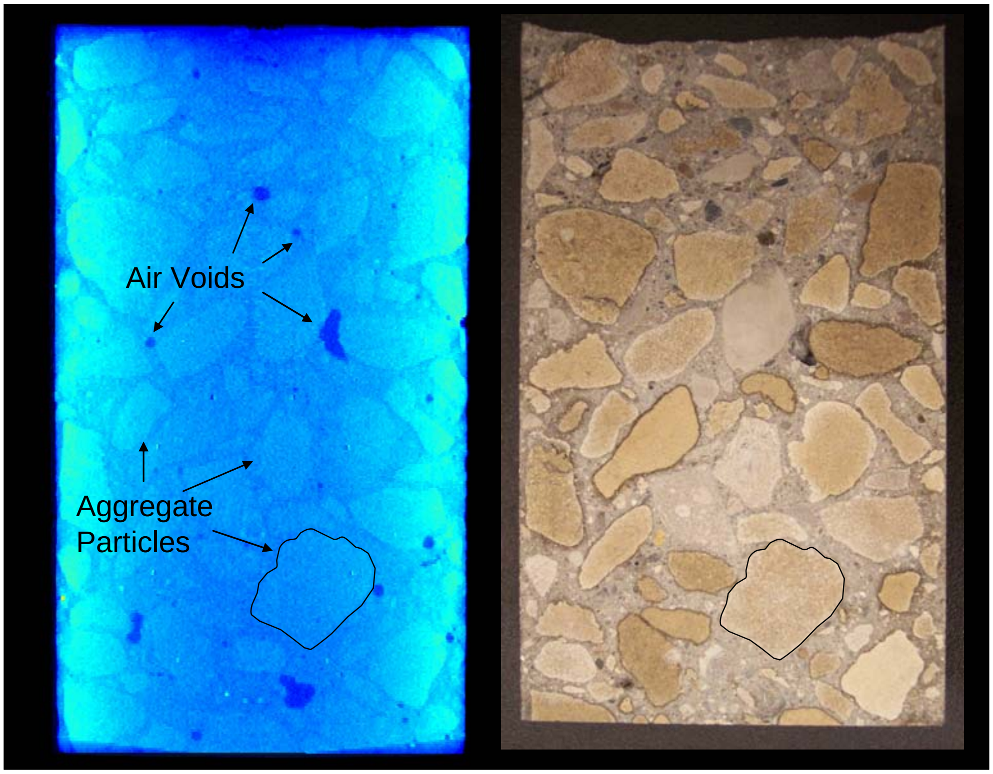


Figure 7. SCC sample vertical profile

Geogrid-Reinforced Sand

A profile view of an epoxy sand sample reinforced with three geogrids (Figures 8 and 9) is very hard to interpret using the naked eye. Therefore, the use of image post-processing such as that presented in this paper coupled with a program to determine void ratio in finite areas throughout the sample are currently being produced to better analyze these samples.

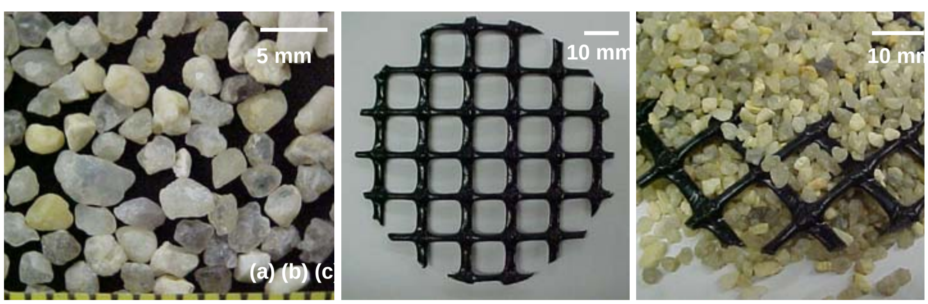


Figure 8. (a) ASTM quartz sand with D_{50} = 2.18 mm, (b) polyethylene biaxial geosynthetic reinforcement, (c) geosynthetic-reinforced sand

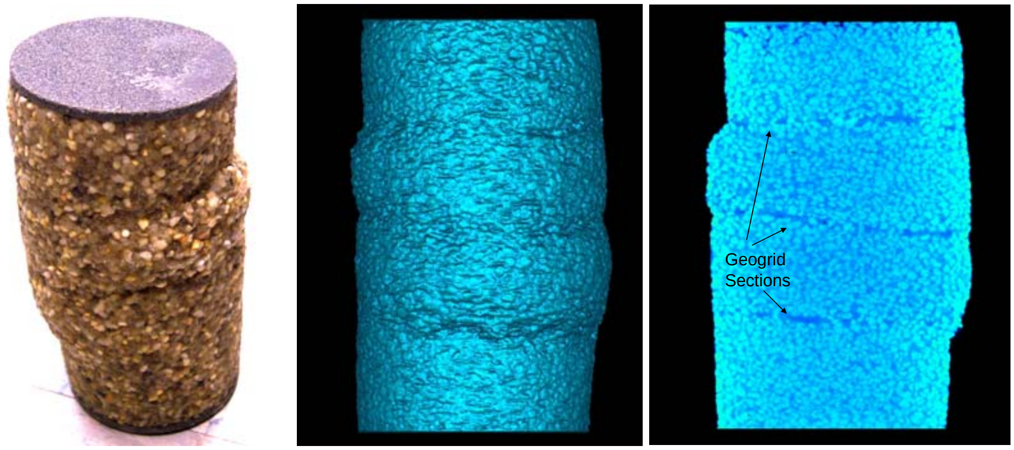
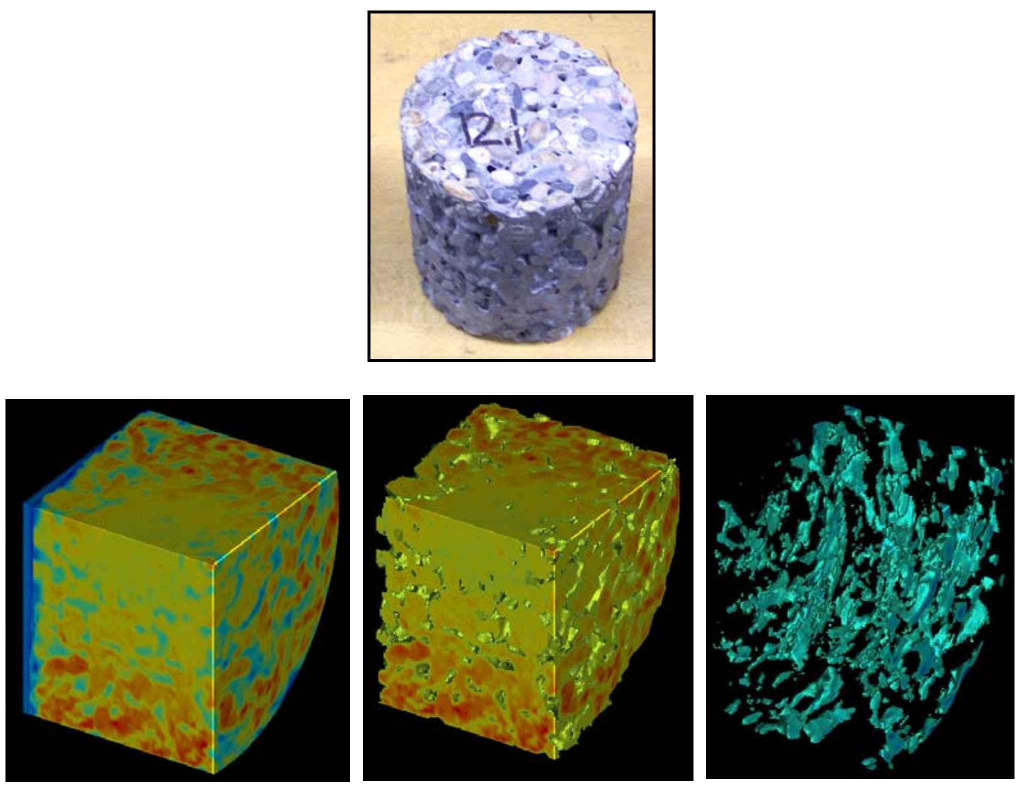


Figure 9. Epoxy sand sample vertical profile

Pervious Concrete

A horizontally tipped profile view of pervious concrete (Figure 10) shows pore spaces found within the sample. By looking through a volumetric sample, large voids that can collect fluid, referred to as nodes, and the thinner void channels connecting them, referred to as throats, can be measured and used to determine water flow through the concrete and pore space continuity, enabling better design for rapid water transport while retaining high strength.



(a) (b) (c)

Figure 10. Pervious concrete section: (a) composite sample, (b) solids component, and (c) air voids

CONCLUSIONS

X-ray CT provides a means to view the entire volume of materials used in transportation engineering. Using scanning equipment available at Iowa State University, samples consisting of laterally consolidated silt, self-consolidating concrete, sand reinforced with geogrid, and pervious concrete have been scanned and illustrated. Silt samples show cracks due to the changes in density found at failure planes. Self-consolidating concrete can be viewed to determine whether problematic aggregate settlement is occurring. Sand reinforced with geogrid can be post-processed in such a way that further computer analysis may be conducted to find void ratio distribution. Pervious concrete can have its internal void structure measured and modeled to provide a better understanding of fluid flow to aid design. By using post-processing programs and taking simple measurement, qualitative data volumes can be processed quantitatively. X-ray CT bridges the gap between unknown internal material characteristics and engineering solutions.

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Iowa Pavement Marking Management System: Initial Phases

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ABSTRACT

The importance of roadway pavement markings to motorists and pedestrians needs no reinforcement among the staff at the Iowa Department of Transportation (Iowa DOT). With an annual pavement marking program of approximately $2 million dollars and another $750,000 invested in maintenance of durable markings each year, the Iowa DOT is seeking every opportunity to provide all year markings in acceptable condition under all weather conditions. The goal of this study was to analyze existing pavement marking practices and to develop a prototype Pavement Marking Management System (PMMS). A practical and integrated PMMS would allow for more accurate development of annual marking needs, as well as provide guidance on durable pavement marking applications and overall marking strategies on a statewide basis.

Key words: beads—durable marking—operations—paint—pavement marking

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PROBLEM STATEMENT

The importance of roadway pavement markings needs no reinforcement among the staff at the Iowa Department of Transportation (Iowa DOT). With an annual pavement marking program of approximately $2 million dollars and another $750,000 invested in maintenance of durable markings each year, the Iowa DOT is seeking every opportunity to improve marking performance and minimize costs. A discussion of DOT concerns follows.

The DOT is concerned with pavement marking durability and with finding marking materials which will perform over a 12-month period (or over the winter as a minimum). Having a paint crew in each of the six districts has worked well from an operations perspective. However, given that Iowa is a snow-plow state, the winter maintenance operations present significant challenges in holding these waterborne markings over the winter. In contrast, contractor-applied durable or recessed markings have been found to be more durable, in most cases, but at a considerably higher cost than DOT-applied waterborne paint. As an example, DOT crews are applying waterborne paint at $0.025 per foot compared to durable materials which start at $0.70 per foot and can go up to over $3.00 per foot.

The DOT is also concerned with having the resources to maintain durable markings. With each construction season, the number of miles of durable markings increase on the DOT system as they are a popular item for new roadway projects. This places a considerable financial burden on the DOT as they have to budget for the maintenance of these markings over a two- to four-year cycle. Also, with durable markings being part of the standard bid items, the DOT wants to provide guidance to the districts on where these more expensive materials should be placed and where they will be maintained.

The Iowa DOT established a Pavement Marking Task Force (hereinafter referred to as “Task Force”) which was responsible for facilitating this research effort and for providing linkage with DOT staff and resources.

RESEARCH OBJECTIVES

The objectives of this research were to analyze existing pavement marking practices and to develop recommendations and a prototype Pavement Marking Management System (PMMS) which would allow for more accurate development of annual marking needs and guidance on the use of durable and traditional markings. These objectives were incorporated into the following project Task Force objectives:

1. Develop a statewide inventory of durable pavement marking installations.
2. Utilize retroreflectivity measurements to determine actual marking performance.
3. Develop and publish guidelines for the application of durable markings.
4. Present recommendations to the Highway Division Management Team (HDMT) regarding appropriate funding for statewide markings.
5. Use retroreflectivity data to coordinate activities between conventional and durable marking programs to maximize the effective use of funds for both programs.

This report summarizes existing marking practices, as well as key findings and recommendations from objectives 1 through 3 above.

KEY FINDINGS

This section provides a summary of key project findings. These findings represent a collaborative effort through working with the project Task Force. Task Force members represented a variety of positions within the Iowa DOT:

Will Zitterich	Office of Maintenance, Chair
Ron Beane	Office of Maintenance
Mark Black	District 2 Maintenance
Mark Bortle	Office of Construction
Tim Crouch	Office of Traffic & Safety
Joe Putherickal	Office of Materials
Pat Rouse	District 1 Paint Crew
Dan Sprengeler	Office of Traffic & Safety
Steve Wilson	District 6 Highway Division
Kurtis Younkin	Office of Traffic & Safety

Task Force members were assisted by

Neal Hawkins	CTRE, Project Investigator
Omar Smadi	CTRE, Co-Project Investigator
Zach Hans	CTRE, GIS Specialist

The remainder of this section summarizes the key findings as they are related to each of the 3 project objectives discussed in this report.

Objective 1—Durable Marking Inventory

Durable Markings Database

The DOT has road miles of durable markings, which are typically installed as part of a construction contract. The tracking of this information is less than ideal with the occasional issue of maintenance crews painting over these markings. The task force worked at developing an overall durable marking database format and inventory and at incorporating this information into a graphical GIS format. An example of this is shown on the following page in Figure 2. The Task Force developed alternative techniques for resident construction engineers to enter the durable marking data through what was termed a "section tool." This tool allows for pointing and clicking on a map to obtain the latitude/longitude limits of the durable marking as well as enter the material type and other installation details. This section tool was developed to accommodate a variety of pavement marking conditions and is shown in Figure 1.

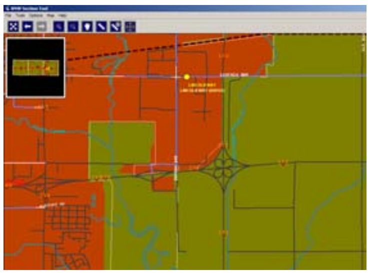


Figure 1. Screen shot of the section tool

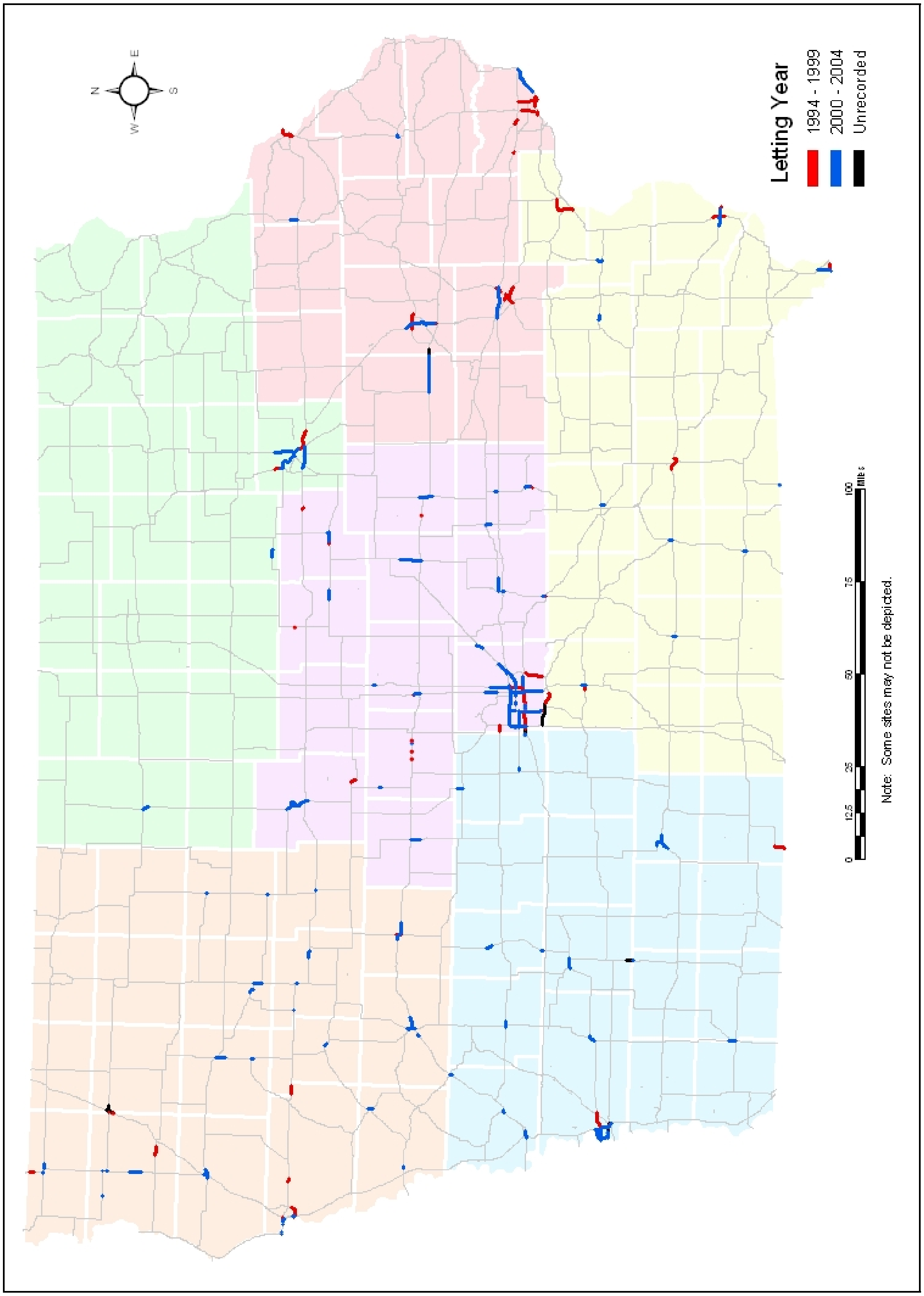


Figure 2. Durable pavement markings inventory

Objective 2—Utilize Reflectivity Information and Track Performance

A variety of issues were researched in an effort to meet the goals for this objective. A summary of these findings is presented below.

Pavement Marking Damage

Through field inspections and discussions with staff, the Task Force categorized the majority of damage to be related to snow or maintenance blades, winter maintenance application of sand and salt, high shear from weaving or cross-over vehicular maneuvers, or installation and material failure. A discussion of some of this follows.

Winter Maintenance. The Task Force worked with state climatologist to review 30 years of average snowfall by each Iowa DOT district and to map this information as shown in Figure 3.

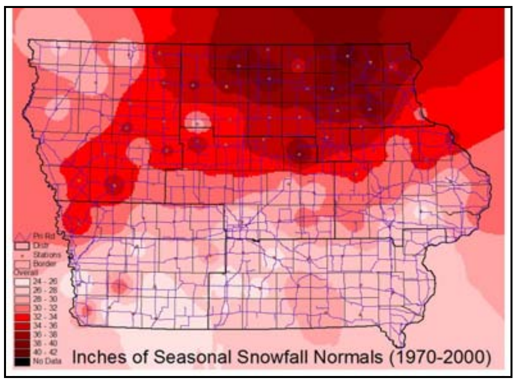
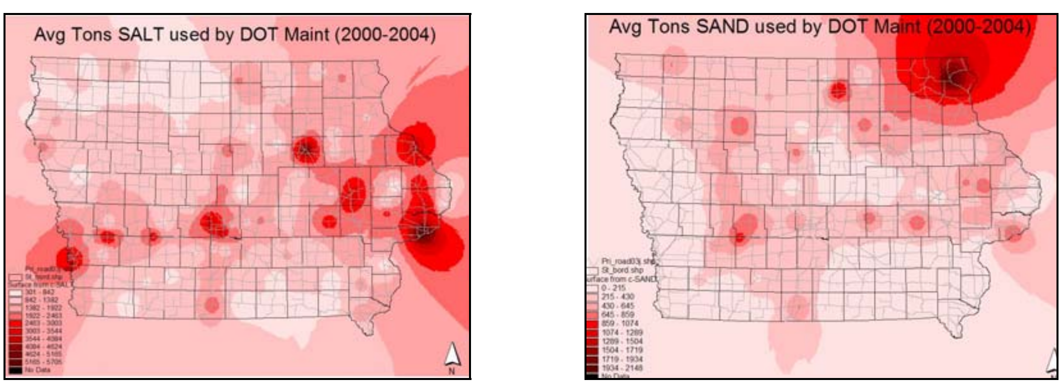


Figure 3. Average snowfall

The variability across the state, along with winter maintenance policies, create differences in the frequency with which individual routes are plowed each year and the opportunity for damage to surface applied markings. Existing snow plow and sanding activities are recorded on a person-hour or quantity basis and not by route/milepost. Each district develops individual winter maintenance procedures. Figure 4 depicts the average tons of salt and sand used by location over the 2000 through 2004 time period.



(a) (b)

Figure 4. Average tons of salt (a) and sand (b) by location over 2000-2004 time period

Edge Rut Maintenance. The Task Force found that edge rut maintenance can have a detrimental impact on marking reflectivity (edge line). Gravel is typically dumped or bladed onto the driving surface and then bladed off smoothly over the shoulder. In the summer of 2004, one District painted a white edge line and measured the reflectivity to be 400 millicandela/meter squared/lux (mcd). Within a week of painting, edge rut maintenance was performed for this stretch of roadway. Following this, the same line was measured and found to have been reduced down to a reflectivity of a little over 100 mcd. Figure 5 below depicts typical edge rut maintenance, where gravel is scraped across the white edge line. The Task Force also discussed a variety of examples where heavy traffic and/or turning movements have a significant impact on marking performance. This analysis on one year of data showed that marking performance is worse on older paved driving surfaces.

Figure 5. Edge line damage due to shoulder maintenance

Measuring Reflectivity

The Iowa DOT has established internal performance targets for markings at 300/200 mcd for initial reflectivity of white and yellow paint, respectively, and 150/100 mcd for replacement of white and yellow markings, respectively. Two methods are used to collect reflectivity information. For interstate, and some major 4-lane roadways, reflectivity is measured with the Lazerlux Van. All other measurements are completed by each District through use of handheld Delta LTL-X machines. A description of each method follows.

Lazerlux Van. The Iowa DOT has one Lazerlux Van which takes continuous readings on Interstate and major 4-lane highways. At the time of the study, the van was not equipped with GPS equipment, thus relying on route and milepost for reference. The van also relied on problematic floppy disks to transfer readings to other computers for analysis or storage. The Van is equipped with a 30 m geometry lazerlux reflectometer which samples markings every second and averages the results based on tenth mile segments. Each segment is then averaged for a specific route, using the mileposts as the beginning and end points of reference. The van is capable of taking readings from either side of the vehicle at an average speed of 55mph. Machine calibration is performed routinely at least once per day or when erratic measurements are observed. Calibration blocks are used for this process. In general, the Lazerlux van is a centralized process which covers the entire state. The crew consists of a driver and an operator who codes reading data by type of line, material, and location. Figure 6 shows a picture of the van.

Figure 6. Lazer Lux Van

Handheld Units. In the spring of 2004, the Iowa DOT purchased 3 additional handheld Delta LTL-X machines for a total of 6 (one per district). Each unit has the ability to record a GPS latitude and longitude with each reading along with the default of entering in the location through route and milepost. In 2003, District 1 began using these units as part of normal paint operations and positioned the unit roughly 2 miles behind the paint machine. As the paint became dry to the touch, an initial reflectivity was recorded and relayed to the paint operators. This constant calibration process became an invaluable tool in maintaining consistency and in dealing with the change in a number of conditions and weather throughout each day. In fact, District 1 found that after adjusting truck speed, paint and bead rates, and paint tips they were still not meeting their initial reflectivity goal. They examined the beads to find they lacked a coating needed for waterborne paint. Without the reflectivity feedback, the crew may have painted all summer with the wrong beads. The hand devices are 30-meter geometry machines and can be set to sample or average samples at a variety of settings. These units also have calibration blocks for validation. Figure 7 shows a picture of the device in use. The Iowa DOT Office of Maintenance has developed a standard protocol for Districts to follow regarding how often and how many readings to take when measuring roadway segments.

Figure 7. Handheld Delta LTL-X machine

Analysis Tools

The 2004 spring retroreflectivity data were used to depict marking condition information in a visual format using route and milepost as a reference. Some difficulties were discovered when using the route and milepost data due to the GIS background mapping which stops each route at the county borders. Figure 8 shows the data points collected with the handheld (left) machine and van (right) methods.

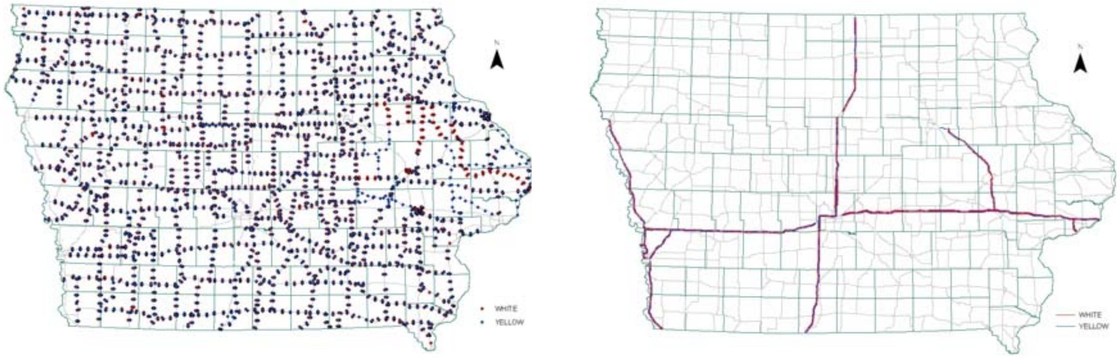


Figure 8. Data points collected with the handheld (left) machine and van (right) methods

Additional interpretation issues arise when route segments are concurrent with other routes and milepost references are confused. Through working with DOT staff, a series of four plots were used to evaluate early spring marking conditions on a statewide and district level. Figure 9 provides a sample of the GIS plots generated for 2004 reflectivity data. Similar plots were generated for each district at a more detailed scale. The plots are categorized to show reflectivity information as follows: Yellow Center Line, White Edge Line, White Dashed Center Line, and Yellow Edge Line.

Database

The Iowa DOT has two separate databases for pavement marking information which consist of reflectivity information (both van and handheld data) and paint application information. The van reflectivity information is read directly into the database through a spreadsheet format. Handheld data are retrieved from each machine electronically and then entered into a CITRIX interface which assists in formatting the data fields. The CITRIX interface also provides the formatting for the daily paint log information which is entered into a separate database. The Task Force spent time outlining the components of a potential Pavement Marking Management System. A focus was placed on existing and future inventory information consisting of pavement marking, pavement condition, and operations. The operations database does not exist and would represent factors such as the difficulty for crews to place markings in certain areas, heavy weaving or turning areas, areas requiring significant traffic control or night-time operations. The pavement condition data already exist from the DOT pavement management system, and it was shown how this can be merged with marking data.

Collecting data strictly on a route and milepost basis creates a number of problems in interpreting the information given concurrent routes and GIS issues at county borders. This effort identified alternative tools to locate segments for paint or reflectivity readings along with the tracking of durable markings. The Task Force examined pavement marking data input and developed a common listing of data input items.

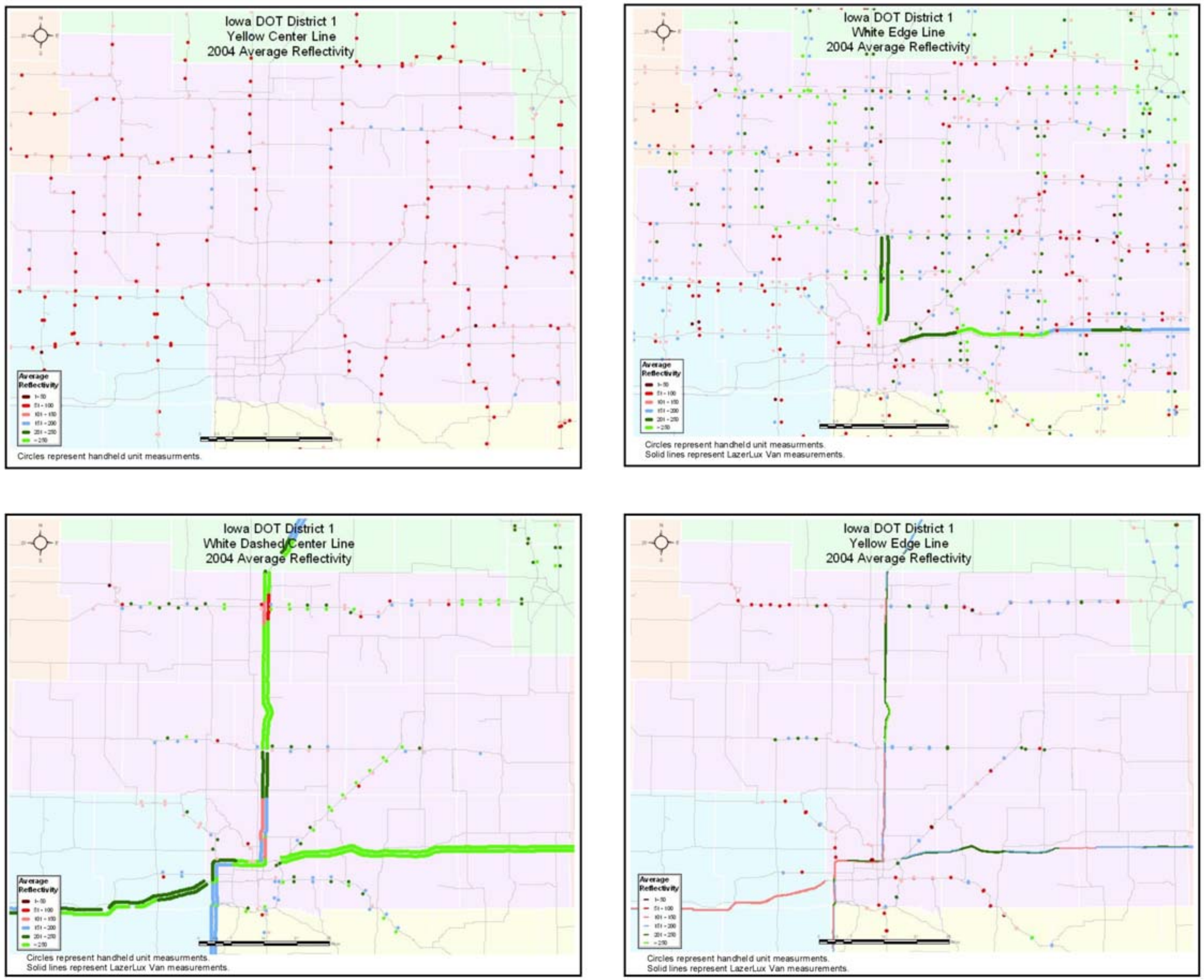


Figure 9. GIS plots for 2004 reflectivity data showing four types of lines

Field Test

The Task Force spent considerable effort in beginning a 3-year test along Highways 5 and 65 within the Des Moines metro (which is the only known test of its size and quality nationwide) to evaluate two types of durable waterborne paints and glass beads. Since the materials were put down using DOT crews, this demonstration has already provided valuable knowledge regarding how to install these new products. A picture of painting during the field demonstration is shown in Figure 10. The reflectivity results to date have shown very good results with expectations that these materials will continue to evolve into providing 3 seasons of service life. The task force is also considering how to groove pavement as part of initial construction to accommodate recessing of the pavement markings.

Figure 10. Painting white edge line during the Highways 5 and 65 field demonstration

Tracking Performance

In 2004, the Iowa DOT was just beginning to collect information when markings were installed and when they will be measured in the future. A comparison of good performance information was not available for this initial phase of this study.

Objective 3—Application Matrix

Development of an Application Matrix

The Task Force developed a Longitudinal Pavement Marking application matrix based upon meeting drivers needs, consideration of roadway type, pavement service life, the performance of materials, and cost. This initial matrix shown in Figure 11 reflects the fact that very little historic information exists today to track material performance over a range of conditions on DOT roadways. However, this information can be collected and used to consider modifications to the application matrix over time.

LONGITUDINAL PAVEMENT MARKINGS					
Remaining Pavement Surface Life	Primary 2 & 3 - Lane	Primary 4+ - Lane		Interstate	
	RURAL + URBAN ≤ 55 mph	RURAL	URBAN High Traffic	< 50,000 ADT	> 50,000 ADT
	7,811 centerline miles (75% of total)	1,059 centerline miles (10% of total)	556 centerline miles (5% of total)	1,000 centerline miles (9.6% of total)	45 centerline miles (0.4% of total)
≤ 2 yrs	**Waterborne**	**Waterborne**	**Durable Waterborne**, Waterborne,	**Waterborne**	**Durable Waterborne,** Waterborne
3 - 5 yrs	**Durable Waterborne**, Waterborne	**Durable Waterborne,** Waterborne	**Durable Waterborne,** Waterborne, Epoxy, Polyurea	**Durable Waterborne**	**Durable Waterborne**$^{E\&R}$ Polyurea$^{E\&R}$ Epoxy$^{E\&R}$
5^{+} yrs	**Durable Waterborne**, Waterbourne, Epoxy, Polyurea	**Durable Waterborne**, Waterbourne, Epoxy, Polyurea	*Durable Waterborne*^{E} *Epoxy, *Polyurea, *Tape	**Durable Waterborne**, Epoxy, Polyurea	**Tape**$^{E\&R}$ Durable Waterborne$^{E\&R}$ Epoxy$^{E\&R}$ Polyurea$^{E\&R}$

E = Enhancements could include reflectorized rpm's, wider markings, supplemental strips of wet reflective tape, roadway lighting, larger beads, paint additives, or other forms of enhanced illumination.
R = Recessed marking within a groove which is milled into the driving surface.
* = Where the characteristics such as heavy volumes, weaving, high speeds, or other conditions exist, markings within this category may be treated similar to Interstate Urban with > 5 years of life.

Figure 11. Longitudinal Pavement Marking application matrix

CONCLUSIONS

This research provided a significant investigation into existing Iowa DOT pavement marking practices, evaluation of potential new durable waterborne paint and bead combinations, development of a pavement marking application matrix, and short and long-term recommendations for implementation and operation of a Pavement Marking Management System for the Iowa DOT. Future phases currently underway include the final implementation and operations tasks. The work of the Iowa DOT Pavement Marking Task Force has made a significant contribution to DOT paint practices in terms of tracking performance, quality, and management of retroreflectivity on a statewide basis.

Waiting for Change

The Introduction of Innovation to Transportation Technology

Address to the Mid-Continent Transportation Research Symposium
Ames, Iowa
August 18, 2005

Neil F. Hawks
Transportation Research Board

Introduction

Early in 2005, the TRB Superpave Committee declared victory in the 12-year campaign to implement the results of the asphalt research findings from the Strategic Highway Research Program. These findings were made publicly available in 1993 under the rubric of "Superpave" and by early 2005 every state department of transportation reported in a committee survey that the specifications derived from the Superpave research had been adopted or plans were in place for adoption in the near future. Apparently a successful conclusion had been reached and the declaration of "victory" was warranted.

But doesn't 12 years seem like a long wait to see the fruits of research in the orchards of everyday practice? And the victory is really far from complete. Use of the Superpave standards among county and local agencies, which exercise care and custody of the bulk of asphalt roadways in America, is limited to 20 states. And in those 20, local application of Superpave is far from universal.

Why do we have to wait so long for innovation? What can be done to shorten the wait? Before exploring those questions, let's review two stories from Iowa highway history.

Slipform Paving: Born in Iowa

Ames, Iowa is the birthplace of slipform concrete paving. The engineering principles were first conceived in 1946 by Jim Allen and Bert Myers of the Iowa State Highway Commission Materials and Testing Laboratory and in 1947, Allen and Myers rolled out a prototype slipform pavement that was 3 in. thick and 18 in. wide. The following year they cast a sidewalk at the Highway Commission Lab that was 6 in. thick by 3ft. wide. In 1949, 2 experimental pavements were placed in O'Brien and Cerro counties, Iowa. These were real roads with two adjacent lanes, each 10ft. wide.

After that, progress seemed to halt. Actually it was only hidden from view. The Highway Commission continued its research; a self-propelled paver was developed for instance, but it became evident to the Commission that it could not, by itself, develop practical, commercial-scale slipform paver. At some point, the Quad City Construction Co. became involved and, in 1955, that firm introduced the first commercial slipform paving equipment.

In 1956, slipform paving moved out of Iowa when the Colorado Division of Highways approved the first instance of slip-form paving on an Interstate Highway. By 1964, 4,000 centerline miles of slipformed pavement had been placed in the US. An impressive number when viewed in the abstract, but most of those 4,000 miles were confined to only four states—Iowa, Colorado, Oklahoma and California. Furthermore, 75% of those 4000 miles were on secondary highways. Only a few hundred miles were placed on Interstate Highways, despite Colorado's pioneering efforts—and this at the time when the Interstate Highway construction boom was in full swing across the country.

As early as 1958, the Oklahoma Turnpike Commission had demonstrated that slipform paving had reduced the cost of concrete pavement by 35 to 45 cents per square yard, a substantial savings at that time. The capital cost of a slipform paving train was only about 50% that of more conventional equipment and yielded a better riding pavement with reduced construction time. Better – faster – cheaper, the highway engineer's trifecta.

Even so, traditional formwork paving was not displaced from regular practice until 30 or more years after Johnson's and Myers' first real highway paving project. A very long time to wait.

Anson Marston, Chief MacDonald and the Mud Roads Platform

Every engineer in Iowa has heard the name Anson Marston. In his day, the early years of the 20th century, he was Dean of Engineering at Iowa State University, one of the founders and first chairman of the Highway Research Board, today's TRB, and the first Chief Engineer of the Iowa State Highway Commission. Upon assuming this latter role in 1904, he remarked that Iowa's state roads, none of which was paved, would be a disgrace to even a barbarian. In 1913, Marston was succeeded as Chief Engineer by his prize student, Thomas MacDonald, arguably the best highway engineer ever, and, ultimately, the father of today's Interstate Highway System. Thus was Iowa blessed. Yet, when Chief MacDonald left the Commission in 1919 to join the U.S. Bureau of Public Roads, the Iowa state highway system was still one of dirt roads.

In 1916, William L. Harding was elected Governor of Iowa by campaigning against a bond issue that would have raised money to pave state highways. Author Pete Davies called this the "mud roads platform." On Election Day in 1919, however, the citizens of Iowa changed their minds and approved a bond issue for state road paving. By 1925 the Lincoln Highway, today's U.S. 30, was paved across Iowa from the Mississippi River to the Missouri River.

What happened in 1919 that allowed "good roads" to trump "mud roads?" Probably many things. A successful conclusion of World War I released pent-up economic demand. More and more citizens were becoming motorists-- frustrated motorists trapped by bad weather and bad roads. Returning servicemen needed jobs. The crystallizing event, however, was the passage that summer through Iowa, and Ames, of the Trans-Continental Military Convoy. This convoy of more that 80 U.S. Army trucks traveled from Washington, DC to San Francisco to demonstrate the strategic feasibility of coast to coast military transportation. What was really demonstrated was the parlous state of American roads, as the convoy took almost 3 months to complete its journey. The message was not lost on Iowa and the many other states and communities that authorized road bond issues in 1919.

Lessons

These anecdotes from Iowa history might be entertaining, but they are also instructive about the nature of innovation in public works. A number of points standout.

1. *Technology R&D takes time—lots of time.*
Visionary technologist Buckminster Fuller once remarked that there is no instant baby. While the 9 years from 1946 to 1955 is much longer than 9 months, to bring a complex technology from conception to commercialization in so short a time is flat out amazing. Nonetheless, this R&D project required great patience on the part of the Iowa State Highway Commission, which no doubt had many other post-war demands to meet. This is a lesson for all of us. It is gratifying to see a growing emphasis on longer-term, more conceptual research at the federal level. While we can't ignore the necessity to use research to solve immediate problems, such research is not the pathway to innovation. Johnson and Myers had the right idea. So did the Highway Commission.

2. *Public – private partnership in technology development pays off.*
The circumstances by which the Quad City Construction Co. became involved in the development of slipform paving are difficult to discern, but clearly this involvement turned a research project into technology. Road building in America has always been a public – private partnership so it's logical that the partnership should extend to R&D. In reality, however, barriers have been erected that inhibit such collaboration. Nevertheless, recent years have seen a growing trend toward active private stakeholder involvement in the planning and oversight of national research programs. An example of this is the recently released Concrete Pavement Research Road Map developed by Iowa State University on behalf of the Federal Highway Administration. The guidance and participation of many private sector experts and stakeholders greatly enriched the vision of where concrete pavement technology needs to go and outlined the path to get there.

   Direct involvement of individual private firms in government sponsored R&D is a trickier situation. Intellectual property rights must be observed, but unfair competitive advantage for any one entity is not a desirable outcome of public works research. In recent years, partnering devices have been introduced that allow access to private sector expertise without compromising marketplace freedom. The test equipment that enables the Superpave system was developed through such partnering devices.

3. *State by state diffusion is the slow way to do technology transfer.* Yet it is still the most common method. It is a myth that an obviously better technology will be readily adopted by practitioners without the need for an elaborate technology transfer effort. Being better, faster and cheaper was not enough to lead to the rapid introduction of slipform paving. In the absence of an organized technology transfer effort, each state had to repeat the same painful learning experiences of states that had already implemented the technology. In 1959 California became the fourth state to seriously adopt slipform paving. Initially the highway department struggled mightily to determine how long the sliding forms on the paver should be and repeated mistakes that Iowa, Oklahoma and Colorado had already made and corrected. Thus is the introduction of innovation slowed.

In 1993, AASHTO, FHWA and TRB initiated an organized effort to introduce the technologies developed as part of the Strategic Highway Research Program. Many novel technology transfer devices were tried to accelerate the pace of innovation. Not all of these devices worked, but one that did was the Lead States Program. As any new technology is introduced, there are always some agencies that move to the lead in adoption, much like the highway agencies of Iowa, Colorado and Oklahoma did in the adoption of slipform paving. Recognizing this, AASHTO organized the early leaders in the adoption of the various SHRP technologies into teams of “Lead States.”

These teams, comprising state DOT staff members knowledgeable in the application of the SHRP technologies, then shared there expertise with other agencies that had not yet tried the technologies. The teams engaged in both general tech transfer with all state DOTs and specific training for DOTs actively adopting the new technologies. Iowa was a Lead State for Anti-Icing/Roadway Weather Information Systems and for Innovative Pavement Maintenance Materials. Iowa’s Lee Smithson was the Team Coordinator for the latter team.

How much more quickly might slipform paving have been adopted if Iowa, Colorado and Oklahoma had been organized into a Lead States Team in 1958?

4. *Introduction of technological innovation is not the logical extension of research.* People cannot leave their past experience behind. On any topic of consequence, whether it’s the best way to build concrete roads or the value of good roads as public policy, if most people think about it at all, they have already made up their minds. Introduction of innovation, then, requires the changing of men’s minds. In other words, it requires effective communications. The findings of the research must be the core of the message, but it’s equally important that the message is listened to. And this is the lesson to be drawn from the Transcontinental Military Convoy and the final triumph of good roads over mud roads in Iowa.

While the convoy might have had an underlying national security objective, it was also a hugely successful public relations campaign on behalf of better roads. Every time the convoy bogged down in mud or sand, had to repair or replace a bridge before it could cross a creek, it was widely reported. By the time the convoy reached San

> Francisco everyone in America must have known just how bad America's roads really were. The convoy was supported and accompanied by car and truck manufacturers, tire and oil companies, was met and feted by local businessmen and civic officials and good roads advocates. At almost all of the overnight stops the convoy made, there were community barbecues, open-air dances and speeches from good roads advocates delivering the facts and figures. When Iowans and other Americans went to the polls in 1919, they passed more than $200,000,000 in state and local bond authorizations, a staggering amount in those days. So staggering, in fact, it took highway agencies and the construction industry years to get organized to spend it all. The following year Congress passed a federal aid highway bill that defined the federal-state partnership that continues today. Clearly the message was listened to. Iowa and America had had enough of mud roads.

Lessons Learned?

What is being done today within the transportation community to drive innovation and change men's minds? Have the lessons of the past improved the way we introduce innovations into engineering practice? The short answer is yes, the lessons have been learned. The transportation community is more willing to attempt long-term research than it was 20 years ago. Technology transfer today is much more sophisticated in approach than it once was. Private sector participation in the planning and conduct of federally sponsored research is now commonplace. Communications has been recognized as a vital part of the tech transfer process. One example will illustrate how things have changed.

With the coming of the new millennium, AASHTO created the Technology Implementation Group (TIG). The TIG, whose members are all senior officials of state DOTs, has as its purpose the identification and promotion of promising, but under-utilized technologies. The TIG also has liaison members from FHWA, TRB, the National Association of County Engineers and the American Road and Transportation Builders Association. If the TIG had existed in 1958, one of the technologies picked for promotion would undoubtedly have been slipform concrete paving.

The TIG prosecutes the implementation of these under-utilized technologies through the use of Lead States teams similar to those used in the deployment of SHRP technologies. Frequently these teams include private sector experts in addition to the state DOT personnel familiar with the technology. The teams are provided staff support by AASHTO and FHWA. In addition, AASHTO has contracted with a communications firm to make sure the necessary communications skills are available to TIG and the Lead States teams.

All of this is supported by a pooled fund subscribed to by the individual state DOTs.

Currently, TIG is supporting the aggressive implementation of a dozen innovative technologies and procedures that range from pre-cast bridge elements and accelerated construction to road safety audits and pre-stressed cable median barriers. Three more technologies will be selected this fall.

Further Reading

This short paper captures informal remarks made on August 18th, 2005 at the Mid-Continent Transportation Research Symposium in Ames, Iowa. The paper attempts to maintain the informal tenor of the luncheon remarks and detailed references have not been provided. The bulk of the factual information was drawn from two books and a website. These were:

Davies, Pete. *American Road: The Story of an Epic Transcontinental Journey at the Dawn of the Motor Age*. Henry Holt and Co., New York, NY, USA, 2002.

Highway Research Board. *Highway Research Record Number 98: Symposium on Slip-Form Paving*. National Academy of Sciences, Washington, DC, 1965

American Concrete Pavement Association. *History in Highways: 100 Years of Innovation.*
http://www.pavement.com/pavtech/abtconc/history.html

Using Visualization to See Transportation Solutions

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ABSTRACT

Visualization is a powerful new tool to help transportation engineers develop support for design and construction projects that are critical to improving the public infrastructure.

Using visualization technology from the movie and video game industries, transportation engineers can create images of engineering projects that range from relatively simple 2-D photo simulations to complex 3-D animations.

While the excellence of these images is sufficient for photo quality, their value lies in comprehension. URS transportation engineers have worked with their clients to use animations to get preliminary layout approval for complex projects. The projects range from seeing how Bus Rapid Transit (BRT) systems work to showing traffic modeling, and to explaining how High Occupancy Toll (HOT) Lane works. In each case when constituents see visualizations of engineering projects, they have a better understanding of the project.

The value of visualization to transportation engineers goes beyond the public presentations into design and construction. By creating a 3-D model of a project, engineers can easily spot and correct flaws during the design stage. Then during construction, the 3-D model of the project can be used for “machine control” to guide field equipment and dramatically speed up the construction phase.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: synergy—transportation—visualization

Analyzing Severe Injury Risk for Crashes Nationally and Within Iowa

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ABSTRACT

Two logistic regression models were used in the comparison of Iowa and U.S. crashes. Of particular interest is the risk of severe injury for male and female drivers within three age groups: middle-age (35-54), older (55-74), and elderly (75 and older). Within Iowa, male drivers have a higher risk of severe injury than female drivers (Odds Ratio = 1.15). However, nationally, male drivers have a lower risk of severe injury than female drivers (Odds Ratio = 0.85). Iowa drivers between the ages of 55 and 74 have the highest risk of severe injury, whereas nationally, drivers over the age of 75 have a greater risk. The differences in gender and age effects are discussed. Among the issues considered are the role of seatbelts in reducing injury and factors related to changes in driving patterns among older and elderly drivers.

Key words: injury severity—logistic regression—older drivers

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INTRODUCTION

According to a 2002 report on older and elderly drivers in Iowa (Thompson, Pawlovich, and Triggs 2002), senior citizens account for nearly 20 percent of Iowa traffic deaths. Intersection accidents are reported to be of particular concern, especially for drivers over the age of 75. These results are consistent with studies of drivers nationally. In the year 2000, the National Highway Traffic Safety Administration (NHTSA) reported that older individuals accounted for 6 percent of traffic crash injuries and 13 percent of traffic fatalities (2001). This increased risk of injury exists despite greater seatbelt usage rates and the lower levels of intoxication (Hakamies-Blomqvist 1993). Evans (1991) showed that drivers over the age of 70 were at much greater risk for fatality than drivers at the age of 20 in a similar crash. The risk of severe injuries and fatality increases with age for both side impact crashes (Austin and Faigin 2003), and frontal impacts (Mercier, Shelley, Rimkus, and Mercier 1997).

Gender has also been shown to be a significant factor in the prediction of injury severity. In a study of Iowa crashes, Khattak et al. (2002) showed that older male drivers experience more severe injuries than older female drivers. However, in a study of national crashes, Dischinger (1996) showed that women of all ages were more susceptible to crash related injuries. Baker et al. (2003) also showed that older women have an increased risk of fatality in a crash, particularly in favorable driving conditions. This discrepancy in gender effect points to possible differences in risk factors between drivers in Iowa and the US as a whole.

In order to effectively assess the roles of gender and age on crash injury risk, it is necessary to adjust for any other factors that may differentiate Iowa from other regions of the country. Among these are age and population density. According to the 2000 US Census, Iowa ranks second in the nation in the proportion of people 85 years of age and older and fourth in the nation in proportion of adults older than 65 years of age (Iowa 2001). In terms of population density, Iowa is also a very rural state. Nationally, only 21 percent of US residents live in rural areas, but in the state of Iowa, rural residents account for almost 39 percent of the state's population (US 2000).

The goal of this study was to investigate the incidence of severe injury for crashes in the state of Iowa. Separate regression models were developed based on data from the Iowa state crash database and a national crash database. These databases include information on the severity of injury, as well as biographical information of drivers, vehicle information, and information regarding the nature of the crash. The focus of this study is to evaluate the overall severity of injury and the biographical, vehicular, and environmental factors that may have contributed to the level of severity.

METHODS

Data Sources

The primary data source for this study was the Iowa State Crash Database from the year 2000. This is a comprehensive collection of all vehicle crashes in the state of Iowa in which a minimum level of property damage was sustained regardless of injury. The Iowa crash database contained information on 115,812 injured and uninjured occupants involved in crashes in Iowa in the year 2000. The General Estimates System (GES) was used for purposes of comparing Iowa and national crash injury risks. This dataset is a stratified sample of crashes from 27 sampling units across the United States, containing injury data for over 114,000 cases. After weighting, 12,647,821 occupants involved in motor vehicle crashes in the United States during the year 2000 were represented. Only drivers were included in this analysis since the

Iowa crash database did not report demographic information such as age and gender for uninjured passengers. This information was available for drivers involved in injury-free crashes.

Model Description

The goal of this study is to predict injury severity. The injury codes for both databases are based on a five-point scale: no injury, possible injury, non-incapacitating injury, incapacitating injury, and fatality. The data was segmented into two major categories of severity. Incapacitating injuries and fatalities were considered severe injuries, while the other three categories were considered not severe.

Logistic regression models were used because of the categorical nature of the dependent variable severe injury (yes or no). The predictive models are chosen based on factors shown to be important in literature and other research on occupant injuries. The following dependent variables were included in the models: gender, age, location (rural or urban), type of collision (head on, rear end, or side impact), presence of adverse weather conditions, and vehicle type (car or light truck/van/sport utility vehicle). These variables were considered likely to be significant based on previous studies (Baker et al. 2003; Lefler and Gabler 2004; McGwin and Brown 1999) and are largely independent. Additionally, each of these variables could be either directly obtained from or derived from both databases.

Each of the listed variables is binary (present or not present as a condition of the crash) except age, which was segmented into 3 categorical variables (35-54, 55-74, and 75 and older), which are consistent with other similar studies (Cooper 1990; McGwin and Brown 1999). First-order interactions between gender, age, and various crash conditions were also considered in the model. The final model contains only those variables found to be statistically significant in predicting injury severity.

The logistic model is set up as follows:

$$\ln\left(\frac{p}{1-p}\right) = \beta_o + \mathbf{\beta X} + \varepsilon \qquad (1)$$

where p is the probability that a serious injury will occur in a crash (yes or no), β_0 is the intercept, $\boldsymbol{\beta}$ is the matrix of coefficient estimates for each respective predictor variable (e.g., seat track position, gender), and ε is the error (normally, and independently distributed) associated with parameters not included in the model. The coefficient β can be used to generate the relative risk of crash injury severity given the existence of some condition. Using a logistic regression model to calculate odds ratios is a powerful tool because it allows for the evaluation of many possible conditions using a single model. Additional information regarding logistic regression models can be found in Washington et al. (2003), Fox (1997), and Pampel (2000).

RESULTS

Table 1 shows the distribution of Iowa drivers involved in vehicle crashes in the year 2000 by age and gender. The distribution by age for males and females is similar. The age groups were chosen to represent three distinct populations: middle-age, older, and elderly adults. Younger drivers are not shown because this study focuses primarily on older drivers. Studies have shown that younger drivers are clearly at an increased risk for crashes (Arnett 2002). As seen in Table 1, there were more vehicle occupants in the middle-age group than in the two older age groups. However, there is still a substantive number of elderly drivers (n=5317).

Table 1. Number of male and female Iowa drivers by age in crashes

	Male		Female	
Age Group	Count	Percent Distribution	Count	Percent Distribution
35 to 54	17742	74.71%	14598	75.35%
55 to 74	2996	12.62%	2470	12.75%
75 and Older	3011	12.68%	2306	11.90%
Total	23749	100.00%	19374	100.00%

The results of the logistic regression model are shown in Table 2. Female drivers are less likely to be severely injured as a group. Age was also shown to be significant, with drivers between age 55 and 74 having the greatest risk of severe injury. While drivers 75 and older are less likely to be severely injured as a group, females in this age range are more likely to be severely injured than their male counterparts. Rural collisions were associated with a greater risk of severe injury. Head on collisions and side impacts also resulted in a greater likelihood of severe injuries, whereas rear impacts were less likely to have the same result.

Weather conditions and vehicle type were not shown to be significant in predicting injury severity for Iowa crashes. Additionally, the only interaction effect was between gender and the elderly age group. Females in the oldest age group showed a greater increase in risk as compared with elderly males. This suggests that any difference in injury risk between male and female drivers is consistent across collision types, rural and urban environments, and drivers less than 75 years of age.

For comparison, a similar model was created based on national crash data. The results of the model are shown in Table 3. The only additional parameter evaluated was seatbelt use. This was included because of the strong impact that seatbelts have on driver safety. There is also a significant difference in seatbelt usage among driver populations. Within the national database, seatbelt usage is lower for men than women ($\chi^2(1)=294.03$, $p<0.01$). This variable was not uniquely identified in the 2000 Iowa crash database. Unlike in the Iowa model, both adverse weather and vehicle type were significant in the national model. Additional interactions reported in Table 3 were significant in the national model, but not significant in the Iowa model.

Table 2. Logistic Regression Parameter Estimates-suffering severe injury in a crash (Iowa Drivers)

Variable	Parameter Estimates	p-value
Constant	-3.7176	< 0.0001
Female	-0.1362	< 0.0001
Age 35 to 54	0.1493	< 0.0001
Age 55 to 74	1.2634	< 0.0001
Age 75 and Older	-0.3398	< 0.0001
Crash in Rural Area	0.8904	< 0.0001
Rear End Collision	-0.5429	< 0.0001
Head On Collision	1.5536	< 0.0001
Side Impact Collision	0.4141	< 0.0001
Interaction: Female age 75 and Older	0.7801	< 0.0001
Log-likelihood at zero		37694.2
Log-likelihood at convergence		35205.9
Number of Observations		112815

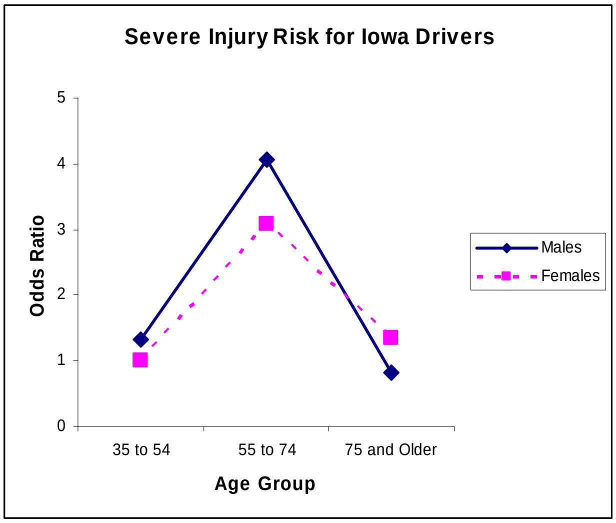
Nationally, females were more likely to be severely injured than males. Increases in age were also associated with higher risk of severe injury, with the largest increase for those drivers over the age of 75. Again, rural crashes and head on collisions were more likely to result in severe injury, whereas these injuries were less likely to occur in adverse weather and to drivers of light trucks, vans, and sport utility vehicles. Interactions between gender and age did not change the trend of increasing injury risk with age for either gender. Females were slightly more likely to be severely injured in bad weather and slightly less likely to be severely injured in rural crashes than males, but the general trend reported in Table 2, with regard to adverse weather and rural crashes, held for both genders.

Table 3. Logistic Regression Parameter Estimates-suffering severe injury in a crash (US drivers)

Variable	Parameter Estimates	p-value
Constant	-2.0059	< 0.0001
Female	0.1632	< 0.0001
Age 35 to 54	0.0744	< 0.0001
Age 55 to 74	0.1950	< 0.0001
Age 75 and Older	0.7285	< 0.0001
Crash in Rural Area	0.7603	< 0.0001
Rear End Collision	-1.0025	< 0.0001
Head On Collision	0.8619	< 0.0001
Side Impact Collision	-0.3238	< 0.0001
Seatbelt Use	-1.9775	< 0.0001
Adverse Weather	-0.2295	< 0.0001
Light Truck, Van, Utility Vehicle	-0.2006	< 0.0001
Interaction: Female age 35-54	-0.0407	< 0.0001
Interaction: Female age 55-74	0.1108	< 0.0001
Interaction: Female age 75 and Older	-0.3012	< 0.0001
Interaction: Female wearing Seatbelt	0.1738	< 0.0001
Interaction: Female in Adverse Weather	0.0387	0.0029
Interaction: Female in a rural Crash	-0.0702	< 0.0001
Log-likelihood at zero		2144012.3
Log-likelihood at convergence		1941430.4
Number of Observations		9246898

With regards to gender, a significant difference is shown between Iowa crashes and crashes nationally. Almost all literature about gender effects, specifically increased injury risk to females, is based on national data. The Iowa data, however, indicates that females are at a decreased risk of severe injury. Additionally, differences exist between the models with regard to age effects. The national model shows a consistent increase in risk with age, whereas the Iowa model shows a significant decrease in risk for those drivers over the age of 75.

Figures 1 and 2 show changes in severe injury risk for drivers in Iowa and nationally. There is very little difference in gender among 35- to 54-year-olds in either model, or for those drivers 75 and older nationally. The difference in female injury risk between the national and state models is primarily a function of the 55 to 74 year age group. Additionally, for the Iowa model, the gender effect is actually inverted between the 55 to 74 age group and the over 75 and older group. To further investigate this, the relative difference in risk between age groups for each population will also be considered. Identifying consistencies (or the lack thereof) in changes in risk may be more useful in determining where the true differences lie between the state and national model results.



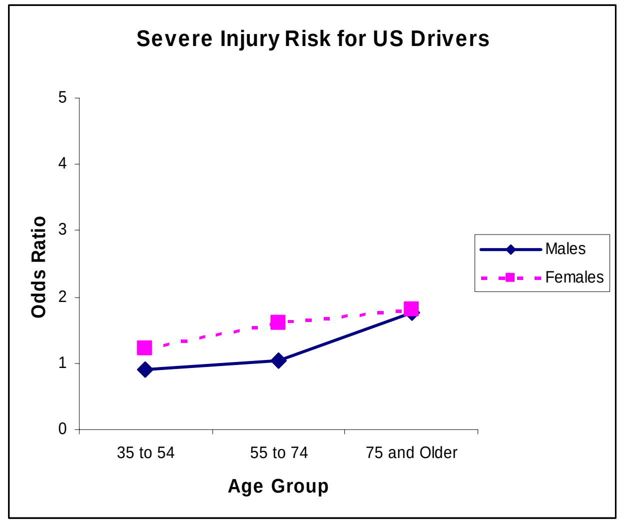


Figure 1. Odds of severe injury for Iowa drivers by age and gender

Figure 2. Odds of severe injury for U.S. drivers by age and gender

In all cases, the risk of severe injury increases for drivers in the older age group (55-74), as compared with the middle-age group (35-54). Nationally, females are at a greater risk of severe injury in both groups. Within Iowa, males are at greater risk. In comparing the two models, only the direction of change in injury risk (increase or decrease) should be considered, rather than the magnitude of the change. An increasing risk of severe injury between middle-age and older drivers is apparent in both models. The difference in risk between older drivers (55-74) and elderly drivers (75 and older) is more complicated. Within Iowa, the risk of severe injury decreases, whereas nationally, this risk increases. Among Iowa drivers, this is the one age group where females are at greater risk of severe injuries than males. At the same time, nationally, males in this group have a similar risk of severe injury as females.

The lack of information concerning seatbelt use in the State of Iowa database also must be evaluated when comparing the two models. Data for more recent years is currently being complied and will include more specific information on seatbelt use. In the future, this data should be analyzed to determine if males are actually at higher risk for severe injury and if older drivers are at a lower risk for severe injury, or if this is an artifact of the effect of seatbelt usage rates. In the absence of this data, another means of assessing this issue is to evaluate the national data without considering seatbelt usage to see if the national risk profiles without considering seatbelt use represent the state results in Figure 1 more closely. A comparison of national models with and without considering seatbelts explicitly is shown in Figure 3.

The solid lines in Figure 3 represent the baseline national model including seatbelt use as a parameter. This is the same data as shown in Figure 2. The dashed lines represent the same national model without considering seatbelt usage. If difference in seatbelt usage rates was the primary cause for the discrepancy in gender effects seen in Figure 1, then it would be expected that the males nationally would have a greater risk when seatbelt usage is not included in the model. However, as seen in Figure 3, females have a greater reported risk of severe injury regardless of whether or not seatbelts are considered in the national model. Additionally, if failure to consider seatbelt usage was responsible for the apparent decrease in risk for the elderly in Iowa, one would expect significant decreases in injury risk for the elderly nationally, when seatbelt usage is ignored. However, the overall risk increases with age regardless of whether or not seatbelt usage is considered nationally. Additionally, neither national model (belt usage known or unknown) shows the same crossover effect between males and females in the oldest two age groups.

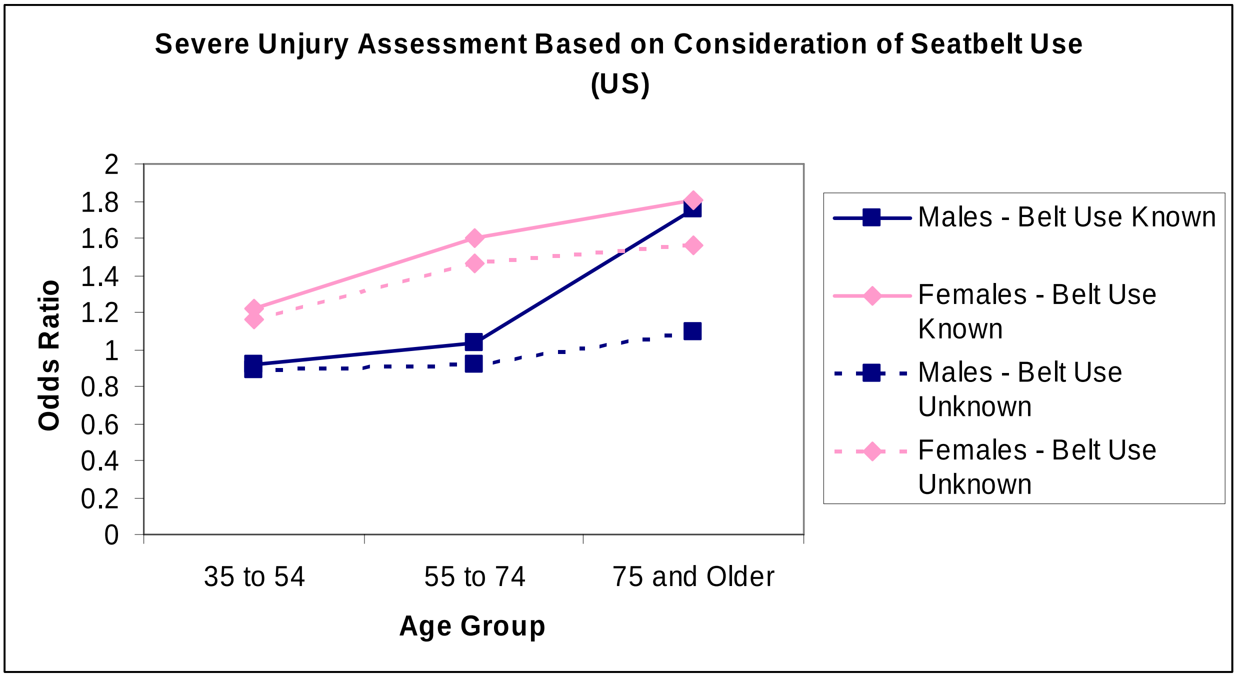


Figure 3. A comparison of severe injury risk based on the explicit consideration of seatbelt use in the logistic regression model

DISCUSSION

The investigation of severe injuries in crashes in the state of Iowa has led to some interesting findings. The first finding is that among middle-age and older drivers, males in Iowa have a greater risk of severe injuries than females. This is in contrast to trends seen nationally. One possible answer to the first question regarding the increased risk of injury for males in Iowa is that it is a function of the model parameters. The Iowa database from 2000 does not provide information that can be used to ascertain seatbelt usage. Since the national model took this into account, but the state model did not, the increased risk of males in the state model may be masking the effect of a lower seatbelt usage rate among males, particularly since the national model showed such a significant increase in the odds of injury for unbelted occupants (odds ratio = 7.2). Examination of the national model with regard to considering seatbelt usage indicates this is not a sufficient explanation, and other factors are likely involved. This may suggest a greater disparity in high-risk driving habits between males and females in Iowa than nationally.

The second finding was that injury risk for male and female drivers decreases after age 75 within Iowa, while increasing nationally. Additionally, elderly male drivers in Iowa see a more dramatic decrease than females, whereas nationally, elderly males have an increasing risk relative to females. The decrease in risk for Iowa drivers over the age of 75 may also be influenced by the affect of seatbelt usage since studies have shown that older drivers wear seatbelts more regularly (Hakamies-Blomqvist 1993). This might also explain the decrease in risk for men relative to women among the elderly. If elderly drivers of both genders are consistently more likely to wear seatbelts, then this would negate the effect of seatbelt use on the risk associated with a particular gender.

Although including seatbelt usage as a parameter in the state model would certainly result in greater information about the injury risk for older and elderly drivers, it does not appear that the differences in injury risk between Iowa drivers and those nationally can be explained solely by considering belt use in the model. It is more likely that these differences are attributable to behavioral issues. While one of these issues could be seatbelt usage rates, which could be assessed in future studies with new state crash data, another possible explanation is related to modifications in driving behavior.

One such modification is driver self-regulation. Driver self-regulation involves adjusting driving patterns to avoid driving situations that are perceived as risky or dangerous. Several factors influence modification of driving patterns, including flexible schedules, less need to drive, and recognition of impairments which could impact safety (Ball et al. 1998). It is possible that elderly drivers in Iowa do a better job of adjusting driving patterns than those drivers nationally. This could involve avoiding rush hour traffic, highways, intersections, and trips outside of familiar areas. Additionally, older drivers may benefit from lower traffic levels and lack of highways in rural areas.

Further research is required in order to understand the role of seatbelts in preventing severe injuries as Iowa drivers age. Additionally, future studies pertaining to the manner in which Iowa drivers, particularly those in rural areas, adjust driving patterns are needed. This will help to clarify the root cause of the reduction in injury risk among elderly Iowa drivers. Once it is understood why elderly drivers (ages 75 and older) in Iowa have successfully reduced their risk of injury in a crash, recommendations can be made to reduce the risk of injury within the older driver population (ages 55-74).

ACKNOWLEDGMENTS

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Effects of Aggregate on Flow Properties of Mortar

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ABSTRACT

The effects of aggregate characteristics on flow ability of mortar mixtures were investigated. Two types, five single sizes, and three gradations of fine aggregate were considered. Uncompacted voids of the aggregates were measured. Flow properties of 166 mortar batches made with the aggregates, together with different sand-to-cement ratios (s/c) and water-to-cement ratios (w/c), were evaluated using the ASTM C109 flow table test method. The results indicated that w/c, uncompacted voids, size, and volume of aggregate have a great impact on mortar flow ability. Generally, aggregate having higher uncompacted void content provides mortar with a lower flow. When aggregate content was low (s/c=1), the size of aggregate had little effect on flow ability of the mortar; while aggregate content was high (s/c=3), the size of the aggregate significantly influenced the mortar flow ability. A statistical model was developed for predicting flow ability of mortar.

Key words: angularity—content—fine aggregate—flow—size

INTRODUCTION

Aggregate characteristics (such as size, gradation, shape, and surface textures), as well as its volume fraction, have a significant impact on workability, especially on flow ability, segregation resistance, and compactibility of concrete. Based on the concept of particle packing, well-graded (or well-packed) aggregate has fewer voids between particles than poorly-graded aggregate, and it requires less cement paste to fill the voids. Thus, additional amount of cement paste will coat the aggregate particles and improve the concrete flow. Research results from Smith and Collis (2001) have proved this concept. Differently, Quiroga (2003) recently found that the concrete mixtures optimized for maximum packing density produced poor workability and high susceptibility to segregation, which was possibly due to the reduced spacing and increased friction between aggregates. For the same reason, increase in volume fraction of aggregate in concrete generally results in a reduced flow (Szecsy 1997; Geiker et al. 2002). Crushed sands often require higher water content than natural riversands for given workability owing to their angularity and surface texture (Malhotra 1964; Geiker et al. 2002). However, because of the difficulties in quantitative characterization of aggregate, most existing research focuses on aggregate gradation and volume fraction, and limited work on other aggregate characteristics (such as shape and surface textures) has been done. In the present study, fine aggregate type, size, gradation, shape, and surface textures are all considered, and their effects on mortar flow properties are studied.

RESEARCH SIGNIFICANCE

Aggregate characteristics have significant effects on concrete workability and other properties. However, limited work has been conducted studying the effects systematically and quantitatively. Using the ASTM standard flow table test, this study provides a systematic study of effect of aggregate on mortar flow ability. The research results will provide engineer with an insight on concrete selection and mix design.

EXPERIMENTAL WORK

Materials

Type I Portland cement was used for all mortar samples. Riversand and limestone sand were used as fine aggregates. The specific gravity of the fine aggregates was 2.63 and 2.59 for riversand and 2.53 and 2.42 for limestone under the saturated surface dried (SSD) condition and oven dried (OD) condition, respectively. All aggregates were oven dried before use. Five single-sized aggregates (#8, #16, #30, #50, and #100) and three graded aggregates (G1, G2, and G3), as shown in Figure 1, were employed.

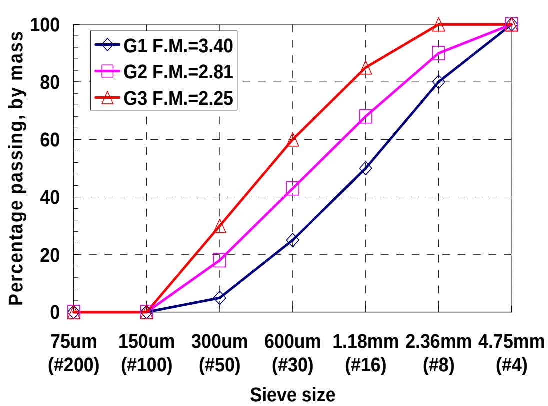


Figure 1. Gradation curve

Mixture Proportions

The mortar mix design is listed in Table 1. The design variables include aggregate types, sizes, gradations, sand-to-cement ratios (s/c), and water-to-cement ratios (w/c). The different w/c was selected to ensure that all mixtures could have a measurable flow. A total of 166 mortar batches were prepared.

Table 1. Mix design for mortar

s/c	Sand	w/c	
		Riversand	**Limestone**
3	#100	0.50, 0.53, 0.56, 0.62	0.37, 0.42, 0.47, 0.57
3	#50	0.44, 0.47, 0.50, 0.53, 0.55	0.32, 0.37, 0.42, 0.47, 0.52
3	#30	0.39, 0.42, 0.45, 0.48	0.35, 0.40, 0.45, 0.50
3	#16	0.33, 0.36, 0.39, 0.42, 0.45	0.24, 0.34, 0.44, 0.49
3	#8	0.27, 0.29, 0.32, 0.35, 0.38, 0.41	0.21, 0.26, 0.31, 0.36, 0.41
3	G1	0.30, 0.35, 0.40, 0.50	0.20, 0.25, 0.35, 0.45
3	G2	0.30, 0.35, 0.40, 0.50	0.20, 0.25, 0.35, 0.45
3	G3	0.33, 0.38, 0.43, 0.53	0.20, 0.25, 0.35, 0.45
2	#100	0.36, 0.41, 0.46	0.31, 0.36, 0.41
2	#50	0.34, 0.36, 0.41, 0.46	0.27, 0.32, 0.37
2	#30	0.30, 0.33, 0.35, 0.40, 0.45	0.28, 0.33, 0.38
2	#16	0.27, 0.30, 0.32, 0.37, 0.42	0.28, 0.33, 0.38
2	#8	0.25, 0.30, 0.35, 0.40	0.20, 0.25, 0.30
2	G1	0.33, 0.38, 0.43	0.29, 0.34, 0.39
2	G2	0.34, 0.39, 0.44	0.28, 0.33, 0.38
2	G3	0.36, 0.41, 0.46	0.28, 0.33, 0.38
1	#100	0.28, 0.32, 0.38, 0.48	0.26, 0.31, 0.41
1	#50	0.28, 0.32, 0.38, 0.48	0.21, 0.31, 0.41
1	#30	0.23, 0.27, 0.37, 0.37, 0.47	0.22, 0.32, 0.42
1	#16	0.21, 0.26, 0.36, 0.46	0.21, 0.31, 0.41
1	#8	0.20, 0.25, 0.35, 0.45	0.20, 0.30, 0.40
1	G1	0.37	0.32
1	G2	0.37	0.32
1	G3	0.38	0.32

Mixing Procedure

All mortar batches were mixed according to ASTM C305 “Standard practice for mechanical mixing of hydraulic cement pastes and mortars of plastic consistency.”

Test Method

Fine aggregate angularity

Particle shape and surface texture of aggregate were characterized with an angularity test based on ASTM C1252 “Standard test methods for uncompacted void content of fine aggregate (as influenced by particle shape, surface texture, and grading).” Fine aggregate angularity was defined as the percent of air voids in a loosely compacted fine aggregate. High void content indicates that the aggregate contains a large amount of fractured and irregular particles.

Flow measurement

Flowablility of mortar was measured using a flow table method modified from ASTM C230, "Standard Specification for Flow Table for Use in Tests of Hydraulic Cement." This ASTM test was originally designed to determine the water content needed for a cement paste sample to obtain a given flow spread of 110$\pm$5 mm. The test shall be performed with a standard flow table after the table drops 25 times. In the present research, a mortar sample was placed on the same flow table and subjected to 25 drops, and then the spread diameter of the sample was measured. The flowability of the mortar was expressed as a flow percentage—the percentage of the spread diameter over the original bottom diameter of the sample.

Since the standard flow table had a limited diameter, some mortars having high flow ability flowed out off the flow table when 25 drops were applied. To solve this problem, smaller number of drops was used for the wet samples. The spread diameters of these samples at 25 drops were predicted from a newly developed relationship between the number of the flow table drops and the spread diameter. To develop this relationship, a larger plate was attached on the standard flow table to increase the table surface area. A group of 40 samples were then tested on this modified flow table and their spread diameters were measured under various numbers of flow table drops.

TEST RESULTS AND DISCUSSIONS

Fine Aggregate Angularity

Figure 2 presents the uncompacted voids of all aggregates used. As shown in the figure, limestone sand had a higher percentage of uncompacted voids than riversand because of its irregular shape of the particles. Graded aggregate had lower void content than single-sized aggregate due to proper particle packing.

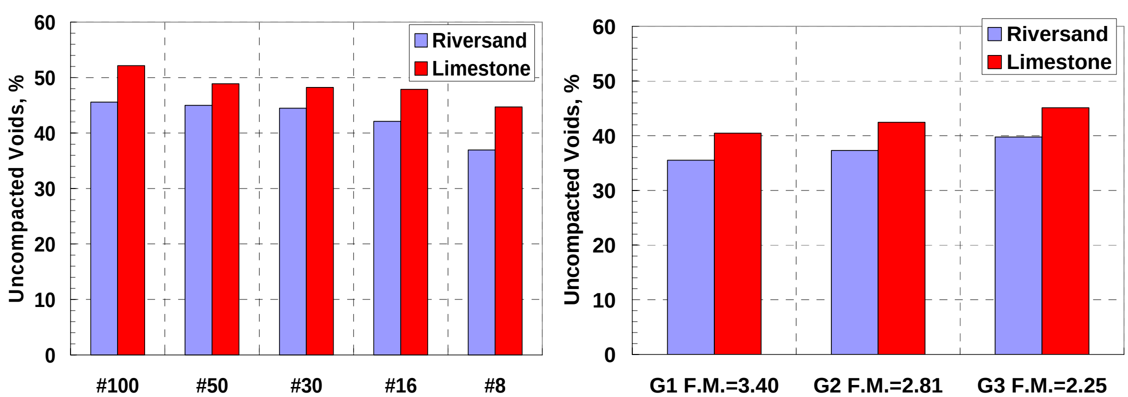


Figure 2. Fine aggregate angularities for different aggregate

Relationship Between Flow Table Drops and Sample Spread Diameter

Figure 3 demonstrates that the flow percentage had linear relationship with the number of flow table drops in logarithm scale. This relationship was used for prediction of the 25-drop flow percentage of the samples actually tested under less than 25 drops. It can be expressed as follows:

$$F_{25} = F_t + 46.779(\ln 25 - \ln t) \tag{1}$$

where t is the drop number of flow table, F_{25} is the flow percentage of mortar at 25 drops, and F_t is the flow percentage of mortar at t drops. The R square value of Equation (1) is 0.98, indicating that the predicted value is very close to the tested value.

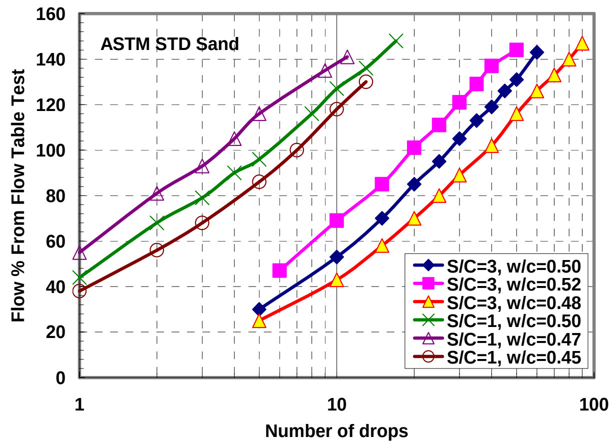


Figure 3. Number of drops versus flow percentage of flow table test

Flow Properties

Figure 5 shows the effects of aggregate size, w/c, and s/c on flow ability of mortar. For a given w/c, the higher s/c (or high aggregate content), the lower the mortar flow. For mixtures with a low s/c (Figure 5 (a)), the difference in mortar flow from different sizes of aggregate was not significant. However, this difference became clear when s/c increased (Figures 5(b) and 5(c)). For a given aggregate content, the mixtures with larger size of sand had a higher flow. Mortar with high w/c provided higher flow ability. Similar results were also found for mortar with limestone as fine aggregate.

Figure 6 shows that limestone provided mortar with lower flow ability comparing to riversand. This result is expected because the high percentage of the uncompacted voids in limestone required more cement paste to fill up the space between aggregate particles. As a result, less amount of cement paste was available to provide mortar with same flow.

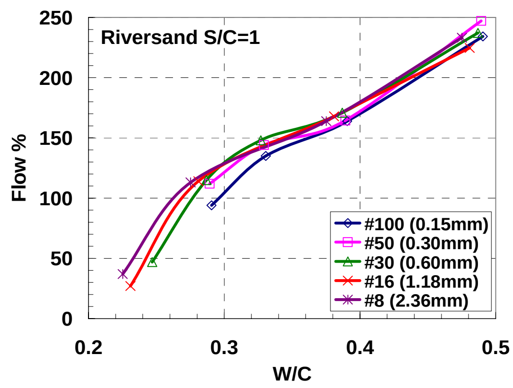
Figure 7 indicated that graded aggregate provided the mortar with a higher flow than single-sized aggregate for similar fineness modulus. It is because graded aggregate had low uncompacted void content and hence required less amount of cement paste to provide the same flow. Statistic analysis presented below confirmed this explanation.

Statistic analysis with linear regression was performed using selected parameters. According to the test results from 166 different mixes, the following equation was obtained:

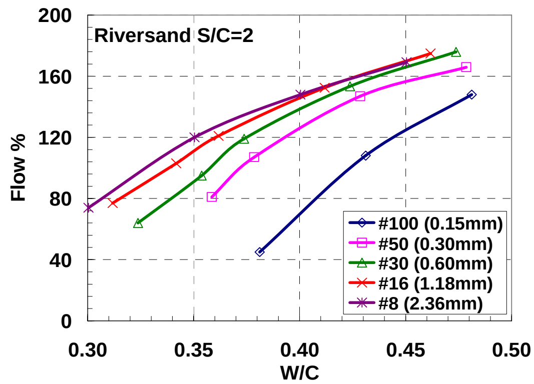
$$Flow\% = 348 - 3.84V_n + 391w/c - 4.59V_s + 12.27r \qquad (2)$$

where V_n is the uncompacted voids (%), r is the average aggregate size (mm), and V_s is the volume of sand (%). The R square value of Equation (2) is 0.79.

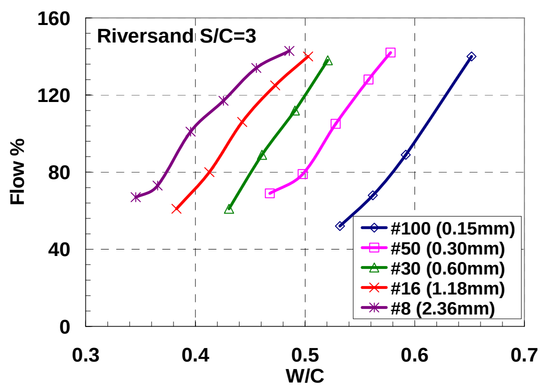
Equation (2) not only demonstrates the major factors that significantly affect mortar flow, but also can be used to predict or quantify mortar flowability based on aggregate characteristics and mix proportions.



(a)



(b)



(c)

Figure 5. Effect of aggregate size and s/c on mortar flow (riversand)

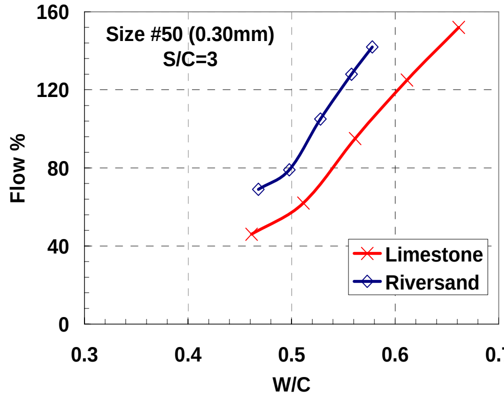


Figure 6. Effect of aggregate type

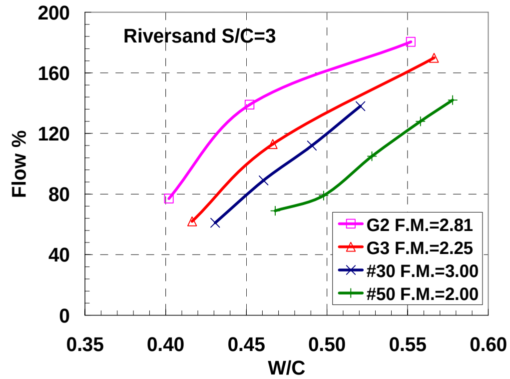


Figure 7. Effect of gradation

CONCLUSIONS

The following conclusions can be drawn from the present research:

1. Aggregate shape, surface texture, and gradation can be all characterized by the uncompacted voids between the aggregate particles. Generally, aggregate with higher uncompacted void content provides mortar with a lower flow.

2. In addition to w/c, uncompacted voids, size, and volume of aggregate are also significantly influence mortar flow ability. (Note that the effect of cement properties was not considered in the present study.) For mortar with a low s/c, the effect of aggregate size on the mortar flowability may not be significant, but the effect will become significant as s/c increases.

ACKNOWLEDGMENTS

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Providing Intelligent Spatial Decision Support using Expert System and Web Technologies

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EXTENDED ABSTRACT

Integrating GIS and web technologies with expert systems can provide valuable real-time spatial decision support. Traditional decision support systems (DSS) lack expert knowledge and are mostly PC-based software components. The benefits of the traditional DSS can be further leveraged by embedding expert knowledge and implementing the DSS using web-based technologies. Embedding expert knowledge with the DSS provides intelligent decision support rivaling a human expert, and implementing the intelligent DSS over the web provides many advantages, such as anytime-anywhere access, no distribution costs, ease of use, and centralized data storage. This paper discusses implementing one such architecture for snow removal operations planning that integrates GIS, expert systems, and web technologies.

Providing speedy and cost-effective snow removal management during winter months is overwhelming for local government personnel. Considering the massive costs of snow removal operations and their importance, it is essential that snowplowing operations be planned carefully and efficiently to minimize costs and provide a quick response. However, existing methods for snowplowing planning operations are mostly handled manually. For example, resource allocation and management is done without the help of analytical tools for optimization, and routing is often done without considering the least costly snowplow. To overcome these disadvantages, researchers have developed stand-alone, PC-based systems for planning snow removal operations. While manual methods often result in non-optimal resource allocation, increased deadhead time, and inefficient routing, the stand-alone systems lack advanced mapping capabilities or decision support and are often difficult for novice computer users. Thus, most agencies involved in snowplowing do not use any substantial analytical tools or methods before planning snowplowing to reduce costs and maximize the effectiveness of snowplowing operations. Also, there is no integrated use of weather information that could alert snow removal crews and provide scenario-based decision support. Therefore, significant need exists for developing an intelligent spatial decision support system (ISDSS) and a set of analytical tools for snow removal planning operations while providing ease of use for such a system.

The web-based ISDSS the authors developed integrates an intelligent software component with geo-spatial and analytical techniques for providing real-time decision support for planning snow removal operations. The system achieves the objectives of providing intelligent decision support by including

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expert knowledge on snow removal operations within the system. Further, the real-time weather information accessed from the internet, along with expert knowledge, helps in making real-time decisions based on different weather conditions. To provide ease of use, the user interface components, such as the toolbars, layers, and the map, were adapted to emulate the look and feel of standardized GIS software like ArcGIS. The current prototype has capabilities such as a user-friendly interface, powerful analytical tools for effective management and optimal allocation of resources (e.g., material, vehicles, and drivers), efficient routing, easy inventory management and analysis, an integrated real-time weather system, and expert knowledge. This system is capable of streamlining various tasks involved in planning snow removal operations and helps cut costs by increasing the efficiency of snow removal procedures.

The methodology for developing the ISDSS comprised the following four steps: (1) knowledge elicitation, (2) web-based GIS system design, (3) intelligent software component development, and (4) GIS and intelligent software component integration. The expert knowledge on snow removal procedures was gathered from snow removal officials for the Iowa DOT in Waterloo, IA, and street department officials in the cities of Cedar Falls and Waterloo, IA. Through a series of direct interviews, information on various standard operating procedures for snow removal was gathered. This knowledge was then embedded into the system to be referenced during spatial data analyses and decision making. Expert knowledge is crucial for arriving at an advised decision, and thus the system designed will be able to replicate human knowledge. A website was then designed containing the GIS-based road information of the study area of Black Hawk County, IA, in the form of shapefiles. These shapefiles were converted into a form suitable to be published on the web. The website also provides a way to perform various road data analyses, such as route creation and assigning material to routes. From the website, live weather data can be read and scenario-based analyses can be performed. There is also a provision for managing resources such as materials, drivers, and vehicles. In this way, the website integrates the GIS-based analytical and routing tools and weather data into a single interface, providing intelligent decision support.

The system uses ESRI ArcIMS ActiveX Connector, ArcIMS Route Server Extension, and related web technologies such as ASP, JavaScript, HTML, XML, and RSS. Figure 1 depicts the architecture that integrates the intelligent component (expert system) with spatial and non-spatial tools, providing a web-based interface. The ArcIMS ActiveX Connector was customized to provide mapping and analytical tools for snow removal tasks, such as routing, resource allocation, and inventory analysis. The ArcIMS route server extension was configured and customized to route efficiently between the snowplowing station and routes. Expert knowledge is encoded as "business rules" using Rule Machines' Visual Rule Studio, and is integrated as a dynamic link library into the system. RSS technology using XML weather feeds was used to read live and forecast weather information. The weather information in conjunction with the expert rules provides intelligent decision support for efficient routing, optimal resource allocation, and inventory control. Important features of the system include optimized route creation to reduce deadhead time, provision of pre-defined routes used by the DOT to avoid creating routes manually, embedded real-time weather data, efficient resource management and allocation, inventory analysis, and visual feedback on routes. The sample test runs show that the system effectively plans snow removal based on live and forecast weather data, and thus can improve snowplowing efficiency and cut costs.

The intelligent software component implemented for the current system is an expert system. This expert system helps in decision making by using a set of rules and reading real-time weather data. The expert system can use the current and forecast weather data to advise decisions regarding the shortest paths, prioritized routes, and optimal resource allocation. The expert system was designed using knowledge gathered from snow removal experts in Cedar Falls and Waterloo, IA, as well as the Iowa DOT. These rules were encoded as business rules in the form of a set of conditions resulting in a set of actions. These rules were then embedded into the system. Once the system prototype was implemented, it was demonstrated to county and city officials. Their feedback was sought to improve and refine the system.

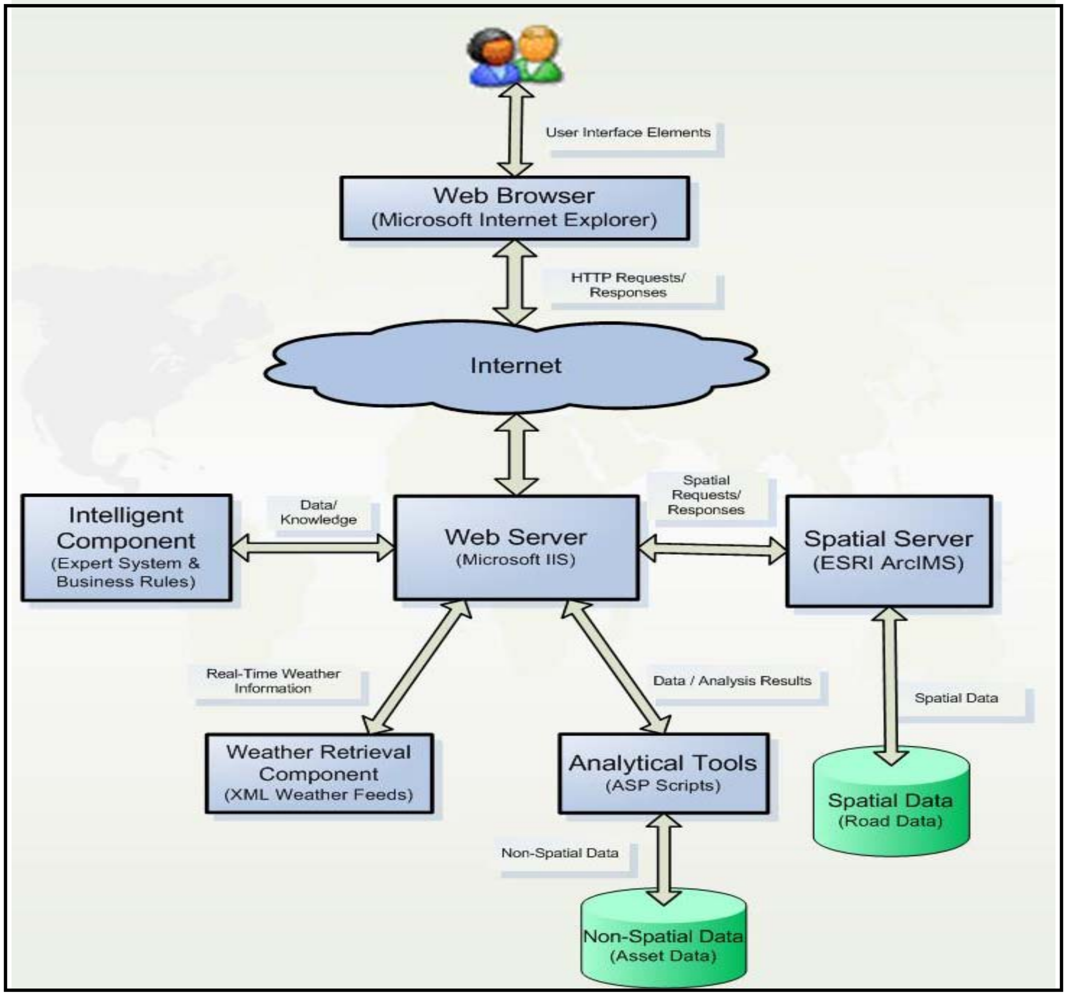

Figure 1. Integration of expert system and web-based tools for intelligent spatial decision support

In summary, integrating web technologies with expert systems helps provide intelligent decision support for spatial problems. This paper provides a proof-of-concept by developing an application that provides decision support for snow removal operations. This system integrates the expert knowledge on snow removal operations and web-based technologies to provide a user-friendly interface and analytical tools for snowplowing operations. Systems like these are capable of advised decision support through a web-based interface, thus facilitating ubiquitous access and ease of use. Future improvements for this research include utilizing ArcGIS Server 9.x and ArcObjects for realizing advanced analysis capabilities, utilizing individual road segment weather information, adding an intelligent software agent to manage resources and weather conditions, and providing advanced inventory analysis.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Interstate 235 ITS Program (Invited Presentation)

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ABSTRACT

This presentation will provide an overview of the Iowa Department of Transportation's Interstate 235 ITS program in Des Moines, Iowa.

Note: Contact the presenter above for more information on this topic.

Key words: intelligent transportation systems—interstate reconstruction—traffic operations

Economics of Upgrading an Aggregate Road

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ABSTRACT

This research provides local government agencies with information and procedures to make informed decisions about advantageous times for upgrading and paving gravel roads. It also provides resources to explain to the public why certain maintenance or construction techniques are used or policies are decided. Two approaches were used for estimating future costs. The first was a historical cost analysis based on the spending history for low-volume roads, found in the annual reports of selected Minnesota counties. The effects of traffic volume and road surface type on cost are included in the analysis. The second was a method for estimating the cost of maintaining gravel roads, which is useful when the requirements for labor, equipment, and materials can be predicted. Additional information was gleaned from interviews with local road officials. Considered maintenance and upgrading activities included maintenance grading, regraveling, dust control/stabilization, reconstruction/regarding, paving, and others. As part of this presentation, an analysis will be presented that compares the cost of maintaining a gravel road with the cost of upgrading to a paved surface. This analysis can be modified to address local conditions. Such an analysis may be used as a tool to help make decisions about upgrading a gravel road to a paved road.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: aggregate road—costs—decisions—gravel road—maintenance

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Thin Maintenance Surfaces for Municipalities

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ABSTRACT

In spite of increasing demands for service, many municipal street departments in Iowa have faced budget shortfalls due to revenue limitations. These budget shortfalls do not allow them to maintain an aging street system adequately without careful planning. Thin maintenance surface (TMS) techniques have been developed to extend pavement life by mitigating existing distresses. TMS techniques, such as seal coat, slurry seal, micro-surfacing, and fog seal, are cost effective preventive maintenance techniques that, when properly used, reduce pavement life-cycle costs. Currently, many municipal street officials cannot expend the effort to test or improve existing preventive maintenance techniques. By performing these evaluations, researchers hope to encourage improved selection of maintenance techniques in urban settings.

This research will show how TMS might be properly included in municipal street maintenance programs. The research evaluates the use of TMS in urban settings by constructing three sets of test sections in three Iowa cities. Condition surveys were performed before and after the application of the test sections and will be performed after the first winter to evaluate the performance of the each new surface. Construction was observed and crews used typical construction techniques so the results will be similar to those achievable using current techniques. This paper will report on the results of this effort based on information available at the time of writing.

Final results will be generalized to develop a practical guide of recommendations for TMS and preventive maintenance, which will be available for street officials.

Key words: micro-surfacing—preventive maintenance—seal coat—thin maintenance surfaces

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PROBLEM STATEMENT

Cities across the United States are continually facing budget cuts from decreased revenues. "In the National League of Cities' latest annual survey of city finance directors, more than three in five respondents (63%) said their cities were less able to meet financial needs during 2004 than in the previous year. Looking ahead, 61% say they expect their cities to be less able to meet their 2005 needs, relative to the current fiscal year" (Pagano 2004). As a result of decreased revenues, city officials are forced to cut budgets and streamline services provided. A major sector requiring large amounts of a city's budget is municipal public works departments. These departments are responsible for facilities such as streets, sewers, water works, electricity, cemeteries, and parks maintenance. Although a city cannot radically reduce public works funding for obvious reasons, the funding may not be adequate to meet current needs nor be increased to support growth.

To compound the problem of budget shortfalls, public works officials must also deal with aging public works systems, specifically streets. As these streets age, street officials must deal with rehabilitating and reconstructing these pavements to maintain safety and a comfortable ride. The maintenance, rehabilitation, and reconstruction activities of these streets are very costly, which reduces the amount of work the public works department can perform each season. This lack of resources and attention causes other streets and facilities to continue to degrade.

As public works officials evaluate the current condition of their street systems, cost effective methods and means of extending service lives will be necessary or the overall condition of the streets system will continue to deteriorate. By extending the life of a pavement beyond the designed service life, the streets department is able to spread out the workload and decrease the amount of rehabilitation and reconstruction necessary to maintain the system in good condition.

However, many street officials are not only responsible for maintenance of streets; they are also responsible for cemeteries, sewer systems, and city mowing. These additional responsibilities reduce time that could be used to investigate and test different techniques to determine how to maintain pavements effectively. Currently, many street officials have limited awareness of and experience with different maintenance techniques and their uses.

Public works directors and street superintendents are beginning to utilize preventive maintenance (PM) strategies to address their aging streets. As defined by the American Association of State and Highway Transportation Officials (AASHTO), PM is "the planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without substantially increasing structural capacity)." Many types of preventive maintenance techniques have been developed to maintain and extend the service life of streets. Each of these surfaces and techniques can effectively mitigate or prevent distresses such as cracking and raveling that shorten a pavement's service life. However, many of the techniques that work well with certain types of pavements and distresses are not effective on others.

Thin maintenance surfaces (TMS) are a set of cost effective preventive maintenance surfacing techniques that can be used to extend the life of a bituminous pavement. These surfaces do not increase the strength or structure of a road, but rather mitigate existing distresses and prevent future distresses that shorten a pavement's service life. TMS solutions include surfacing techniques, such as seal coats, slurry seals, micro-surfacing, fog seals, and smooth seals. By addressing the issues that cause early pavement deterioration with a TMS, a municipality is able to extend the life of a pavement, allowing other funds to be allocated to ones in need of rehabilitation or reconstruction.

Extensive research and numerous studies have been performed to improve and evaluate the effectiveness of these various TMS techniques. These studies have focused on developing new design techniques for each surface, determining which distresses are mitigated, determining ideal application times, defining best practices for construction, developing decision matrices for determining the surface that should be applied, and analyzing life-cycle costs of the maintenance techniques. Many of these studies are lengthy and do not provide street and road officials easy access to relevant material. Although these studies may provide useful information, street department superintendents are weary of placing trust in something that they cannot see for themselves or evaluate based on their colleagues' experiences. These studies also encourage municipalities to develop pavement management programs and overhaul existing programs to incorporate PM and TMS. However, if officials struggle to learn and test new techniques, the chances of adopting a new program are small. Furthermore, these types of programs are also not suitable for smaller cities that do not need an extensive program or simply cannot afford to overhaul an existing program.

RESEARCH OBJECTIVES

This research project is expected to provide street officials with appropriate solutions for pavement maintenance that officials can easily test and include in their current programs. Previous phases of TMS research projects have provided excellent information about using TMS in rural settings. A result of these previous phases was a decision matrix that officials can use to select treatments that have been developed. The results have limited application to urban settings that pose unique road maintenance challenges.

Under this research project, three test sections were constructed using TMS in urban settings. Test section sites and surfaces were selected to suit the needs of the cities, each site with different distresses and maintenance needs. The performance of the constructed test sections will be observed for one year.

These test sections will serve as great examples to street officials for many reasons. The test sections give officials the opportunity to observe and evaluate TMS in an urban setting. The generalizations will help these officials to make well thought-out decisions about their own maintenance procedures. Moreover, some TMS applications are still relatively unknown in Iowa, and these test sections will make officials aware of the various types of surfaces and materials that can be used. The test sections will be an example to municipalities of how to test different TMS applications and materials.

During construction of the test sections, hindrances were identified and lessons learned will be included in a technology transfer program that is being developed in conjunction with test section construction.

RESEARCH METHODOLOGY

City street engineers who expressed interest in this project were asked to serve on the technical advisory committee and recommend streets in their respective cities to become candidate test sections. The labor, equipment, and materials for test section construction was supplied by the sponsoring city. In each case, test section construction addressed a current maintenance need for each of the sponsoring cities. The researchers specified that the street needed to be located in an urban setting, the existing top surface needed to be bituminous, and the traffic count needed to be similar to what other urban streets were experiencing. Moreover, the city was asked to outline its current street maintenance program. This included brief descriptions of current maintenance practices and previous experiences with TMS. Researchers needed to become familiar with the needs of the city, as well as the level of funding available for the test sections.

Each city provided three to six streets of varying age, location, traffic loads, and pavement conditions. The researchers toured the designated roads and made final selections. Consideration was given to the pavement type, types of distresses present, density of the distresses, and the traffic volume. The researchers also wanted to test pavements with higher traffic volumes so streets with higher average daily traffic (ADT) were favored over others with lower ADT. The pavements needed to have a bituminous surface of either a hot mix asphalt pavement or a seal coat. A seal coat is an application of asphalt binder followed by a single layer of aggregate chips. If the test section had distresses that indicated structural failure of the pavement, the pavement was considered for a stopgap procedure, an attempt to extend pavement life for a few years before a more expensive rehabilitation or reconstruction project can take place. Because TMS techniques do not effectively mitigate or prevent structure-related distresses, researchers recommended that base stabilization be performed in the problem areas before application of the new surface. Three test sections were selected in West Des Moines, two in Cedar Rapids, and one in Council Bluffs, making a total of six.

Discussion between the city engineers and researchers helped in the process of selecting the test sections. Researchers interviewed the city engineers to collect further details on each test section to determine the goals for each effort. A number of TMS applications were suggested for each pavement that was a likely candidate. The advantages and disadvantages of each surface were discussed for each street. This discussion involved many topics, from construction limitations to material availability and funding concerns. City engineers questioned material availability and were not certain whether the materials for certain TMS applications could be acquired. These discussions required a number to ensure that all concerns would be identified and uncertainties could be mitigated.

After the final TMS selection for each test section, researchers finished investigating how the surface would be constructed, what materials were available, and which materials were the best to use. This investigation involved a review of past experiences, the literature, and case studies on TMS. Material suppliers and contractors were contacted.

A description of the construction process for each test section is described below.

Test Section Surveys

The survey type used to evaluate the condition of the different test sections was the pavement condition index (PCI), developed by the U.S. Army Corps of Engineers. The PCI is a numerical index, ranging from 0 for a failed pavement to 100 for a perfect pavement. PCI is calculated by visually measuring the type, severity, and density of pavement distresses. After the survey is completed, the ratings of the distresses are used as input for a series of formulas, and the output is the condition index for each sample section. The average PCI for all of the sample sections is the final PCI for the test section. This average addresses possible variability in the pavement condition due to location.

While performing PCI surveys on seal coat roads, the surveyor chose to ignore some distresses that were not caused by problems with the construction of the TMS themselves. An example of this is grooves left from a motor grader's ripper teeth before the application of the first seal coat. Although the grooves could be considered rutting, they were not caused by the seal coat construction process and therefore were not included in the survey.

Unfortunately, thieves broke into the graduate student's vehicle and stole a number of items, including the folder that contained many of the pre-construction surveys. Because the surveys were stolen after the application of the new surfaces, the PCI of the pavements before construction are unknown. However, the researchers performed surveys on similar streets with similar structure, history, and traffic for a general comparison. If the test section had no similar streets, no comparative analysis was made to quantify the added value of the surface. Only the performance of the surface was analyzed.

TEST SECTION DESCRIPTION AND CONSTRUCTION

College Road in Council Bluffs, Iowa

College Road is a collector street located near Iowa Western Community College in the northeast corner of Council Bluffs. College Road is a 2-inch (50.8-mm) full depth seal coat road, with pea rock as the aggregate for the surface course. The road has little to no shoulders and the ditches do not appear to be well drained. However the ditches are deep, approximately 10 feet, and standing water should not affect the sub-base. The test section is located on a curved section of the road where the speed limit is reduced to 25 mph because of the tightness of the curve. The traffic before construction was approximately 1,800 vehicles per day. The PCI of College Road before construction is estimated at 30–50 (poor). Some of the more severe distresses included alligator cracking, potholes, bleeding, and rutting. The alligator cracking and potholes were located in the outer wheel paths in the lane on the outside of the curve. Most of the rutting was also located in the outer wheel paths of both lanes. The structural deficiency was the largest contributor to the poor condition of the pavement. TMS applications are not usually recommended for pavements in poor condition. Researchers realized that a new seal coat would not yield any benefit to the pavement and made efforts to correct some of the structural distresses before construction, which would increase the PCI to a fair rating.

Because of the structural deficiencies, dynamic cone penetrometer (DCP) tests were conducted prior to construction to test the stability of the subbase. A DCP test (ASTM D 6951–03) is a procedure that measures the penetration of a steel rod with a cone tip into subbase, using a hammer of prescribed height and drop to drive the rod into the ground. The penetration rate may then be correlated to the California Bearing Ratio (CBR). The tests concluded that the base was sufficiently strong. Consequently, the researchers recommended only tearing up and stabilizing the areas where the rutting, potholes, and alligator cracking occurred. However, the city crews restabilized the entire test section. Reconstruction of the test section included mixing the seal coat in with the subbase, adding limestone aggregate and water, regrading the road, compacting, and applying a primer coat (MC150). When the road was regraded, the surface was inconsistent, with areas of tightly and loosely compacted subgrade. Due to a late start in the season (late September), crews were rushed through the reconstruction of the road and sufficient time was not given for the primer to cure before the seal coat was placed. After three days, the new seal coat was applied.

A three-eighths-inch pea rock aggregate was used with a CRS-2P (cationic, rapid setting, polymer-modified) emulsion. An emulsion is a mixture of asphalt binder, water, and a surfactant that allows the cold (150° F) application of an asphalt binder. Cationic refers to the charge of the emulsion (cationic is positive and anionic negative) and the setting time refers to how quickly the emulsion breaks (water separates from the asphalt binder). Council Bluffs typically uses a high-float (non–polymer-modified) emulsion; however, the original seal coat was experiencing areas of bleeding. A high-float emulsion is slow setting and very forgiving. High-float emulsions are typically used when dusty aggregate is common. Researchers recommended the polymer modifiers for the emulsion in order to correct the problems with bleeding, as well as to help resist any minor cracks from forming in the seal coat. These minor cracks were caused by the high shear stresses induced on the pavement due to the turning traffic and possible deflection of the stabilized base. Because the city of Council Bluffs does not own an electronically calibrated distributor or chip spreader, no design was made for the application of the seal coat. Application rates were similar to those the seal coat crew had successfully used in the past.

Prior to construction, the majority of the traffic was local to the Iowa Western Community College and consisted mostly of cars. After construction, earthmoving began on a new construction project adjacent to the test section. The earthwork required considerable use of tandem-axle dump trucks. There was also reconstruction on a section of College Road on the opposite end of the new construction. This reconstruction involved replacing the existing asphalt pavement with a concrete pavement. The construction equipment, dump trucks, and many of the concrete trucks used College Road as an access road to get to both job sites.

The CRS-2P emulsion is a rapid setting emulsion and requires application of chips within one to two minutes of the application of the emulsion. However, the seal coat crew was familiar with a high-float emulsion, which does not require the immediate application of chips. This unfamiliarity challenged the seal coat crew, as they were not used to having to cover the emulsion with aggregate quickly.

Aspen Drive in West Des Moines, Iowa

Aspen Drive is a residential street located in north-central West Des Moines. Before construction, Aspen Drive was a PCC pavement with an ACC overlay of unknown thickness. The street has concrete curbs, gutters, and storm sewers that provide excellent drainage. The major distresses prior to construction were low to medium severity reflective cracking and longitudinal and transverse cracking. Many of the reflective cracks had been sealed in 1991. The test section is located in a subdivision, and, based on previous experience, city officials estimated an ADT of approximately 500. The PCI of the test section was 49. One of the main concerns with the new surface citizen complaints about aesthetics.

Micro-surfacing was selected as the best surface for the West Des Moines test sections. Micro-surfacing is a slurry of aggregate, polymer-modified emulsion, mineral filler, and water that is mixed before placement on a pavement. The slurry application is only one-stone thick. Micro-surfacing was chosen because of it's robustness against crack spalling, its waterproofing effect on the surface, and its appearance, which is similar to a new asphalt pavement. A limestone aggregate type III micro-surfacing mix was used (for gradation and other specifications, see "ISSA Recommended Performance Guidelines for Micro-Surfacing, A143 (revised), May 2003"). The contractor for all West Des Moines test sections was Sta-Bilt Construction, based out of Harlan, Iowa, and the emulsion provider was from Koch Materials. Researchers had experience in previous phases for this project and indicated that the Type III gradation band could produce an aggregate that is too course; therefore, they specified a modified gradation with more fines. The added fines helped the spreader apply a tighter surface with fewer drag marks, as the fines prevent the larger aggregate from getting stuck underneath the screed by creating friction between the pavement and the larger pieces of aggregate.

Fourth Street and Vine Street in West Des Moines, Iowa

Fourth Street is a collector street located on the east side of West Des Moines. Before construction, 4th Street was a PCC pavement with an ACC overlay of unknown thickness. Both 4th and Vine streets have concrete curbs, gutters, and storm sewers, which provide excellent drainage. The major distresses prior to construction were low severity reflective cracking and longitudinal and transverse cracking. Many of the cracks had been sealed in 1993. The test section is located in a residential/commercial area. An industrial company is on the north end of the test section, so a small amount of truck traffic delivers to that business. The estimated ADT is approximately 2,000 for 4th Street.

Vine Street is a collector street located on the east side of West Des Moines. Before construction, Vine Street was a PCC pavement with an ACC overlay of unknown thickness. The major distresses prior to construction were low severity reflective cracking and longitudinal and transverse cracking. There was also a moderate amount of low (1/4-inch to 1/2-inch) to medium (1/2-inch to 1-inch) severity rutting. Many of the cracks had been sealed, and the seals were in good condition. The test section is located in a residential/commercial area and has a low amount of truck traffic, which is only local to surrounding retail stores. The estimated ADT is approximately between 5,000–6,000.

Both limestone and quartzite aggregate were used for the type III micro-surfacing mix used on 4th Street and Vine Street.

When micro-surfacing is laid, many pieces of aggregate stand on edge, which increases tire noise. After the aggregate has been worked down to its side, the added tire noise diminishes. In order to help the aggregate lay down more quickly, a pneumatic tire roller was specified during construction. However, during actual construction, researchers and the city engineers concluded there was no noticeable added value, so the use of the roller was ceased.

Vermont Avenue in Cedar Rapids, Iowa

Vermont Avenue is a local residential street located on the south side of Cedar Rapids. Prior to construction, Vermont Avenue consisted of a seal coat over a full-depth asphalt pavement. The seal coat was laid many years ago (actual date unknown) and a pea rock aggregate was used as the cover aggregate. The major distress experienced by the pavement was low to medium alligator cracking. The street foreman in charge of maintenance on Vermont Avenue said that the alligator cracking reflected through the pavement after the first few years of the seal coat construction. He also said that since the initial alligator cracks formed, the alligator cracking has not propagated significantly, and at the time of test section construction, the cracks were dormant (the pavement did not pump under traffic). Other distresses included low severity longitudinal and transverse cracking and low severity raveling. A high percentage of the alligator cracks and longitudinal and transverse cracking had been sealed by the city in previous years, and the seals were in good condition. No efforts were made by the street maintenance crew to patch or reseal any cracks before the new surface was applied. The test section is located in a subdivision and experiences only local residential traffic, with the exception of a public transportation bus. One of the main concerns with a new surface was the aesthetics. The test section was located in a residential area and the city engineer was concerned with citizen complaints about dust problems and the appearance of a gravel road instead of an asphalt pavement.

For the construction of the test section, researchers recommended that a seal coat be placed over a majority of the pavement and a process called chipmat performed over the areas experiencing alligator cracking. A chipmat is a tack coat of hot asphalt binder or emulsion sprayed over the alligator cracking. A geo-technical fabric is then rolled over the tack coat. A layer of sand is spread over the fabric, and the fabric is seated into the tack coat with a pneumatic tire roller. The sand is then swept off of the fabric, and a standard seal coat is applied to the entire pavement. If an emulsion is used for the tack coat, the fabric should set for two to three days in order for the water to cure out of the emulsion. This is done to prevent the water from being trapped under the fabric and the seal coat, causing delamination between the fabric and tack coat. Using an emulsion is not common practice because of the problems that the water presents. Researchers realized that using an emulsion was not ideal; however, it was not feasible to use a hot asphalt tack coat because the city only had one distributor truck and it was normally filled with high-float emulsion. The procedure for changing from emulsion to hot asphalt is very time consuming. For Vermont Avenue, a pre-coated limestone chip and a high-float emulsion (HFE-90) were used.

Seventy-fourth Street in Cedar Rapids, Iowa

Seventy-fourth Street is a residential street located on the north side of Cedar Rapids. Seventy-fourth Street was an existing full-depth seal coat road with a limestone cover aggregate. It is not certain whether the limestone chip was pre-coated or not. The main distresses that the pavement experienced before construction was medium- to high-severity bleeding, low-severity alligator cracking, and potholes. A majority of the alligator cracking was on the east end of the project in a section approximately one block in length. Seal coats do not commonly have a large amount of longitudinal and transverse cracking because a seal coat is much more flexible and resilient than asphalt pavements. Before the new surface was applied, the street maintenance crew performed full-depth hot mix asphalt patches on any of the areas experiencing alligator cracking or potholes. These patches significantly improved the condition of the road. On the far east end of the project, where the alligator cracking was more severe, the crew stabilized the subgrade with magnesium chloride. The main concern with this test section was the aesthetics because it was located in a residential area, and the city engineer was concerned with citizen complaints about dust and the appearance of a gravel road instead of an asphalt pavement. The city engineer mentioned that 74th Street was going to be replaced with a concrete pavement in the next three to five years and only wanted the road to hold together until that construction took place.

The test section was divided into four sections to test the various types of aggregate locally available near Cedar Rapids. The following is a description of each segment:

- Segment I: Seal coat. 3/8-inch pre-coated limestone chips with high-float emulsion (HFE-90). Application rates unknown.
- Segment II: Seal coat. 3/8-inch washed limestone chips with high-float emulsion (HFE-90). Application rates unknown.
- Segment III: Seal coat. 3/8-inch pea rock with high-float emulsion (HFE-90). Application rates unknown.
- Segment IV: Double seal coat. 1/2-inch washed limestone base chip with a 3/8-inch pre-coated limestone cover chip with high-float emulsion (HFE-90). Application rates unknown.

Application rates were unknown because both the distributor’s and the chip spreader’s computers had malfunctioned and there was no accurate way to know the application rates. Time constraints did not allow a manual calibration of the equipment. Quality control during the construction of the various seal coats was not rigorous. The crews were rushed because construction did not start until very late in the season. The pneumatic tire roller did not arrive on the site until three hours after start of construction on the second day. For best results, it is desirable to set the chips in the emulsion quickly by using a pneumatic tire roller. Both the washed limestone chip and the pea rock aggregate were very dusty. Dusty aggregate sometimes does not bind well with the emulsion; however, the use of high-float emulsion mitigates such problems.

RESULTS

The following are the results of the test sections after one winter of performance. Because the test sections experienced a variety of traffic loadings, had different amounts of subbase support, and experienced different distresses, each test section stands as its own case study, and thus the sections cannot be compared to one another to measure performance.

College Road in Council Bluffs, Iowa

The post-construction survey revealed that there was little to no improvement of the condition of the road. The new PCI of the road was 48, which is a fair condition rating (previous estimated PCI was 30–50). This value does not include the portion of the test section adjacent to the new construction site, where earthwork was occurring and the truck traffic had caused a total failure in the pavement. This portion was left out because it did not reflect the condition of the rest of the pavement. The distresses in the new seal coat were similar to that of the original seal coat. Rutting, alligator cracking, bleeding, and potholes had formed in the same areas as the original distresses, even though the entire road was restabilized. As in the original seal coat, the majority of the distresses were located in the outer lane. The area adjacent to the entrance of the construction site had failed due to the heavy truck traffic. High-severity rutting, alligator cracking, bleeding, and raveling consumed most of the pavement near the entrance. Interestingly, the areas on the original seal coat where the seal coat was in excellent condition were the same areas that were in good condition on the new seal coat.

Aspen Drive in West Des Moines, Iowa

The post-construction survey revealed that there was considerable improvement in the condition of the road. The original PCI of the Aspen Drive section was 49, while the new PCI was 82, which is a very good rating and an increase of 33 points. The only distress present in the new micro-surfacing was reflected cracks. Many of the original reflected cracks in the overlay reflected through the new micro-surfacing treatment. This reflection is typical of micro-surfacing treatments. The micro-surfacing is performing well. In a few high spots, a snowplow sheared off the high edges of the limestone aggregate, but sufficient aggregate still remains to cover the original pavement.

Fourth Street and Vine Street in West Des Moines, Iowa

The post-construction survey revealed the PCI for 4th Street increased from an estimated 81 (very good) to 86 (excellent), while the PCI for Vine Street decreased slightly (81 to 80, very good). The estimated preconstruction PCI is taken from Maple Street, a pavement with a condition similar to 4th Street and Vine Street before the application of the micro-surfacing. Similar to Aspen Drive, the main distress for both 4th Street and Vine Street was reflected cracking. Vine Street had experienced some light-severity rutting before construction, and the micro-surfacing did not fill in the ruts (rut filling was not specified in the contract). The lack of rutting on Maple Street accounts for why Vine Street has a lower PCI than Maple Street. There was only one small spot where the snow plow completely removed the micro-surfacing mix. Both types of aggregate, quartzite and limestone, were used for both streets and are performing equally well. The sections of micro-surfacing with limestone aggregate seemed to have a tighter surface texture because there was a higher amount of fines in the limestone gradation.

Vermont Avenue in Cedar Rapids, Iowa

The post-construction survey revealed that the PCI for Vermont Avenue was 72, which is a fair rating. The seal coat is in very good condition, with no raveling or bleeding. The main distress was light-severity reflective cracking from the previous seal coat. The chipmat also performed very well. There was no reflective cracking in the areas where the chipmat was placed. For some of the areas that had alligator cracking, the researchers chose not to place fabric in order to provide a comparison. In these places, the alligator cracking reflected through to the new seal coat. There were a few concerns with the chipmat, including the inability of chipmat to prevent reflective cracking, its inability to bond well to the existing seal coat, and the potential for significant bleeding over the chipmat. Inspection after the first winter indicated no distress consistent with the previously mentioned concerns. The city engineer was very pleased with the chipmat and is considering using it again in the future.

Seventy-fourth Street in Cedar Rapids, Iowa

The post-construction survey revealed the overall PCI for 74th Street was 72, which is rated very good. As expected, results varied with each of the types of aggregates. The section of seal coat that performed the best was the section with the 3/8-inch pre-coated aggregate with a PCI of 81, a rating of very good. Other than slight raveling, the seal coat was performing well, with no visible distress. The seal coat lost many PCI points because of rutting and other distresses that existed before construction and could not be addressed by the seal coat. Interestingly, one of the sections of the pre-coated aggregate had been sitting for over a day and the other section had been sitting for many hours before a pneumatic tire seated the aggregate in the emulsion. This shows how well the emulsion and the pre-coated chips bonded to one another and gives evidence of the robustness of the pre-coated chip and high-float emulsion combination.

The 3/8inch washed limestone chip section did not perform well, with a PCI rating of 63, which is rated good. A large area of the seal coat was heavily raveled due to snow plow damage. The raveling problem might be attributed to poor bonding caused by the dusty aggregate used and the lack of the compaction by the pneumatic tire roller. It is possible that the raveling would not have been as severe had a roller been used because much of the raveling was outside the wheel paths. This is because the chips in the wheel path were seated in the emulsion by the passing traffic. The city engineer also mentioned that this section of the test section created large amounts of dust for several days after construction, which caused numerous citizen complaints. The city attempted to reduce the dust by washing the section with a water truck, but this effort was mostly unsuccessful.

The 3/8-inch pea rock section also did not perform well, with a PCI rating of 66, a rating of good. Similar to the washed limestone chip section, there was considerable medium- to high-severity raveling. Again, the raveling might be caused by dirty aggregate and a lack of compaction. This section also caused complaints of excessive dust generation.

The double seal coat section, 1/2-inch washed limestone chip base with a 3/8-inch pre-coated limestone chip, performed quite well, with a PCI rating of 95, which is excellent. This section experienced only slight bleeding and other distresses that were present before construction and could not be addressed by seal coating. There was one area with medium-severity alligator cracking and rutting; however, these distresses were not included in the survey because they were outside the sample boundaries. The lack of raveling can be attributed to the fact that this test section was rolled by the pneumatic tire roller immediately after application of both layers of chips. There were concerns that this section would have issues with bleeding. However, to date, there has been no bleeding on this section.

CONCLUSIONS

The following conclusions from this project were taken from the various test sections constructed. No statistical analysis was performed on the test section data to interpret the results because each test section stands as its own case study; the sections cannot be compared to one another to determine the most successful surface.

- Proper construction technique is one of the primary influences on the success of TMS. Failures on a TMS that occur early in the life can be traced back to poor construction or quality control.

- A conclusion could not be made regarding the effectiveness of polymer-modified emulsion. The test section used to investigate this topic had a number of construction issues that confounded the results.
- The use of micro-surfacing was successful. There were no apparent differences in the use of quartzite or limestone aggregate.
- The higher proportion of fines to the limestone aggregate gradation for the micro-surfacing helped in creating a smoother and tighter surface.
- The use of the pneumatic tire roller to aide in smoothing the micro-surfacing and reducing noise was not found to cause noticeable improvement**.**
- Cedar Rapids' current program, which uses a high-float emulsion (HFE-90) and a pre-coated limestone chip for a seal coat, is a very robust combination. This combination has out-performed the other aggregates, even though it was not rolled promptly with a pneumatic tire roller and quality control was not rigorous.
- The washed limestone chip and pea rock aggregate received numerous public complaints due to fugitive dust. This should be considered in areas where complaints regarding fugitive dust are likely to be an issue.
- The use of a pneumatic tire roller is essential to embed aggregate into the emulsion for successful seal coat construction.
- Emulsion can be used as a tack coat for a geotechnical fabric-reinforced seal coat if the tack coat and fabric are given ample time to cure before the first seal coat is applied. The chipmat using emulsion as a tack coat is a feasible and cost-effective solution for alligator cracking that does not deflect under load. This conclusion is based on the construction of one demonstration section.

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Long-Term Performance of Cold In-Place Recycled Asphalt Roads

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ABSTRACT

Within three to five years following construction of asphalt pavements, reflected cracks may be observed, one of the primary forms of distress in hot-mix asphalt overlays of flexible pavements. Reflected cracks affect ride quality when rolled down and allow water to penetrate into the pavement and the base, causing the asphalt mix to deteriorate and the base to soften. Consequently, the service life of pavements is reduced. Cold in-place recycling (CIR) provides an economical rehabilitation method that mitigates crack reflection by pulverizing the asphalt pavement surface, thus destroying the old crack pattern in the recycled layer. While the performance of recycled roads is generally good, there is some inconsistency. Several years after recycling, some roads are in excellent condition, while more cracking and rutting is observed on other roads. These differing behaviors can be observed on roads constructed in the same county by the same contractor in the same construction season. Thus, the difference in performance is probably not from such factors as weather, equipment, contractor experience, and construction procedures. Rather, other factors more prominently affect pavement performance, such as recycled pavement age, traffic volume, support conditions, and aged engineering properties of the CIR materials.

This paper discusses a partially completed investigation to identify how aged engineering properties of the CIR materials and other factors affect pavement performance. A selection matrix consisting of 18 sample roads was developed based on previous study. These 18 sample roads represent various ages (young/medium/old), traffic volumes (high/medium/low), and support conditions (strong/weak) in a geographically balanced sampling in Iowa. Pavement condition index (PCI) ratings were collected using an automated pavement distress digital image collection and analysis system. Engineering properties of CIR materials (density, compressive strength, indirect tensile strength, resilient modulus, and asphalt and aggregate content) will be examined through field and lab tests. Statistical analysis will be conducted to describe the relationships between pavement performance and the prominent factors. It is expected that the conclusions and recommendations from this study can be used to improve the performance of future CIR projects in Iowa and other states.

Key words: aged engineering properties—asphalt pavement—cold in-place recycling—pavement performance—recycling—reflected crack

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INTRODUCTION

Flexible pavements deteriorate with time. Typically three to five years following construction, reflected cracks may be observed, one of the primary forms of distress in hot-mix asphalt (HMA) overlays of flexible pavements (Myers, Roque, and Ruth 1998). When rolled down, reflected cracks affect ride quality. Moreover, reflected cracks allow water to penetrate the pavement, which accelerates the deterioration of the overlay and the underlying pavement. Soft base is formed, and hus the service life of pavements is reduced.

HMA overlay is often applied to extend the service life. Experience indicates that cracks reflect through the new overlay within two to four years (McKeen, Hanson, and Stokes 1997). This problem highlights the need for other rehabilitation methods that mitigate reflected cracks.

Cold in-place recycling (CIR) provides an economical rehabilitation procedure that mitigates crack reflection by pulverizing the asphalt pavement surface, thus destroying the old crack pattern in the recycled layer. In 1998, the Iowa DOT and Iowa Highway Research Board initiated an evaluation of the performance of CIR asphalt cement concrete roads (HR-392) (Jahren et al. 1998). Research results from 18 sample roads showed that CIR retarded the development of transverse cracking (reflected cracks). Additionally, CIR roads within the state of Iowa and with an annual average daily traffic (AADT) of less than 2,000 were predicted to have an average service life of 15 to 26 years.

PROBLEM STATEMENT

Several years after recycling, some roads are still in excellent condition with only a few minor cracks while more cracking and rutting is observed on other roads. These differing behaviors can be observed on roads that were constructed in the same county, by the same contractor, and in the same construction season. Thus, the difference in performance is probably not from such factors as weather, equipment, contractor experience, and construction procedures. Rather, other factors become more prominent in affecting pavement performance, such as the following:

- Age of the recycled pavement
- Traffic volume
- Support conditions
- Aged engineering properties of the CIR materials

RESEARCH OBJECTIVES

The objective of this paper is to answer two questions concerning CIR performance:

1. How do aged engineering properties, traffic, and subgrade conditions affect pavement performance?
2. What changes should be made with regard to design, material selection, and construction in order to improve the performance of future recycled roads?

This paper will provide a program report for the study and describe the experimental design.

EXPERIMENTAL DESIGN METHODS AND PROCEDURES

The design of this research project is shown in the flowchart in Figure 1 and further described in this section.

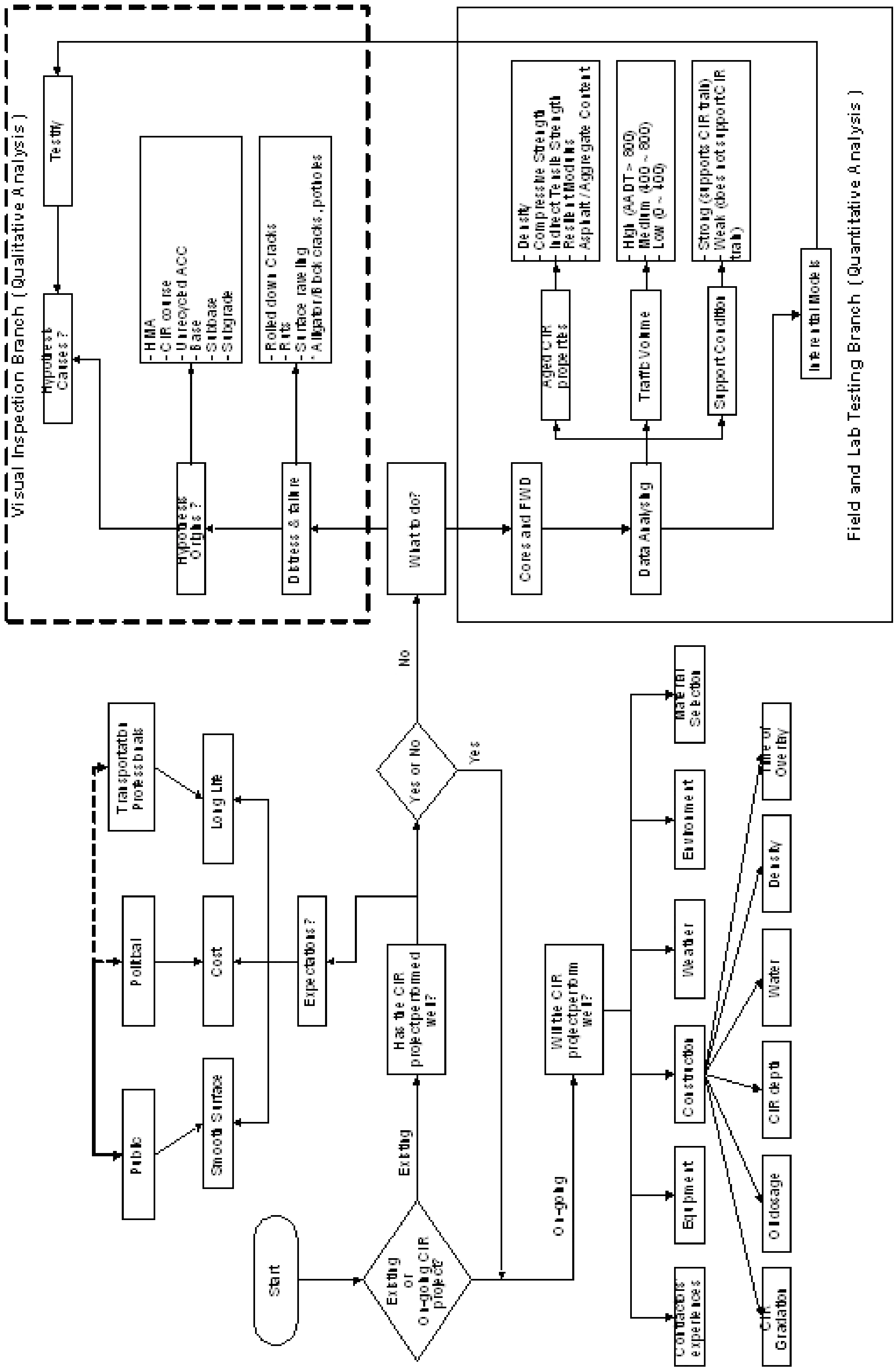

Figure 1. Research design

Scope of the Study

When this study is completed, the final report will summarize the results of a comprehensive evaluation of test results from field distress surveys, field samples, and lab tests from 18 projects constructed between 1986 to 2003 at various locations throughout the state of Iowa. Of these 18 projects, 12 projects were selected from the sample roads in the previous research (HR-392). Six projects were selected among newly constructed CIR projects in Iowa after 1999.

Design Considerations

Three factors are believed to have the most influence on the performance of recycled pavement: the age of the recycled pavement, traffic volume, and support conditions. Each factor was considered to be a multiple-level factor. The breakdown of the structures of the three factors are shown in Table 1:

Table 1. Factors believed to influence CIR performance

Age	Young	1999 ~
	Medium	1993 ~ 1998
	Old	1986 ~ 1992
Traffic (AADT)	High	> 800
	Medium	400 ~ 800
	Low	0 ~ 400
Support	Strong	Supports recycling train
	Weak	Does not support recycling train

Sampling

The 18 projects were selected to represent all Iowa counties, project ages, traffic levels, and support conditions, as shown in Figure 2.

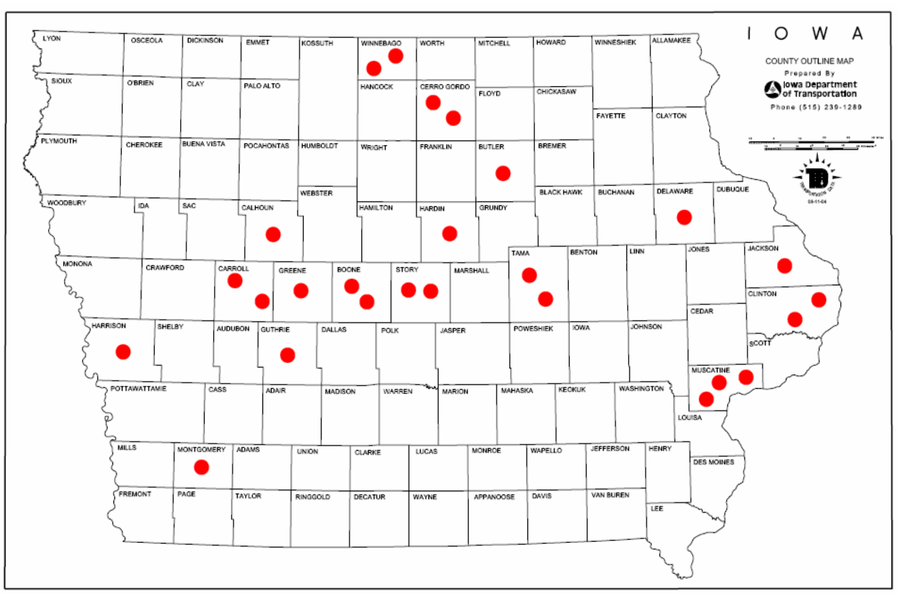

Figure 2. Distribution of candidate roads

Data Collection

Selection matrix

Interviews with county engineers, Iowa DOT personnel, and forepersons were conducted in order to obtain information on the pavement support conditions. The results were used to select candidate roads to complete the selection matrix (Figure 3).

		Good Support / Drainage			Poor Support / Drainage		
		Low Traffic (0~400)	Medium Traffic (400~800)	High Traffic (>800)	Low Traffic (0~400)	Medium Traffic (400~800)	High Traffic (>800)
Age of Pavement	Young (1999~)	N-58 (Carroll)	IA-44 (Harrison) S-14 (Story)	US-20 (Delaware) US-61 (Jackson) S-27 (Story)	N of Breda (Carroll)		IA-48 (Montgomery)
	Medium (1993~ 1998)	R-34 (Winnebago)	V-18 (Tama)	IA-175 (Calhoun) IA-4 (Guthrie)		T-16 (Bulter)	F-70 (Muscatine)
	Old (1986~ 1992)	Boone (E-52)	B-43 (Cerro Gordo) Z-30 (Clinton)	E-66 (Tama)	198th (Boone)	E-50 (Clinton) S. S. (Cerro Gordo)	Y-14 (Muscatine)

Figure 3. Selection matrix

Pavement Condition Index Ratings

An automated pavement distress digital image collection and analysis system was used to perform distress surveys. This survey was conducted by researchers at the University of Iowa under Dr. Hosin Lee. The pavement condition index (PCI) rating of each road was calculated with the aid of computer software. In order to obtain inferences on a longitudinal study similar to the previous research, the digital pavement crack analysis was conducted on the same portions of the 18 roads previously surveyed.

Field and Laboratory Testing

It is expected that the following parameters will be measured to obtain the AEP of CIR materials (Robert et al. 1996):

- Density
- Compressive strength
- Indirect tensile strength
- Resilient modulus
- Asphalt and aggregate content

Data Analysis

The goal of the data analysis is to identify the effects of the aged engineering properties of CIR materials, age of the pavement, traffic, and support conditions on the performance of recycled asphalt roads. Aged engineering properties, age of pavement, traffic, and support conditions are explanatory variables. Performance is the response variable that can be defined by PCI ratings. Three steps are planned for the data analysis phase of this project.

In the first step, the aged engineering properties of CIR materials will be estimated within each cell of the selection matrix. This estimation will be performed by the regression function obtained from statistical regression analysis of observed pavement data (density, compressive strength, indirect tensile strength, resilient modulus, and asphalt and aggregate content).

In the second step, PCI ratings will be estimated by aged engineering properties, age of the pavement, traffic, and support conditions. It seems likely to the research team that the PCI ratings are strongly influenced by the performance of the HMA overlay. Serving as a base for the HMA overlay, the condition of the CIR layer can be quantitatively defined by the aged engineering properties. Thus, an association between PCI and aged engineering properties exists theoretically. It is rational to infer that relationships exist between PCI and other explanatory variables.

In the third step, a fitted model will be developed by fitting the expected PCI ratings (obtained in the second step) against the observed PCI ratings (obtained from the automated distress surveys performed by researchers from the University of Iowa). The resulting relationships will provide a method to evaluate the individual contribution that each variable has the performance of CIR roads. These relationships can be used on other CIR roads in Iowa to evaluate the field performance of asphalt pavements.

If the degree of variability of the observed data is too large to establish significance, neural network analysis may be performed to draw inferences in future research.

CONCLUSIONS / RECOMMENDATIONS

It is expected that this investigation will provide answers regarding how aged engineering properties, traffic, and subgrade conditions affect the pavement performance; it is also expected that this research will suggest changes that should be made with regard to design, material selection, and construction in order to improve the performance of future recycled roads. Recommendations will also be provided for future researches regarding field performance of CIR asphalt roads. It is possible that some of these recommendations can be generalized to geographic locations outside of Iowa.

ACKNOWLEDGMENTS

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Characterization of Utility Cut Pavement Settlement and Repair Techniques

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ABSTRACT

Pavement settlement in and around utility cuts results in uneven pavement surfaces, driver annoyance, and further maintenance. A survey of municipal authorities and field and laboratory investigations were conducted to identify factors contributing to the settlement of utility cut restorations in pavement sections. Survey responses from seven Iowa cities indicate that backfill material generally consists of imported granular material, utility cut restorations last less than two years, and quality control during restoration construction is limited. To evaluate the performance of existing utility cuts and construction practices across Iowa, restorations in seven cities were observed. The observations indicate that backfill material varies from city to city, backfill lift thickness often exceeds 12 inches, and backfill is often placed at bulking moisture contents. The backfill materials were investigated in the laboratory to characterize gradation, specific gravity, relative density, and collapse potential. The collapse tests indicate that, at the field moisture contents encountered, the backfill materials have collapse potentials ranging from 5% to 25%. Falling weight deflectometer (FWD) data and elevation shots indicate that the maximum deflection in the pavement occurs in the area around the utility cut restoration. The FWD data indicate a zone of influence around the perimeter of the restoration extending two to three feet beyond the trench perimeter.

Key words: backfill—pavement maintenance—trenching—utility cuts

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INTRODUCTION

Utility cuts are made in completed pavement sections to install electric, water and wastewater utilities, and drainage pipes under roadways. Once a cut is made, a restoration is constructed, resulting in a patched surface on the pavement. Utility cuts not only disturb the original pavement, but also the base course and subgrade soils below the cut. Once the utility is repaired and in place, the cut is backfilled and surfaced. Utility cuts in roadways affect the performance of the existing pavement, as settlement and/or heave occurs in the backfill materials of the restoration. If the backfill materials used are not suitable for the site conditions and not properly installed, the materials will begin to settle relative to the original pavement. Utility cuts have the greatest damaging impact on newly paved streets and therefore reduce roadway life considerably (Department of Public Works 1998). Bodocsi et al. (1995) noted that new asphalt pavement should last between 15 and 20 years; however, once a cut is made, the pavement life is reduced to about 8 years in the area of the patched pavement.

Statistical data reported by the Department of Public Works in San Francisco (1998) are shown in Figure 1. It can be seen that the pavement condition and rating decreases as the number of utility cuts made increases. The rating system is based on the Pavement Management and Mapping System developed for the city of San Francisco, which considers the area of the street, pavement condition, last year the street was paved, and the number of utility cuts (Department of Public Works 1998). For example, the pavement condition score for a newly constructed pavement is reduced from 85 to 64 as the utility cuts increased to 10 or more. In fact, several excavations in pavements by utility companies can reduce road life up to 50% (Tiewater 1997). In some cities, millions of dollars have been spent on maintenance and repairs for utility cuts made in pavements each year (APWA 1997). With the continual growth and need for the repair of utilities, the effect of utility cuts on pavement performance is becoming a larger problem.

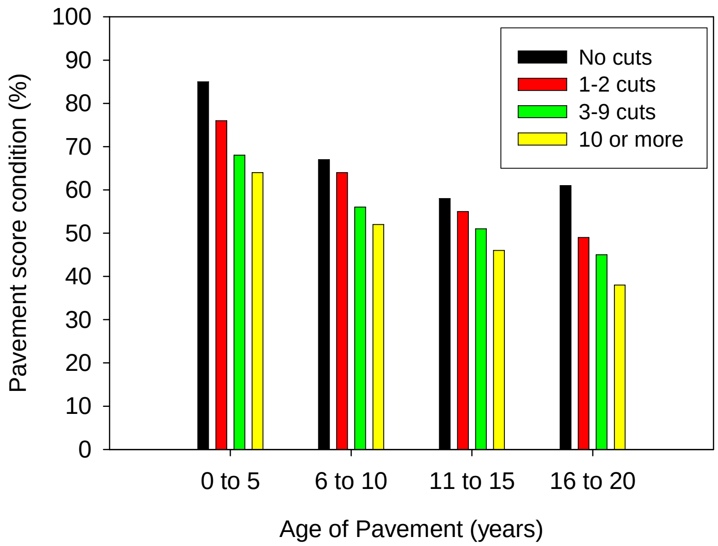


Figure 1. Effects of utility cuts on pavement condition

When a utility cut is made, the native material surrounding the perimeter of the trench is subject to loss of lateral support, as shown in Figure 2 (Department of Public Works 1998). This leads to loss of material under the pavement and bulging of the soil on the trench sidewalls into the excavation. Studies have shown that the affected zone, termed the zone of influence, is two to three feet around the trench (Department of Public Works 1998). Subsequent refilling of the excavation does not restore the original strength of the soils in this weakened zone.

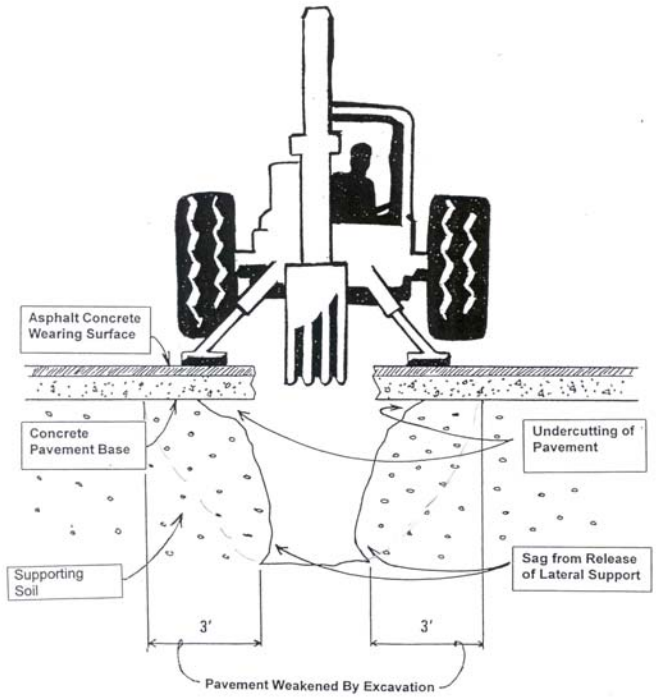


Figure 2. Overstressing of the pavement and natural materials adjacent to the trench

Poor performance of pavements over and around utility trenches on local and state systems often causes unnecessary maintenance problems due to improper backfill placement (e.g., undercompacted, too wet, too dry). The cost of repairing pavements as a result of poorly performing utility cut restorations can be avoided with an understanding of proper material selection and construction practices. Current utility cut and backfill practices vary widely across Iowa, which results in a range of maintenance problems. This research aims to improve utility cut construction practices with the goal of increasing the pavement patch life at an affordable cost, thereby reducing the maintenance needs of the repaired areas.

RESEARCH METHODOLGY

The research was conducted in several stages. A survey was compiled and distributed to several cities in Iowa to determine the extent of the problem and factors needed for consideration. Site visits were made to document the problems at existing utility cuts. Subsequently, construction practices were observed in several cities across Iowa. Several trenches were tested to obtain in situ parameters, such as moisture content, density, and stiffness. Some of these utility cuts were monitored for a year using falling weight deflectometer (FWD) and elevation readings. Furthermore, backfill materials were also tested in the laboratory (i.e., classification, relative density, and collapse potential). The results of these stages are discussed in the following sections.

SURVEY RESULTS

A survey on utility cut standards and performance was developed to determine problem areas that city personnel observe. The survey was sent to major cities across Iowa and responses were received from seven cities (see Jensen et al. 2005). The survey results are generally opinions or estimated values, since in many cases there is little or no documentation, inspection, or measurements kept on utility cut restorations. Somewhat surprisingly, all cities agreed that the standard of practice currently in use for each city provided satisfactory results. However, such agreement may be a result of a varying definition of the term "poorly constructed trenches," since settlement in utility cut restorations was clearly evident in all cities. The survey results also show that trenches that perform poorly often result in reconstruction within two years, which is consistent with previous research (APWA 1997). Determination of the time of year that water line breaks occurred were an estimate in most cases; however, Ames did provide statistical data with January and December the predominate months for occurring breaks. This trend may be a result of frost loading, which could substantially increase vertical loads (i.e., up to twice the original load) on buried pipes (Moser 1990).

A variety of materials were reported as being used in the construction of the utility trenches. Material commonly used include the following: native material, select material containing no organics, Class A crushed stone, material passing 3/4-inch sieve, limestone crusher dust, flowable fills, 3/8-inch minus limestone chips, and manufactured sand. The city of Burlington previously used a sand backfill material, but due to considerable settlement issues, flowable fill called K-Krete is used currently. K-Krete has been used in Burlington for seven years and continues to perform well.

In general, all the cities follow some standard of practice, such as Iowa Statewide Urban Design Standards and Specifications (SUDAS), or AASHTO, for compaction of backfill materials. Dubuque, Waterloo, and Des Moines require 95% of Standard Proctor compaction, whereas Davenport requires 90% Standard Proctor to 18 inches below the finished grade and 95% above this region. Although moisture content is the most important parameter in the proper compaction of soils, only Des Moines provides guidance on the compaction water content, indicating that it should be at or near the optimum water content.

In general, if trench backfill and the pavement restoration are properly constructed, a life of 15 to 20 years is expected. The results of the survey indicate that utility cut restoration lasts much shorter than this. Failures of patched cuts have occurred in as little as one week, with typical responses of two to five years before maintenance of the restoration. Cities reported poorly performing patches occurring in the range from about 1% up to 40%. Problems typically reported include improper bedding and backfill operations, construction during winter and adverse conditions, poor compaction, improper backfill materials, use of native materials with high water content, and difficulty in obtaining compaction in a confined space.

PROBLEMS AT EXISTING UTILITY CUTS

A site in Ames provides an illustration of the problem, where settlement of over one inch occurred. Figure 3 shows the site and settlement that was induced as a result of the utility cut restoration. No documentation of the restoration at this site was found, so subsurface materials and construction practices are unknown. Figure 4 shows measured elevations across the site, which indicates the settlement that has occurred on the site with respect to the existing pavement. It can be seen that the differential settlement of this 14-foot section relative to the undisturbed pavement ranges from 0.3 inches to 1.5 inches.

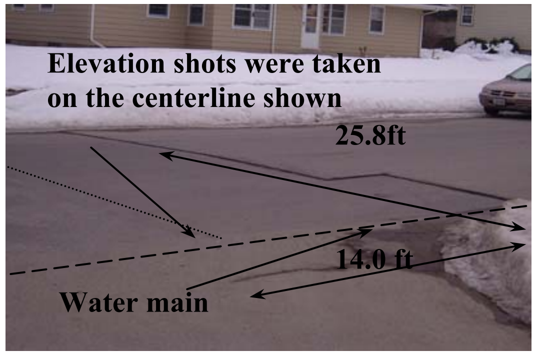


Figure 3. Utility cut at the intersection of Wilson Ave. and 16th St. in Ames, IA

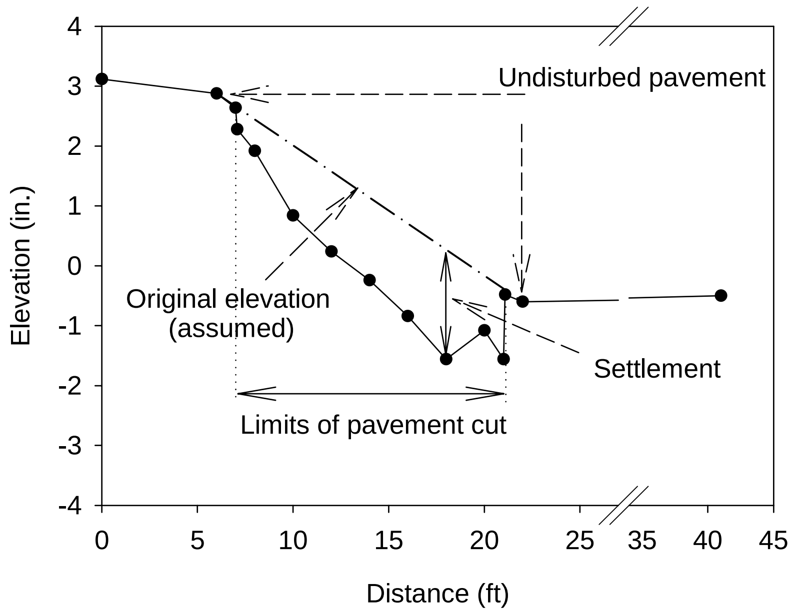


Figure 4. Cross-section of utility cut restoration settlement on Wilson Ave. and 16th St. in Ames, IA

FIELD OBSERVATIONS

Seven cities in Iowa were visited for documentation of current practices. Of these seven cities, restorations in four cities were tested in the field for in situ parameters. These cities included Ames, Cedar Rapids, Davenport, and Des Moines.

In general, the restoration of a utility cut involves a cut, excavation, repair, and compaction of backfill materials. Items of general concern in the backfilling process include backfill materials, lift thickness, placement water content, and desired density. In the field, several practices were observed that should be avoided during trench construction. The cities' required lift thicknesses range from 6 inches to 12 inches. However, such lift thicknesses were not observed in many cases. A lift thickness of two to four feet dumped into the trench was observed. By applying such lift thicknesses, the ability of the material in the lower portion of the lift to reach proper compaction is reduced, as depicted from dynamic cone penetrometer tests performed at four test sites (Jensen et al. 2005). Furthermore, backfill materials were placed at natural moisture contents with no moisture control. Another common practice observed was incorporating refuse, such as leaves and pop cans, into the trench. Such additions to the excavated area increase potential void development and therefore induce potential settlement. Finally, in several instances the area surrounding the excavated area is cleaned into the trench, including material that has been saturated. This practice should be avoided, since saturated material is difficult to compact.

A cutback or T-section is when pavement is removed past the limits of the trench excavation. Such sections have been found to be advantageous in the construction of trenches, alleviating the effects of the lateral support loss in the trench due to the excavation (Peters 2002). Peters (2002) indicates that a T-section with two to three feet of pavement cutback is now constructed adjacent to the excavated area as a bridge over the zone of influence. The field observations during this study revealed that a cutback or T-section was rarely used. In most Iowa cases, a T-section was constructed due to material eroding under a plated trench or the need to shape trench edges. In both cases, the pavement was cut back, but no further compaction was completed in this area.

LABORATORY AND FIELD RESULTS

Several materials were observed for use as backfill material, with differences generally based on regional availability (see Figure 5). Laboratory tests conducted on these materials include specific gravity, sieve analysis, relative density, and collapse index. Table 1 summarizes the classification of backfill materials used at four cities across Iowa, using both AASHTO and Unified Soil Classification systems. Figure 5 indicates that granular backfill materials are used by each city.

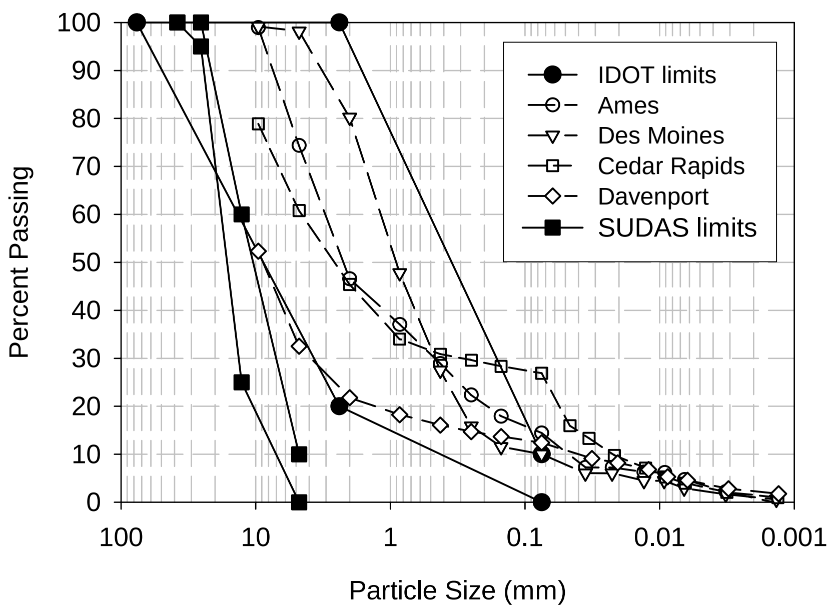


Figure 5. Particle size characteristics of backfill materials used by several Iowa cities compared with Iowa DOT backfill gradation requirements

In addition to classification tests, relative density (ASTM 4253 and ASTM 4254) and collapse index tests were conducted. Relative density tests were conducted at several moisture contents to determine the materials’ bulking moisture content range. At this moisture content range, the soil develops an apparent cohesion where water film develops between soil particles because of capillary tension, forming an open soil structure. Capillary tension forces resist compaction, giving an indication of well-compacted soil, despite the low soil density due to the open soil structure. As moisture content increases, the capillary forces decrease, resulting in the collapse of the open soil structure (i.e., settlement).

Table 1. Imported sample classification

City (sample)	Soil Classification AASHTO		USCS	
Ames (3/8)	A-1-a	Stone fragments, gravel, and sand	SM	Silty sand
Cedar Rapids (3/4)	A-2-4	Silty clay, gravel, and sand	SC	Silty clay
Davenport (3/4)	A-1-a	Stone fragments, gravel, and sand	GC	Clayey gravel
Des Moines (manufactured sand)	A-1-b	Stone fragments, gravel, and sand	SW-SM	Well-graded sand or silty sand

The results of the relative density test for backfill material used in Ames are shown in Figure 6, which indicates a bulking moisture content range of 6% to 8%. Collapse index tests performed on the same material, which was placed by first dropping the material from a height of three feet without applying any compaction effort and then flooding the material, indicate a collapse potential of about 10% within the range of bulking moisture content (see Figure 6). Table 2 compares the results of field moisture contents with the bulking moisture content range determined from the relative density tests. This table shows that all materials were placed at moisture contents within the range of bulking moisture content. Material placed slightly under this bulking moisture content could lead to a collapse due to infiltration or a rise in the groundwater table, thereby inducing settlement in the trench. Table 2 shows the field moisture contents compared to the bulking moisture contents obtained from laboratory tests.

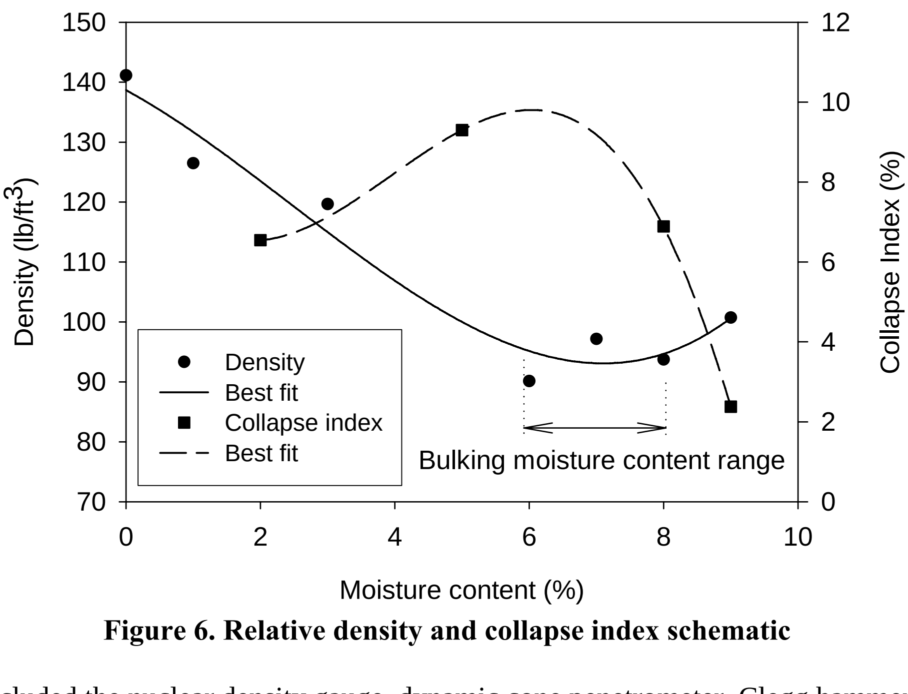


Figure 6. Relative density and collapse index schematic

Field testing included the nuclear density gauge, dynamic cone penetrometer, Clegg hammer, and Geogauge stiffness tests. These instruments were used to determine in situ parameters, such as dry density, moisture content, stiffness, and penetration values. The nuclear density gauge was of particular importance in the research, allowing relative density and moisture contents to be determined. Space limitations preclude further discussion of the results of these tests. Further information can be found in Jensen et al. (2005).

Table 2. Field moisture content range compared with bulking moisture content range

Sample	Classification	γ_{Max} (lb/ft^3)	*W%* (bulking)	*W%* (field)
Ames	SM	140	5 to 8	4 to 5
Cedar Rapids	SC	130	6 to 8	5 to 7
Davenport	GC	140	4 to 6	6 to 8
Des Moines	SW-SM	138	8 to 10	6 to 11

MONITORING TECHNIQUES

Following field testing and construction of the trenches, further monitoring was conducted on three trenches located in Ames, Cedar Rapids, and Des Moines. The monitoring techniques consisted of FWD testing and the determination of elevation profiles using a transit. The FWD testing was conducted by the Iowa DOT and the results illustrate the effect of the zone of influence around the trench. The FWD testing was completed in a three-stage loading setup, which compensates for the various loadings that may be applied as a result of traffic. Figure 7 shows a plan view of a utility cut in Des Moines that was constructed in a concrete roadway. The limits of the excavation and cutback are shown, as are the locations selected for determination of deflection profiles. The FWD deflection profiles for the trench are shown in Figure 8. The large deflections just outside the edge of the trench indicate a weaker soil response from the underlying materials and identify the zone of influence of the trench. The zone of influence was prominent on each monitored site, with larger deflections at the side of each trench than at the center and far field areas of the trench.

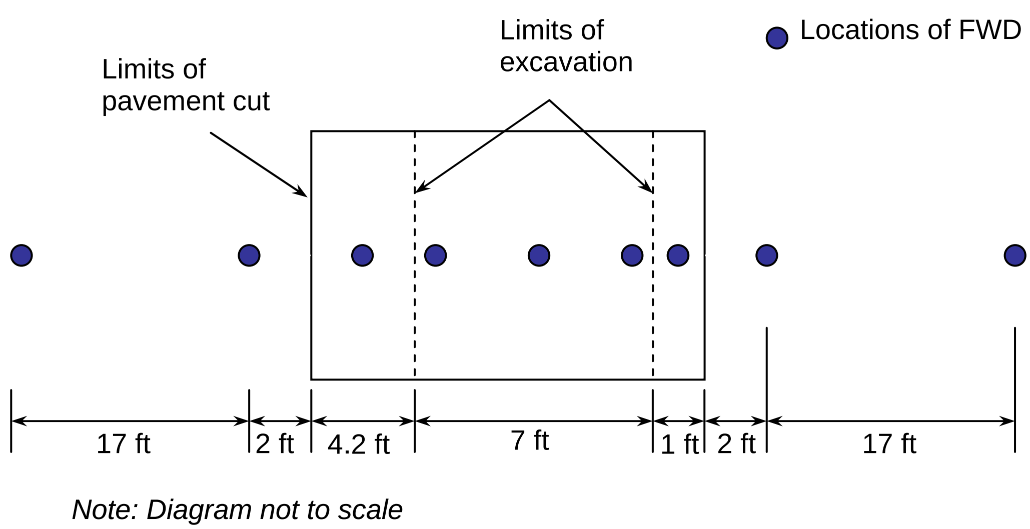
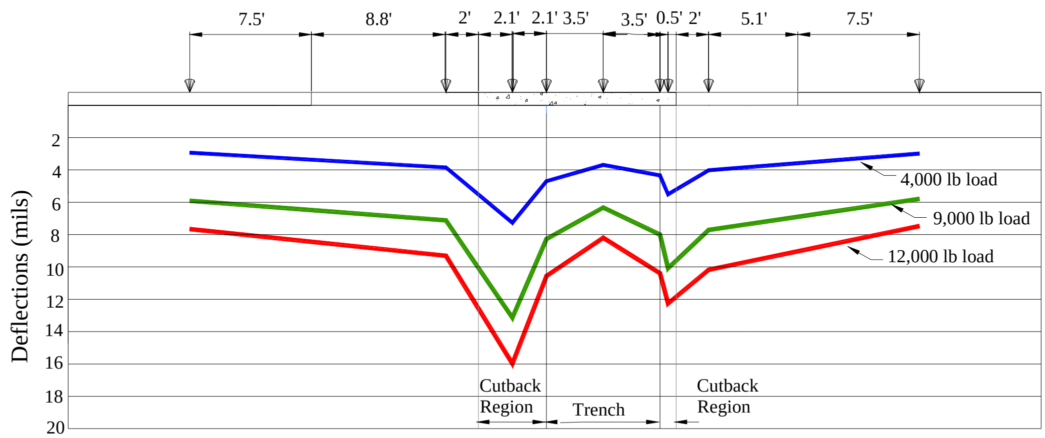


Figure 7. Locations of the FWD tests performed at the utility cut in Des Moines, Iowa



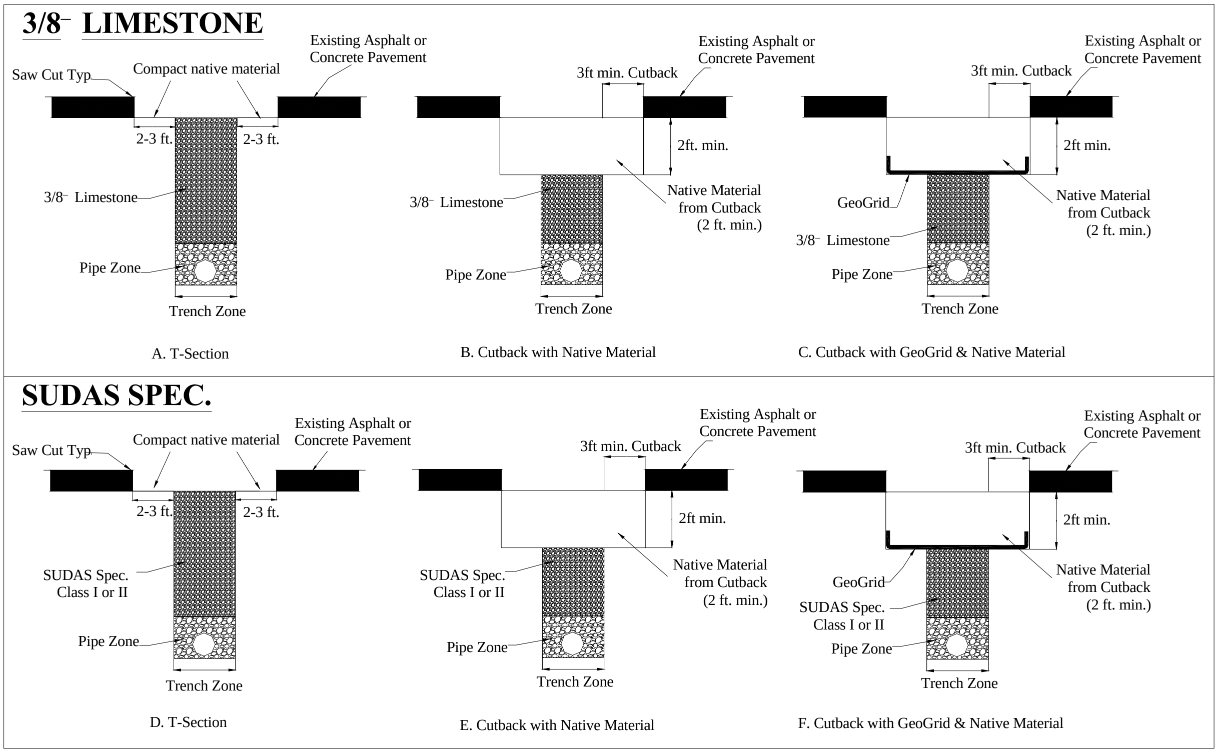
Note: 1) 1 mil=0.001 inches
2) Deflections reflect point where FWD drops weight
3) Weights were dropped on centerline of trench in the longitudinal direction of the pavement

Figure 8. Profile of FWD response performed at a utility cut in Des Moines

TRIAL TRENCHES

Following the laboratory and field studies, several new designs were proposed for further analysis. These proposed designs are anticipated to be constructed in Ames, Iowa during the spring and summer of 2005. These proposed profiles consisted of three trenches using Ames standard backfill of 3/8-inch limestone, three trenches consisting of a new specification designed by SUDAS, and one trench using a flowable fill. Figure 5 shows the gradation of the Ames 3/8-inch limestone and the SUDAS backfill limits.

Three types of trenches were proposed: The first is a standard cut with a two- to three-foot pavement cutback, which should then be further compacted. The second is a cutback two to three feet past the excavation limits and then excavated down two to three feet. Imported material will be placed up to this two- to three-foot cut. Then, unsaturated excavated native material acts as a bridging layer, followed by imported material to the surface. The third is a trench constructed similarly to the second type, except that a geosynthetic material is placed over the cutback and excavated area, adding a bridging support, which is followed by native material. Imported material again may be needed up to the surfacing level (see Figure 9).



Note: 1: Trenches should be constructed to appropriate widths and depths according to site specifications. These cross-sections specify only specific cutback distances, material use, and depth of compacted native material.

2: If native material is not sufficient to fill trench, imported material (3/8- limestone or SUDAS Spec.) should be used to fill trench to the surface.

Figure 9. Trial trenches to be constructed in Ames, Iowa

RECOMMENDATIONS

Based on the field observations and measurements and laboratory testing, the following recommendations can be made:

1. Proper compaction is generally determined according to Standard Proctor compaction in most cities. However, relative density should be used to determine granular compacted material (Spangler and Handy 1982). Figure 10 (Spangler and Handy 1982) illustrates the misleading results that occur in the compaction of granular material when using the Standard Proctor test (AASHTO T99 Compaction). This is also evident in that the Standard Proctor uses a dropping hammer, whereas the relative density test uses vibration for compaction, with vibration typically the method of field compaction. When determining compaction based on relative density, a value of 65% or greater should be obtained to achieve a densely compacted material (Spangler and Handy 1982).
2. Moisture is one of the most important parameters in material evaluation in geotechnical engineering. Throughout this research, moisture has proven to be an important factor in utility cut restorations. As a result of relative density tests, the bulking moisture content region for each sample was obtained. It is recommended that the restorations be constructed at moisture contents exceeding this bulking moisture content region. Since in many cases the imported material was placed at or slightly below this region, it is important to water the material on site. The material as placed will then overcome the collapse potential that could be induced on the pavement patch as a result of infiltration or an elevation in the groundwater table.

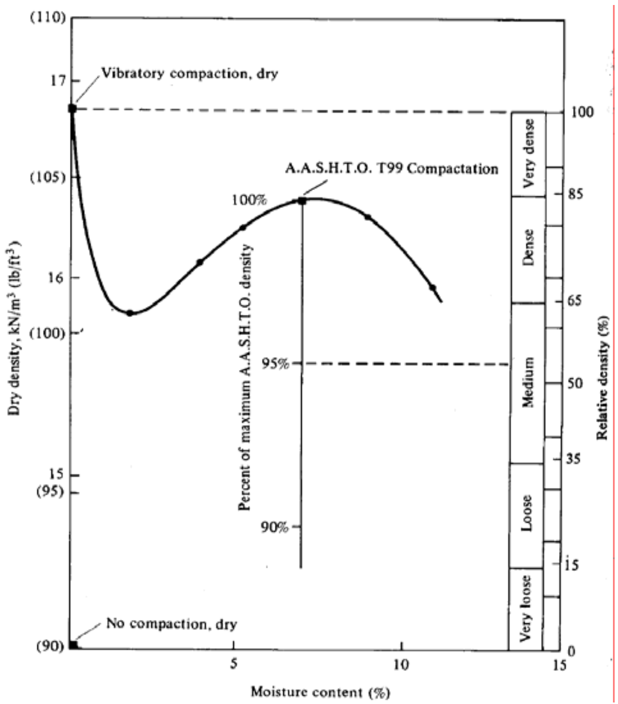


Figure 10. Relative density vs. AASHTO T99

3. It was observed that instrumentation and quality control was rarely used to assure that standards and proper construction procedures were being met. It would be advantageous to incorporate the nuclear density gauge into a quality control program. This device requires a certified user due to the radiation that is emitted during the use. However, this device would provide both a dry density and moisture content that can be used to ensure properly constructed backfill materials in the trench, specifically for comparisons to relative density test results for granular materials. The nuclear density gauge (and/or dynamic cone penetration test) would also be useful for determining whether the moisture content has exceeded the bulking moisture content region.
4. The zone of influence has proven to be a critical factor in the construction of these utility trenches because of the loss of lateral support in the trenching limits. To compensate for the weakened material in this zone, it is recommended that a pavement cutback of two to three feet be constructed. The pavement cutback and excavated area should be compacted again with a vibrating plate before the pavement surfacing is placed.
5. A continuation of this research should be conducted to monitor the performance of the constructed trenches. According to previous studies, a restored trench will begin to show signs of settlement as early as one to two years; therefore, to determine the performance of the trenches accurately, monitoring should continue for a minimum of one additional year.

CONCLUSIONS

This study has identified several key factors in the construction of utility cut restorations, including the weakened zone of influence around the trench and difficulties with material placement density and moisture content. Several recommended courses of action are proposed to reduce the influence of these factors. Pavement cutbacks and T-sections are recommended to reduce the influence of the weakened materials at the edge of the trench. Placement of backfill material at higher moisture contents is recommended to avoid collapse potentials associated with bulking moisture contents. It is also recommended to move towards a relative density specification rather than the traditional Standard Proctor density definition, because relative density is a better indicator of density in granular materials.

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MnROAD Overview and Update

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ABSTRACT

The Minnesota Department of Transportation (Mn/DOT) constructed the Minnesota Road Research Project (MnROAD) between 1990 and 1994. The MnROAD site is located 40 miles northwest of Minneapolis/St. Paul and is an extensive pavement research facility consisting of two separate roadway segments containing 50 500-foot–long test cells. The 3.5-mile mainline test roadway, containing 31 test cells, is part of westbound interstate 94 and carries an average of 24,000 vehicles daily (14% trucks). Parallel and adjacent to the mainline is a low-volume roadway that is a 2.5-mile closed loop containing the remaining 19 test cells. LVR traffic is restricted to a MnROAD-operated 18-wheel, 5-axle tractor/trailer with two different loading configurations of 102 kips and 80 kips. Subgrade, aggregate base, and surface materials, as well as geometric design methods, vary from cell to cell. Daily information is gathered via a computerized data collection system that monitors over 4,500 mechanical and environmental sensors. All data collected is entered into the MnROAD database available to Mn/DOT and other researchers free of charge. More information can be obtained from the MnROAD web page: http://mnroad.dot.state.mn.us/research/mnresearch.asp.

This presentation will review MnROAD's existing resources, current test cells available for reconstruction, seven key research topics, and the expected actions needed to accomplish the objectives for MnROAD's next phase, including how to participate in the second phase of MnROAD.

Note: Contact the presenter above for more information on this topic.

Key words: MnROAD—pavement research facility

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Enhancing Electronic Highway Design Standards and Specifications

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ABSTRACT

Within the last decade, highway design standards in electronic format have been developed by many organizations to help engineers and contractors work without piles of design standard books. Since the Iowa Electronic Reference Library (ERL) was developed for the Iowa DOT, designers, inspectors, contractors, and owners have studied the project requirements, specifications, and standard road plans more easily than before. Users can search or query information they need by using the internet-based ERL instead of reading through piles of books, as in the traditional research process. However, to make the ERL more usable, two areas of development are provided in this paper: (1) a framework for future development, and (2) recommendations for enhancing the existing system.

(1) The existing ERL can be more powerful and useful if it is always available from desktop computers and cell phones, or if it is used between web browsers and CADD software (e.g., Microstation or AutoCAD). Users would always have data when they need it. A designer could embed a road specification and a standard road plan into a drawing simply by clicking an icon in Microstation, and contractors can then read the drawing and retrieve the specification and standard by clicking another icon. However, ERL is currently a standalone application that is difficult to integrate with other software. An approach to this problem is to develop a new system on top of the existing ERL. The ERL2 model framework will be introduced in this paper under the concepts of reusability, availability, and extensibility with new web standards and technologies based on Extensible Markup Language (XML) and Resource Description Framework (RDF). The ERL2 architecture is composed of a programming model, a metadata model, and a data model, which are available to be developed separately. The new system will be used for major CADD software and software that implements specifications and standards, such as the Object-Oriented Design and Specification (OODAS) being developed at Iowa State University. The expected benefits of this research include saved work-hours and money.

(2) Currently, the ERL only supports the Microsoft Windows operating system with Microsoft Internet Explorer. Widely used mobile computers, such as PDAs or internet-based cellular phones, and Macintosh computers cannot interpret ERL data properly. To support other users and improve usability in the field, the ERL should be standardized. Other enhancements, including improving the search module and user interface, are also discussed.

Key words: data management—design standards and specifications—information and intelligent systems—knowledge management

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INTRODUCTION

Within the last decade, highway design standards in electronic format have been developed by many organizations to help engineers and contractors work without piles of design standard books. Some organizations provide their standards on CD or make them available over the internet. Some incorporate illustration design standards, animations, and even a search function into their standards. The Washington Asphalt Pavement Association (WAPA) provides interactive animations instructing users about the asphalt design standards. The Iowa DOT provides design standards and drawing plans with a search function on its website. Organizations are migrating to electronic formats because electronic documents are easy to use, share, and maintain.

This paper provides an electronic highway specification development framework based on the Iowa Electronic Reference Library (ERL). Since there have been no major changes from its initial development in the past five years, the ERL uses the same technology. Many new technologies have come out in recent years. New technologies are explained that can adopt web services to enhance and improve the capabilities of electronic standards and specifications, including ERL. This paper describes and evaluates the existing Iowa ERL framework and provides (1) a framework for future development, and (2) recommendations for enhancing the existing system.

Background of the Iowa ERL

The Iowa ERL was developed to provide access to Iowa design standards and specifications, in electronic format since 2000. It is available both online, on the Iowa DOT's website, and in CD format; however, a traditional hardcopy is still published. The ERL has been modified and published twice a year. The main contents in the latest version (April 2005) include standard specifications, supplemental specifications, material instructional memoranda, standard road plans, standard culvert plans, standard bridge plans, the construction manual, the flagger's handbook, and statewide urban standard specifications. They are all stored in HTML and PDF formats. Besides these contents, there are other features. The navigation feature provides navigational link menus on the left-hand panel for selecting different sections. The search feature provides a box that can find a word or phrase in all design standards. Phonebook listings also provide up-to-date contact lists of Iowa DOT officers and engineers. The ERL system has been created and maintained based on HTML, PDF, and Javascript by Microsoft FrontPage, version 5.0. However, it only supports the Microsoft Internet Explorer browser, version 4.0 or later, with the Microsoft Windows platform.

Success of the ERL

The first online ERL survey (Cetin 2001) showed ERL's success with its users, including highway contractors, designers, and Iowa DOT officers, after they had migrated from paper to electronic format. Many field engineers prefer to use the ERL because of the ease of field use for people carrying only a computer laptop with a CD drive, rather than huge specification books. Users also preferred the search and navigation features. The bottom line for users was the up-to-date information provided every six months, displayed in red or strike-through text. This feature warned users of changes, preventing misunderstandings due to conflicts between different specifications.

IMPLEMENTATION OF THE ERL2 MODEL FRAMEWORK

Although the ERL is updated every six months, the ERL updates still use the same technology used in the initial development of the ERL in 2000. This paper introduces new technologies, called the ERL2 model framework, to enhance the capabilities of the existing system.

The major benefits of adopting the new technology are as follows:

- The new system will follow the standards of the World Wide Web Consortium (W3C), which makes data available in any device, from existing desktop computers to internet cellular phones or even new devices that come out after the system is developed.
- The existing data will be ready to use in any software, including CADD software (e.g., Microstation or AutoCAD) or newly developed software.
- The only maintenance issue will involve updating the design standards and specifications, rather than HTML technical issues.
- A glossary will be used and easily created from the existing files.
- The new system will reduce redundant information between design standards and specifications from different years.
- Information about general usage will be gathered automatically for analysis during further developments. Statistical data, including keyword searches, frequency of use, and types of user operating systems or browsers, will be collected.
- The system will follow W3C accessibility standards.

The use of some of the technologies listed here on the existing system without making major changes is discussed at the end of this paper.

ERL2 SYSTEM ARCHITECTURE

The ERL2 system development is based on three main concepts: reusability, availability, and extensibility. Reusability means that data created one time will be available in any format. Availability means that data will be available under any circumstances, such as cell phone access. Extensibility means that data will ideally be infinitely expandable. Ultimately, all data management will be organized by an automated system to reduce costs and human error. However, human reviews will still be needed for the contents.

In 2001, Li demonstrated that an automated system can generate hyperlinks using the Perl and Gwak programming languages and thus decrease labor/maintenance costs. At that time, automated functions did not work well because data was still unstable, and the system could not generate 100% of the data. Cetin (2001) introduced the internet database approach to managing ERL data. In July 2002, Wikipedia, an open-content encyclopedia on the internet, provided an infinitely extensible encyclopedia using software named MediaWiki. MediaWiki (2002) is data management software that works by programming model reading contents, which are kept separately. The results are then generated in plain HTML. This approach has also been used in various types of software, due to the advantages of smaller data size and data reusability. MediaWiki also introduced [[ ]] (double square brackets) to replace the full HTML tags, saving editing time and reducing typos.

Design Approach for the ERL2 Model Framework

The ERL2 will include three models: a programming model, a metadata model, and a data model (Figure 1). The programming model includes the programming coding and user interface. The metadata model will be used as a bridge between the programming model and the data model. The data model will include the main contents (design standards and specifications).

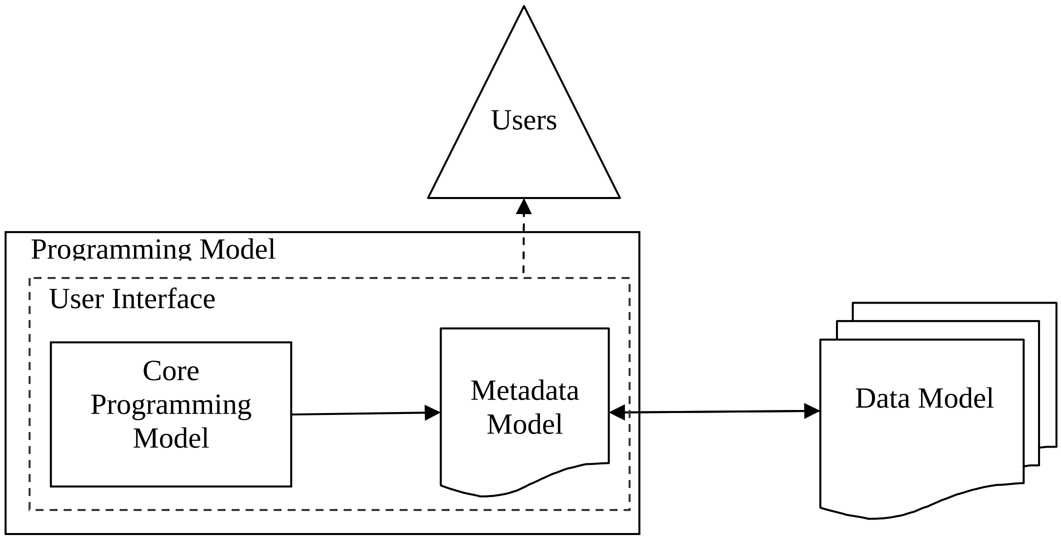

Figure 1. ERL2 model framework

The ERL2 system works by using the programming model as a core. It requests data from data model via the metadata model and then renders the requested data into plain HTML, depending on each user's interface.

Programming Model

The programming model develops the major contents of the ERL2 system, including publishing and maintaining data. The core system works as software that reads and edits the data model.

The user interface will be flexible, changing due to data usage (Thinkmap Inc. 2004). Figure 2 illustrates three different uses of the same data source in ERL2. The spider chart (top left) displays the relationship between each data node. The closed-relationship data nodes are displayed for every data node, so users can navigate quickly. In this example, the data node relates to ERL displays. The clustering chart (top right) shows a data relationship similar to that depicted in the spider chart, but emphasizes the types of relationships by displaying the data in different colors and sizes. In this example, the dark color represents the deeply detailed data. The chronology chart (bottom) shows the data in different timelines, which, in this example, shows standards with different update times. Different user interfaces can be added later without creating a new core system.

Although the programming model is designed for a web-based platform, CADD applications, such as Microstation or AutoCAD, can also access the data model with different user interfaces. Moreover, the Object-Oriented Design and Specifications project currently being developed at Iowa State University is able to access this data model.

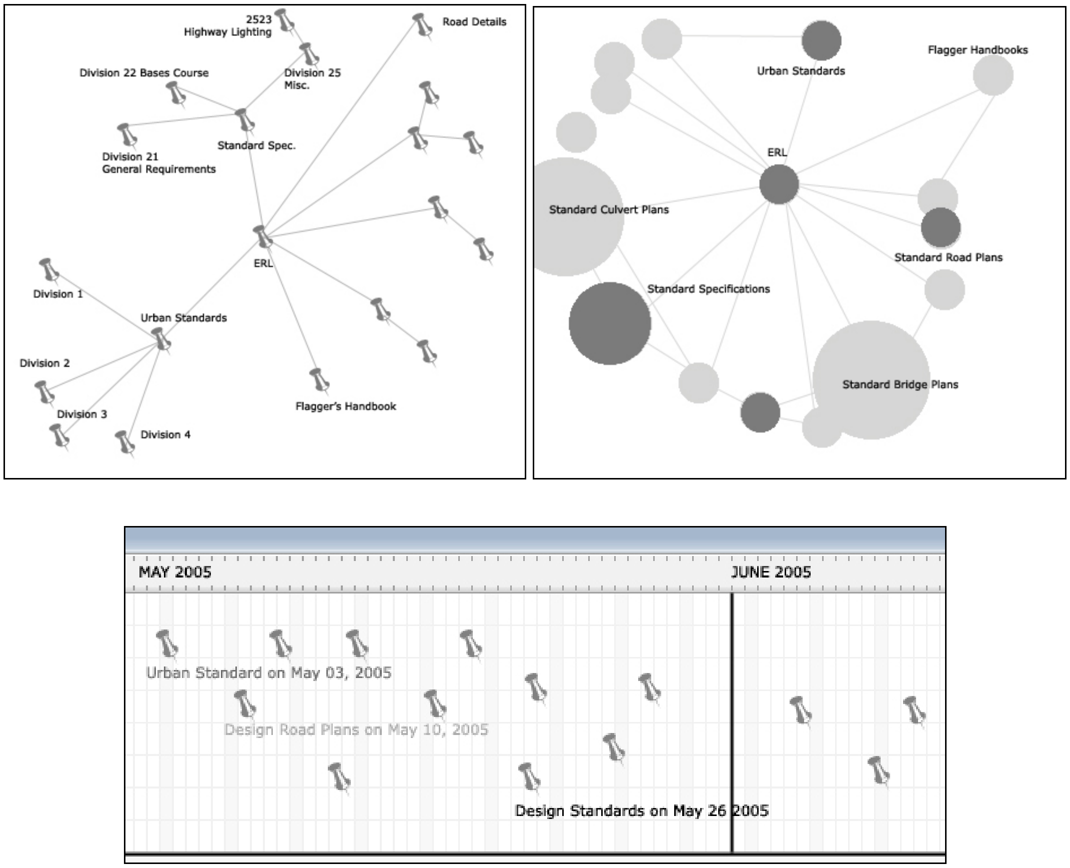

Figure 2. Illustration of different user interfaces for the same data source

Metadata Model

The metadata model explains the data model to the programming model. Instead of using the programming model with the data model, the metadata model organizes the data and decreases the complexity of both the programming model and the data model. Unlike other standards, such as Industry Foundation Class, TransXML, or LandXML, the metadata model does not itself define data. It defines data and the data model location as a resource description framework (RDF), which is a schema in XML.

Figure 3 shows a simple example of metadata in HTML, the code in the left column, and the rendered result in right column. HTML tags, <h1>, <h2>, <h3>, and so forth, are used to define heading formats, shown in the right column. At the same time in the back-end engine, content formatted in <h1> and referring to heading 1 is classified as more important than the content under <h2> and <h3>. The contents are then classified themselves by the metadata categories <h1>, <h2>, and <h3>.

```
<h1>Iowa ERL</h1>
   <h2>Design Standards</h2>
      <h3>Concrete Pavement</h3>
   <h2>Standard Specifications</h2>
      <h3>General Requirement<h3>
      <h3>Equipment Requirement</h3>
```

Iowa ERL
Design Standards
Concrete Pavement
Standard Specifications
General Requirement
Equipment Requirement

Figure 3. Example of metadata heading categories in HTML

XML, instead of using heading tags as metadata, defines its tag names more meaningfully than HTML tag names. That is, the same data can be used in several more ways than data in HTML format. Figure 4 shows the same metadata as Figure 3, but in XML format.

```
<Title name="Iowa ERL">
      <Book name="Design Standards">
            <Subject>Concrete Pavement</Subject>
      </Book>
      <Book name="Standard Specifications">
            <Topic>General Requirement</Topic>
            <Topic>Equipment Requirement</Topic>
      </Book>
</Title>
```

Figure 4. Example of using XML as metadata

RDF, an XML technology, is used in the ERL2 model framework. It allows the sharing and reusing of data between different applications or enterprises. It integrates a variety of data using the XML syntax for pointing to specific data locations, instead of explaining data (see Figure 5). With this framework, the data model is kept separate using the RDF metadata model, which is well-organized and reusable.

```
<RDF>
      <Description about="Iowa ERL Standards
                           and Specifications">
            <Title>Highway Lighting</Title>
            <Location> GS/content/2523.htm</Location>
      </Description>
</RDF>
```

Figure 5. Example of RDF as metadata

Other XML schemas that can be used in the ERL2 model framework are Extensible Style-sheet Language (XSL) and Document Type Definition (DTD).

Data Model

The data model consists of two different parts: contents and a data tag. Contents are plain text or graphic representations of highway design standards and specifications, while a data tag is plain text that defines a feature of the contents. The contents, modified by specification engineers and updated twice a year, as mentioned, include text, images, drawing plans, graphs, tables, media files, and animations. This paper shows examples of the glossary and hyperlink features.

Glossary Feature

The data tag "[[Glossary:]]" (double square brackets containing the attribute "Glossary") is added to the contents to define words or phrases in the contents. The glossary tag defines different words or phrases, creating the glossary links that appear in the results. When users select the tagged contents, the glossary is displayed instead of a hyperlink to other pages, as with the traditional method. Figure 6 displays examples of glossary tags used for contents. The code is in the left column and the rendered result is in the right column.

…All apparatus, materials, and work shall be in accordance with the contract documents and with standards, practices, and codes of the electrical industry. Particular attention is directed to the following. ▪ [[Glossary:National Electric Code (NEC)]], latest edition, including amendments; ▪ [[Glossary:IEEE Standards]] and Practices; The completed lighting installation shall conform to all local and special laws, codes, or ordinances of all Federal, State, and [[Glossary:municipal authorities]] with due jurisdiction. Lighting materials shall meet requirements of [[Division 41]].	…All apparatus, materials, and work shall be in accordance with the contract documents and with standards, practices, and codes of the electrical industry. Particular attention is directed to the following. ▪ National Electric Code (NEC), latest edition, including amendments; ▪ IEEE Standards and Practices; The completed lighting installation shall conform to all local and special laws, codes, or ordinances of all Federal, State, and municipal authorities with due jurisdiction. Lighting materials shall meet requirements of Division 41.

Figure 6. Glossary data tags in the data model

Hyperlink Feature

Similar to the glossary feature, the hyperlink feature creates links by using double square brackets. The current version of the ERL, using plain HTML code, defines a link in the contents as follows:

Section 2523 Highway Lighting

In the ERL2 model framework, double square brackets are used instead, as in the following example:

[[Section 2523]] Highway Lighting

Although the outputs from the HTML code and the double square brackets of the ERL2 framework show the same result, the double square brackets help organize data and reduce data size. Furthermore, the amount of typos and maintenance time will decrease significantly. Other data tags in the ERL2 framework include, but are not limited to, the following:

- `[[___]]` – for same-specification links
- `[[Glossary:___]]` – for the glossary
- `[[GS:___]]` – for links to general specifications
- `[[US:___]]` – for links to urban standard specifications
- `[[IM:___]]` – for links to material instructional memoranda

Maintenance Issues

Because of the automated system for the ERL2 model framework, maintenance issues will only include updating the design standards and specifications from the specification engineers, and not HTML coding. When new highway design standards and specifications are released, the old programming and metadata models will work with the new data model. Even for the next generation, when the new programming model must be changed, the same metadata model and data model can be used without changes.

POSSIBLE CHANNELS AND SOLUTIONS FOR ENHANCING THE EXISTING ERL

Adopting new technologies within the existing ERL will enhance some of its present capabilities. The possible channels and solutions for enhancing the capabilities of existing system are explained below.

Web Standards

The current ERL only supports the Microsoft Windows operating system with the Microsoft Internet Explorer browser, as mentioned in the ERL manual since its initial development. However, users who are not using the Microsoft platform cannot access and use the ERL properly. Widely used browsers such as Netscape (including AOL) and Mozilla Firefox, mobile computers such as PDAs or internet cell phones, and Macintosh computers are not able to interpret ERL data properly. Figure 7 shows the layout mismatch that happens on browsers other than Microsoft Internet Explorer. These standard browsers (Netscape, Mozilla Firefox, Safari, and Opera) do not render the first page of the ERL correctly. To support users who do not use Microsoft products and to improve usability in the field, a standardized ERL data system is a cost effective approach.

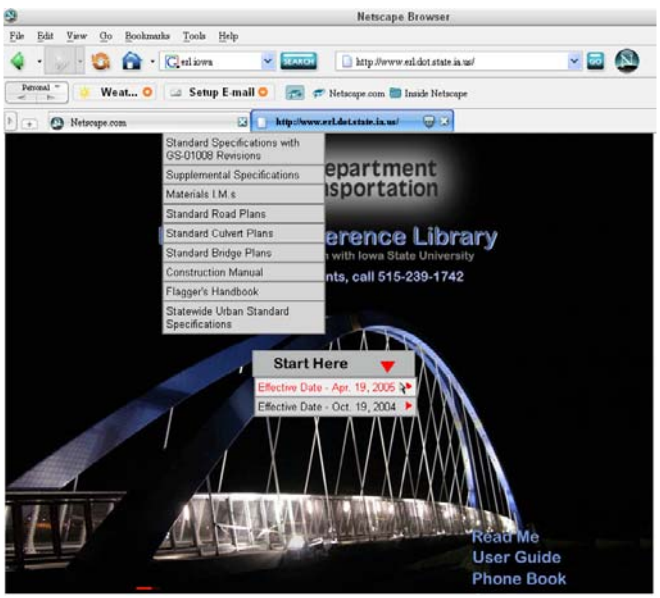

Figure 7. Netscape browser mismatch rendering

Other minor difficulties with accessibility standards include font sizes and font colors. These problems can be diminished by supporting accessibility standards and testing the ERL in standard browsers with both desktop and laptop computers. Besides the standards, the text and table positions should be in relevant size or absolute size, which is less than 240 pixels, to allow mobile users to read tables easily in one page width.

Search Technology

The ERL provides a search function based on the Javascript programming language. This feature helps users easily search specific keywords in an entire section of the specification book. However, publishing both online and in CD format, the Iowa DOT needs to pay the annual license fee for the number of CDs distributed, a major maintenance cost. In the past few years, many search engines have entered the market. They improve search capabilities, providing a faster search at a lower cost. Some of them support a natural language search so users can find information by asking questions in plain sentences. Many of them do not require annual fees to use and publish. Using a new search engine for the ERL will improve its capabilities and at the same time decrease costs in the long term. The following companies, though not limited to these, provide search engines:

- Macromedia RoboHelp provides an online and offline (CD format) search system
- Microsoft HTMLHelp provides a free offline search system
- Google provides a free online search system

Index Module

The index module is similar to a book index, which functions as a pointer to a specific section. The index will help users search for specific keywords, which are human-generated and thus more exact and appropriate than search engines generated by computers. Moreover, an index created by people in the same field as the users, e.g., highway engineers, will help users find the right data better than keywords in a search engine. Adding an index into the current system is a bonus to the existing ERL. Figure 8 shows an example index page in the ERL system. Furthermore, an index will be more useful when adapting data into future developments, such as object-oriented data.

Tree-View Module

The example of the tree-view module, shown in Figure 8, illustrates the expandable menu and submenu. This menu type helps users browse the system easily from section to subsection and vice versa without getting lost.

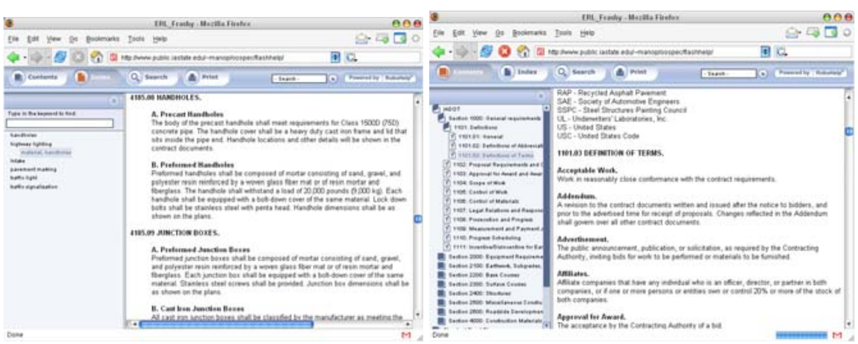

Figure 8. Examples of an index module (left) and tree-view module (right) created by RoboHelp

Test System for Enhancing the Existing ERL

The test system runs on Windows XP, Pentium III 800 MHz, RAM 640 MB, with Internet Explorer 6, Netscape 8.0, and Mozilla Firefox 1.04 browsers. An interactive example is available at the author's web site at http://www.public.iastate.edu/~manop/oospec/flashhelp/.

The example was created for illustrative purposes by RoboHelp X5. The search and index features are limited because of the software demo version. The Iowa DOT general specifications are excerpted only.

CONCLUSION

The ERL2 model framework continues from the success of the existing ERL system. The new system has been created with newly introduced technologies under the guiding concepts of reusability, availability, and extensibility. The three components of the ERL2 architecture, the programming model, metadata model, and data model, are able to develop separately under W3C standards. The primary purpose is to help users work faster by accessing data anywhere using state-of-the-art technologies.

An enhancement for the existing system has also been provided in this paper. Many approaches are provided, including standardizing the contents and improving the existing search engine. Minor enhancements for improving the ERL user interface and offering a tree-view feature were also introduced.

While this paper has explained a system based on the Iowa DOT ERL, other design standards or data management approaches can adopt the same technologies to follow the ERL2 model framework.

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Mix Design Development for Pervious Concrete in Cold Weather Climates

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ABSTRACT

Portland cement pervious concrete (PCPC) is increasingly being used in the United States in sidewalks and parking lots due to its benefits in reducing the amount of runoff water and improving water quality. In the United States, PCPC typically has high porosity and low strength, which has resulted in limited use of pervious concrete in hard wet freeze environments (i.e., the Midwest and Northeast). The purpose of this research is to develop a PCPC mix that not only has sufficient porosity for stormwater infiltration, but also desirable strength and freeze-thaw durability. In this research, concrete mixes were designed with various sizes and types of aggregates, water-to-binder ratios, binder contents, and amounts of admixtures. Porosity, permeability, strength, and freeze-thaw durability of these mixes were evaluated. Preliminary research results indicated that the use of single-sized aggregate could provide concrete with high porosity but not adequate strength. Above a certain level, increasing cement content resulted in a reduced concrete porosity with insignificant influence on concrete strength. Using sand and latex significantly improved the workability of PCPC and resulted in higher strength, appropriate permeability, and freeze-thaw resistance at 180 cycles to date.

Key words: freeze-thaw—permeability—pervious concrete—strength

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INTRODUCTION

Background

Recent amendments of the Clean Water Act require reductions in the quantity of stormwater runoff and also some amount of initial water quality treatment. The United States Environment Protection Agency (EPA) regulates and monitors compliance by granting National Pollutant Discharge Elimination System (NPDES) permits. The new stormwater policy was implemented in two phases: Phase I NPDES required monitoring and treatment of stormwater by municipalities of 100,000 or greater, industries, and construction sites of five acres or more; and Phase II NPDES increased accountability to municipalities greater than 10,000 people and construction sites greater than one acre (EPA 1996). Individual state departments of natural resources (DNRs) have been charged with enforcing the stricter standards.

One of the methods excepted to reduce the volume of direct water runoff from pavement and enhance the quality of storm water is pervious concrete (Water Environment Research Foundation 2005). In the United States, the application of pervious concrete is limited to sidewalks, parking lots, and low traffic density areas. The void ratio and 28-day compressive strength of pervious concrete range from 14% to 31% and 980 psi and 2,830 psi, respectively, with most mixes on the lower side of the compressive strength range (Florida Concrete and Products Association, Inc. 2000; National Ready Mix Association 2004). The National Ready Mix Association (2004) reported the mix design of ten projects where pervious concrete was used in the United States. The mix design used in these projects is summarized in Table 1 (National Ready Mix Association 2004). This table shows that these mixes contain no fine aggregate (i.e., sand) with a water-cement ratio ranging from 0.27 to 0.43. Only one of the reported mixes was installed in a hard wet freeze environment (i.e., the Midwest and Northeast United States) with an average temperature below freezing for 90 days. Although pervious concrete used in the United States has a high void ratio, the 28-day compressive strength is less than that required for structural concrete (i.e., 3,000 psi) (Kosmatka et al. 2002). Low strength values and lack of freeze-thaw resistance has limited the use of pervious concrete in the wet freezing regions (i.e., midwestern and northeastern United States).

This paper summarizes the results of the research performed at the Portland Cement Concrete Research Laboratory at Iowa State University to develop a freeze-thaw resistant pervious concrete that has the required compressive strength and adequate permeability.

Table 1. Typical mix design of existing PCPC

Material	Amount/yd^3
Cement	300–600 lbs
Coarse aggregate	2,400–2,700 lb.
Fine aggregate	0
Water/cement	0.27–0.43

Research Scope

To improve strength and maintain the appropriate void ratio for the PCPC developed in the United States, fine aggregate, supplementary cementitious materials, and admixtures were added to the existing PCPC mixes used in the United States. Mixes that achieved the highest seven-day compressive strength and appropriate void ratio were further tested for long-term compressive strength, permeability, tensile strength, and freeze-thaw durability.

MATERIALS

Mixes were prepared using LaFarge Type I/II cement with two types of single-size coarse aggregate: crushed limestone and river gravel. Three single sizes of river gravel (1/2-in., 3/8-in., and No. 4) and one size of crushed limestone (3/8-in.) were used. The single size of aggregate was defined as the size of the sieve on which 100% of aggregate was retained, but onto which all passed through the sieve above. The properties of these aggregates are summarized in Table 2. The dry rodded unit weight and void ratio of aggregates were measured using ASTM C29. The specific gravity and absorption were 2.62 and 1.1% for river gravel and 2.45 and 3.2% for crushed limestone, respectively (ASTM C127 2003). To improve the strength of single-size coarse aggregate mixes, concrete river sand was added. The sand had a fineness modulus of 2.9, specific gravity of 2.62, and absorption of 1.1% (ASTM C136 2003; ASTM C128 2003).

A styrene butadiene rubber (SBR) latex was used to improve the cement-aggregate bond and the freeze-thaw durability. The SBR latex is approved by the Federal Highway Administration for latex-modified concrete used in bridge deck overlays (Dow 2005).

An air entraining agent (AEA) and high-range water reducer (HRWR) were only used in the mixes that did not contain latex. The specific gravity and pH were 1.01 and 10 for air entraining (Everair plus) and 1.07 and 7.8 for the high-range water reducing admixtures (Glenium 3400 NV).

Table 2. Aggregate characteristics

Aggregate type	Crushed limestone	River gravel			River sand
Aggregate gradation	3/8 in.	No. 4	3/8"	1/2"	
Dry rodded unit weight (lbs/ft^3)	86.5	99.4	100.5	102.5	
Voids (%)	43.5	37.3	38.5	39.2	
Specific gravity	2.45	2.62	2.62	2.62	2.62
Absorption	3.20%	1.10%	1.10%	1.10%	1.10%

MIX PROPORTIONS

The mix design process was divided into two phases. Phase I was conducted to investigate how aggregate size and cement content influence the void ratio and strength of PCPC. These mix proportions are similar to those summarized in Table 1. After the desirable void ratio and strength were achieved, Phase II was conducted to study the effects of sand addition and silica fume and latex (as a replacement of binder) on the PCPC properties. All mix proportions studied are presented in Tables 3 and 4.

Table 3. Phase I mixture proportions

Mix	Aggregate type	Size	Unit weight (lbs/cy) PC	Aggregate	Water	Water/binder
1	River gravel	3/8"	600	2700	162	0.27
2	River gravel	1/2"	550	2700	149	0.27
15	River gravel	#4	578	2700	156	0.27
3	Limestone	3/8"	550	2700	149	0.27

Table 4. Phase II mixture proportions

			Unit weight (lbs/cy)						
Mix	Aggregate type	Size	PC	Silica fume	Latex	G	S	Water	Water/binder
12	River gravel	#4	571	-	-	2500	168	154	0.27
10	River gravel	#4	525	-	52.5	2700	-	116	0.27
8	River gravel	#4	520	-	52	2500	168	114	0.22
13	River gravel	#4	542.5	-	28.5	2500	168	130	0.24
14	River gravel	#4	485.4	-	85.6	2500	168	107	0.22
19	River gravel	3/8"	571	-	-	2500	168	154	0.27
5	River gravel	3/8"	522.5	27.5	-	2700	-	149	0.27
11	River gravel	3/8"	520	52	-	2500	168	114	0.27
4	Limestone	3/8"	522.5	27.5	-	2700	-	149	0.27
16	Limestone	3/8"	571	-	57.1	2500	168	126	0.22
17	Limestone	3/8"	600	-	60	2500	200	132	0.22

SPECIMEN PREPARATION

Two batch sizes (0.5 cf and 3 cf) were used during the mix design. Initially, mixes were prepared using a small trial batch (0.5 cf) to perform void ratio and seven-day strength tests. When the results appeared satisfactory, a larger batch (3 cf) was prepared and further investigation was conducted. To improve the bond between aggregate and cement paste, aggregate was first mixed with dry cement for one minute, and then half of the water containing either latex or AEA was added and mixed for one minute. Next, the remaining cement and water (with or without HRWR) was added. Finally, the concrete was mixed for three minutes, allowed to rest for three minutes, and then mixed for an additional two minutes before casting. Specimens were placed by rodding 25 times in three layers along with applying a vibration for five seconds after rodding each layer. The samples were demolded after 24 hours, placed in a fog room at 98% relative humidity, and cured according to ASTM C192. Before compression testing, the cylinders were capped using a sulfur capping compound following ASTM C617.

TESTING PROCEDURE

Workability of the fresh concrete was determined by a standard slump cone test (ASTM C143). Sulfur-capped samples were tested in compression according to ASTM C39. Cylinders of four x eight inches were used for both compression and tensile strength tests. Splitting tensile tests were performed based on ASTM C496.

The void ratio of pervious concrete was determined using a three- x six-inch cylinder sample by taking the difference in weight between a sample oven dried and a sample under water (Park 2004).

$$V_r = [1 - (\frac{W_2 - W_1}{\rho_w \mathrm{Vol}})]100(\%) \qquad (1)$$

Where V_r=total void ratio, %; W_1=weight under water, lbs; W_2=oven dry weight, lbs; Vol=volume of sample, in.^{3}; ρ_w=density of water, lbs/in.^{3}.

Permeability was determined using the falling head method through the permeameter illustrated in Figure 1. Samples used in the permeability test were three inches in diameter and three inches in length. A flexible sealing gum was used around the top perimeter of the sample to prevent water leakage along the sides of the sample. The samples were then confined in a membrane and sealed in a rubber sleeve, which was surrounded by adjustable hose clamps. The test was performed using several water heights that represented values that a pavement may experience. The initial height of water (h_1) ranged between two and six inches. The coefficient of permeability (k) was determined by the following equation:

$$k = \frac{aL}{At} LN\left(\frac{h_1}{h_2}\right) \qquad (2)$$

Where k=coefficient of permeability, in./s; a=cross sectional area of the standpipe, in.^{2}; L=length of sample, in.; A=cross sectional area of specimen, in.^{2}; t=time in seconds from h_1 to h_2; h_1=initial water level, in.; h_2=finial water level, in. (Das 1998).

Figure 1. Measurement of permeability for PCPC

Samples were subjected to freeze-thaw testing according to ASTM C666A, in which the samples are frozen and thawed in the wet condition (see Figure 2). The test was complete when a sample reached 300 cycles or 15% mass loss. Mass loss was tested every 20 to 30 cycles, depending on the stability of the sample. To date, samples have been subjected to 117 and 180 cycles.

Figure 2. PCPC freeze-thaw test samples

RESULTS

The results of Phase I (Table 5) indicate that the void ratio of baseline mixes (mixes 1, 2, 3, and 15) were all above 25%, and the seven-day compressive strength ranged from 1,145 psi to 2,100 psi. Mix 3 (3/8-in. crushed limestone) provided the highest void ratio but lowest compressive strength. Mix 15 (No. 4 river gravel) provided the highest strength and the lowest void ratio.

Table 5. Phase I results

Mix	Unit weight (lbs/ft^3)	Void ratio (%)	Seven-day compressive strength (psi)
1	116.9	28.8	1771
2	112.9	38.8	1145
15	117.5	25.3	2100
3	104.1	33.6	1396

Considering both the void ratio and strength, 3/8-in. and No. 4 gravel was selected for study in Phase II. The results of Phase II are summarized in Table 6. Full-scale batches of mixes 3 and 15 were prepared and tested. The 28-day compressive strength was 2,506 psi and 1,722 psi for mixes 15 and 3, and the permeability was 0.57 in./sec. and 0.098 in./sec., respectively. Adding silica fume to mixes 1 and 3 (i.e., mixes 5 and 4) did not show strength improvement.

Table 6. Phase II results

Mix	Unit weight (lbs/ft^3)	Void ratio (%)	Seven-day compressive strength (psi)	k (in./s)
15	117.5	25.3	2,100	0.098
3	104.1	33.6	1,396	0.57
5	111.6	33	1,347	-
4	98.6	41.8	784	-
19	130.9	20.5	3,262	0.193
11	127.3	20.2	2,641	0.094
12	126.3	19	3,299	0.043
10	135.2	12.9	3,142	-
8	126.8	19	2,969	0.071
13	120.3	26	1,307	-
14	132.2	14.1	2,735	0.016
16	117.4	25.7	2,483	0.185
17	124.6	17.9	2,910	0

The effect of using sand and latex in the mix was also investigated. Including 7% fine sand to baseline mixes 1 and 15 (see mixes 19 and 12) improved the strength by 57% and 84% and reduced the void ratio by 8.3% and 6.3%, respectively. Comparing the results for mixes 19 and 11 (i.e., 3/8-in. river gravel with sand and with sand and latex) showed that the strength and permeability decreased when latex was included in the mix, although both mixes had the same void ratio. As a result, the permeability was reduced from 0.098 in./sec to 0.043 in./sec. when comparing mixes 15 and 12. Adding 10% latex to baseline mix 15 (see mix 10) improved the strength by 50% but reduced the void ratio by 50%. When using both latex and sand (see mix 8), the increase in seven-day compressive strength was less than that achieved using only sand or latex, and the permeability reduced from 0.098 in./sec to 0.071 in./sec when comparing mixes 15 and 8. Adding sand to mix 15 (see mix 12) improved the tensile strength by 49%. Furthermore, using sand together with latex, the tensile strength improved by 58% (see mix 8).

The effect of the amount of latex added was further investigated using 5% and 15% in mixes similar to mix 8 (see mixes 13 and 14). The results show a decrease of strength in mixes 13 and 14 when compared to mix 8. This result indicates that using 10% latex is the optimum amount for improving strength, workability, and water reduction.

For crushed limestone aggregate mixes, using latex and sand improved the strength by 78% when comparing mix 16 with mix 3. When the cement content was increased to 600 lbs (i.e., mix 17), the strength increased by 108%. However, when comparing mixes 3, 16, and 17, the results indicate that the void ratio reduced from 33.6% to 25.7% and 17.9%, which reduced the permeability from 0.57 in./sec to 0.185 in./sec and 0 in./sec, respectively. Further observations and the trends of strength, permeability, and freeze-thaw for tested mixes are described below.

Void Ratio and Compressive Strength

As expected, the compressive strength of PCPC tended to decrease as the void ratio increased. For both river gravel and crushed limestone, the reduction of strength as a function of void ratio is linear (see Figures 3 and 4). When comparing the reduction in strength as the void ratio increases for 3/8-in. aggregate size (see equations on Figures 3 and 4), it is observed that although river gravel has higher initial compressive strength (at zero void ratio, 5,558 vs. 4,725 psi), the compressive strength diminished in a faster trend (128.9 vs. 96.6). For river gravel, where three sizes of gravel were tested, it was observed that as the aggregate size increases, the samples fail at the contact between the cement paste and the aggregate; however, the samples fail through aggregate for smaller size aggregate. This is expected, since the cement-aggregate contact area in mixes with large-size aggregate is smaller.

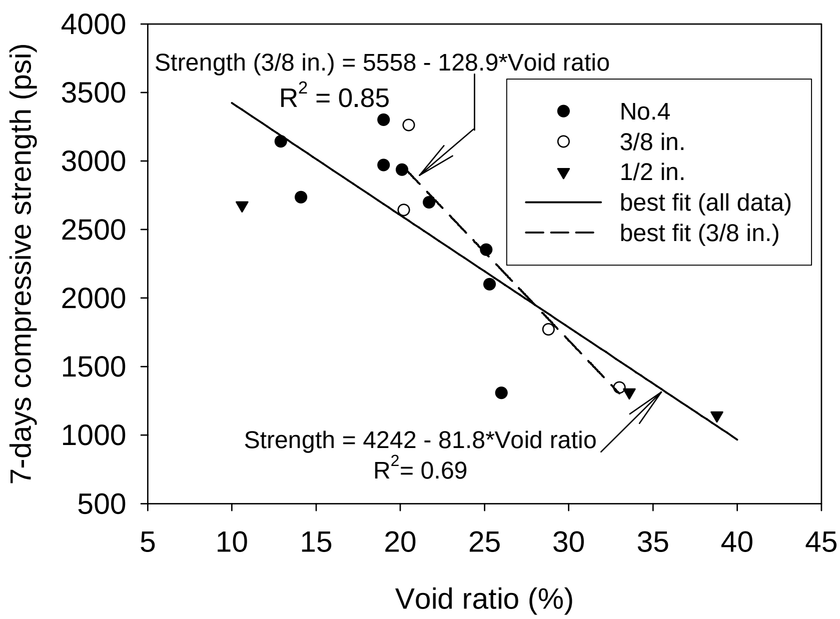

Figure 3. Seven-day strength as a function of void ratio for river gravel

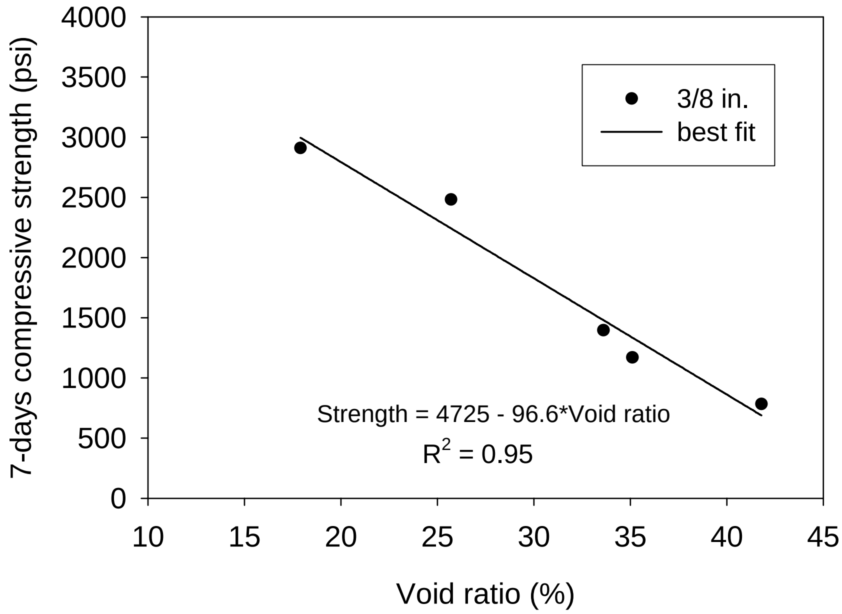

Figure 4. Seven-day strength as a function of void ratio for crushed limestone

Figure 5 shows the development of compressive strength for three mixes with No. 4 river gravel as a function of time. Mix 15 is the basic mix with only aggregate. This figure shows that mix 12 (with aggregate and sand) has the highest strength. Furthermore, it is observed that using latex hindered the improvement of strength between 21 and 28 days for mix 8 (i.e., No. 4 river gravel with sand and latex). This shows that the addition of some sand and/or latex improves the strength of the mix above the aggregate-only mix.

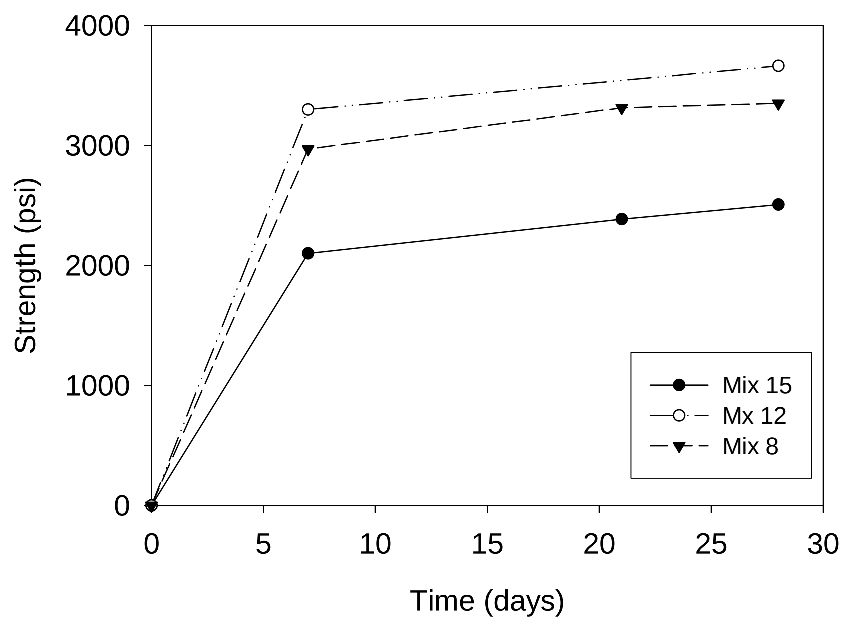

Figure 5. Strength increase as a function of time for No. 4 river gravel mixes

Void Ratio and Permeability

The permeability as a function of void ratio for all tested mixes is presented in Figure 6. This figure shows that permeability increases as the void ratio increases, with 20% void ratio being where the required strength of 3,000 psi was achieved (see Table 6).

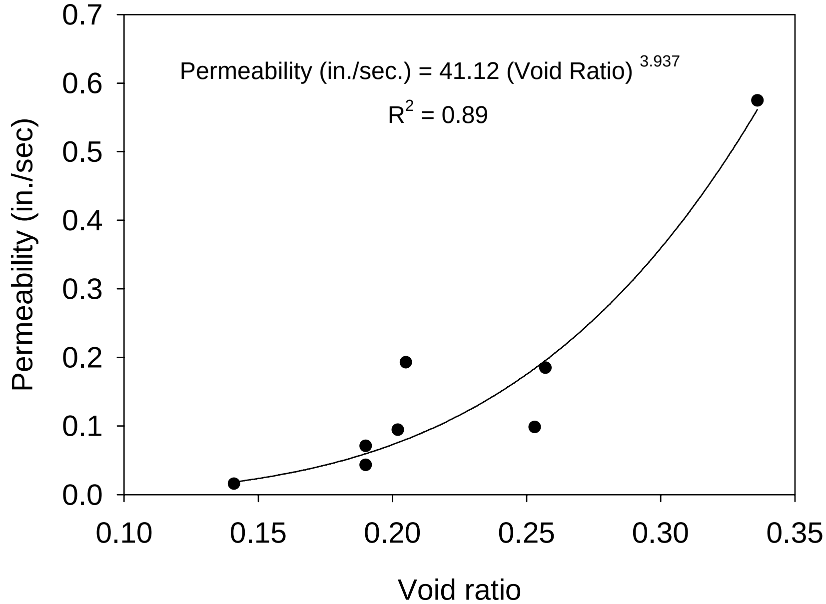

Figure 6. Change of strength and permeability as a function of void ratio

Freeze-Thaw Durability

At the time of the preparation of this paper, mixes 8 and 12 have been subjected to 180 freeze-thaw cycles, while mixes 3 and 15 have been subjected to 117 freeze-thaw cycles, with no sample failing the weight loss criteria of 15% mass loss (see Figure 7). Test results of mix 8 with sand and latex show less freeze-thaw resistance than mix 12, where no latex was used. Latex has some inherent air entraining ability, but the samples containing a standard AEA were showing better freeze-thaw resistance than the sample relying on latex for air entrainment. Future latex mixes will include additional AEA that is currently used in the non-latex mixes. The majority of the mass loss is through the splitting of the aggregate with the river gravel, beginning this process after only a few cycles. The lower strength crushed limestone is showing better freeze-thaw resistance than expected, with mix 3 retaining more mass than either mix 15 at 117 cycles. To date, all mixes tested using freeze-thaw showed less than 6% mass loss.

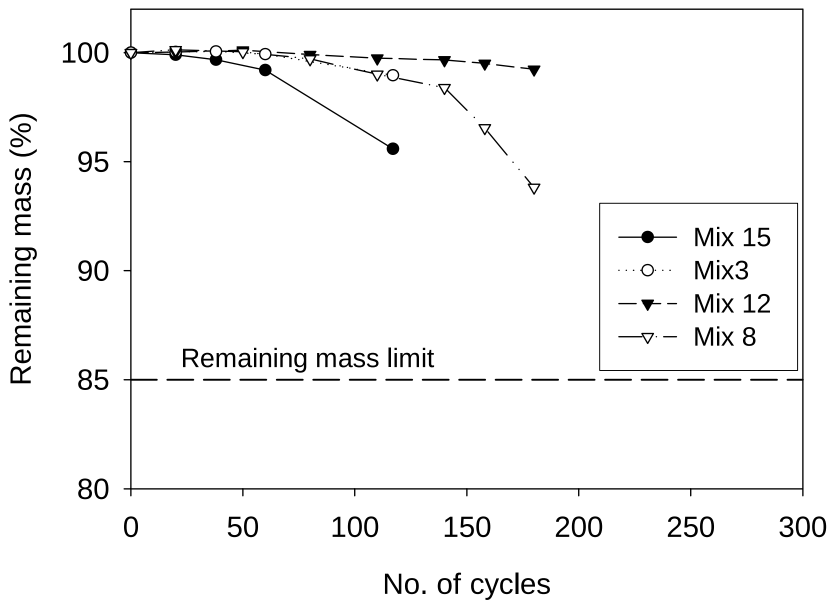

Figure 7. Freeze-thaw results to date

CONCLUSIONS

From this study the following conclusions can be made:

1. Mixes containing river gravel aggregate produced higher compressive strengths than those containing crushed limestone.
2. Although causing reductions in the void ratio, smaller aggregate provided concrete with higher strength.
3. The use of sand together with latex in a mix increased strength while decreasing void ratio and permeability.
4. The use of sand in a mix improved strength more than the use of sand together with latex.
5. Up to 180 cycles, mixes tested using freeze-thaw showed less than 6% mass loss.
6. Based upon the results to date, well-designed pervious concrete mixes can meet strength, permeability, and freeze-thaw resistance requirements for cold weather climates.

ACKNOWLEDGMENTS

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Concrete Pavement Design in Kansas Following the Mechanistic-Empirical Pavement Design Guide

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ABSTRACT

The AASHTO Guide for Design of Pavement Structures uses empirical performance equations for concrete pavement design. The new Mechanistic-Empirical Pavement Design Guide (MEPDG) provides methodologies for mechanistic-empirical pavement design. Five roadway sections, designed by the Kansas Department of Transportation (KDOT) using 1986 and 1993 AASHTO pavement design guides, and three long-term pavement performance (LTPP) sections in Kansas were analyzed using MEPDG. Project-specific construction and materials data and MEPDG default traffic data were used in the analysis. The predicted output variables, IRI and faulting, were compared with those obtained during KDOT annual pavement condition survey or from the LTPP database. The results show that the predicted IRI values are similar to the measured values. MEPDG analysis showed minimal or no faulting, although both predicted and measured faulting values were insignificant for all practical purposes. The sensitivity analysis results show that IRI is the most sensitive output with respect to the traffic inputs. Percentage of slabs cracked increases significantly with increasing truck traffic and decreases with increasing slab thickness. Faulting is the least sensitive parameter.

Key words: concrete pavement design—Mechanistic-Empirical Pavement Design Guide

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INTRODUCTION

The most widely used procedure for design of concrete pavements is specified in the *Guide for Design of Pavement Structures*, published in 1986 and 1993, by the American Association of State Highway and Transportation Officials (AASHTO 1986; AASHTO 1993). A few states use the 1972 American Association of State Highway Officials (AASHO) Interim Guide procedure, the Portland Cement Association (PCA) procedure, their own empirical or mechanistic-empirical procedures, or a design catalog (Hall 2003). The design methodologies in all versions of the AASHTO Guide are based on the empirical performance equations developed using the AASHO Road Test data from the late 1950's. The 1986 and 1993 guides contain some state-of-the-practice refinements in material input parameters and design procedures for rehabilitation design. Due to the limitations of earlier guides, a design guide, based as fully as possible on mechanistic principles, was developed under the National Cooperative Highway Research Program (NCHRP) (NCHRP 2004). The goal was to provide a user-friendly mechanistic-empirical design procedure that would account for local environment conditions, local materials, and actual highway traffic distribution by means of axle load spectra. Since the resulting procedure is very sound and flexible and it considerably surpasses the capabilities of any currently available pavement design and analysis tools, it is expected that it will be adopted by AASHTO as the new AASHTO design method for pavements structures.

BACKGROUND OF THE MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE

Basic Design Concept

Mechanistic-empirical (M-E) design combines the elements of mechanical modeling and performance observations in determining required pavement thickness for a given set of design inputs. The mechanical model is based on elementary physics and determines pavement response to wheel loads or environmental condition in terms of stress, strain, and displacement. The empirical part of the design uses the pavement response to predict the life of the pavement on the basis of actual field performance (Timm, Birgisson, and Newcomb 1998). Yoder and Witczak (1975) pointed out that for any pavement design procedure to be completely rational in nature, three elements must be fully considered: (1) the theory used to predict the assumed failure or distress parameter, (2) the evaluation of the materials properties applicable to the selected theory, and (3) the determination of the relationship between the magnitude of the parameter in question to the performance level desired. The newly developed M-E design guide considered all three elements. While the mechanistic approach to pavement design and analysis is much more rational than the empirical approach, it also is much more technically demanding. However, there are some specific advantages of M-E design over traditional empirical procedures: consideration of changing load types, better utilization and characterization of available materials, improved performance predictions, better definition of the role of construction by identifying parameters that have the most influence over pavement performance, relation of material properties to actual pavement performance, better definition of existing pavement layer properties, and accommodation of environmental and aging effects on materials. In essence, M-E design has the capability of changing and adapting to new developments in pavement design by relying primarily on the mechanics of materials. For example, M-E design can accurately examine the effect of new load configurations on a particular pavement. Empirical design, on the other hand, is limited to the observations on which the procedure was based (e.g., single axle load). Additionally, since the process is modular, new advances in pavement design may be incorporated without disrupting the overall procedure (Timm, Birgisson, and Newcomb 1998).

Design Approach

The design approach followed in MEPDG is summarized in Figure 1. The format provides a framework for future continuous improvement to keep up with the changes in truck traffic technology, materials, construction, design concepts, computers, and so on. As shown in the figure, in this guide, the designer first considers site conditions (traffic, climate, material, and existing pavement condition, in case of rehabilitation) and construction conditions in proposing a trial design for a new pavement or rehabilitation. The trial design is then evaluated for adequacy against some predetermined failure criteria. Key distresses and smoothness are predicted from the computed structural responses of stress, strain, and deflection due to given traffic and environmental loads, such as temperature gradient across the PCC slab. If the design does not meet desired performance criteria at a preselected level of reliability, it is revised and the evaluation process is repeated as necessary (NCHRP 2004). Thus, the designer is fully involved in the design process and has the flexibility to consider different design features, climatic conditions, and materials for the prevailing site condition. This approach makes it possible to optimize the design and to more fully insure that specific distress types will not develop.

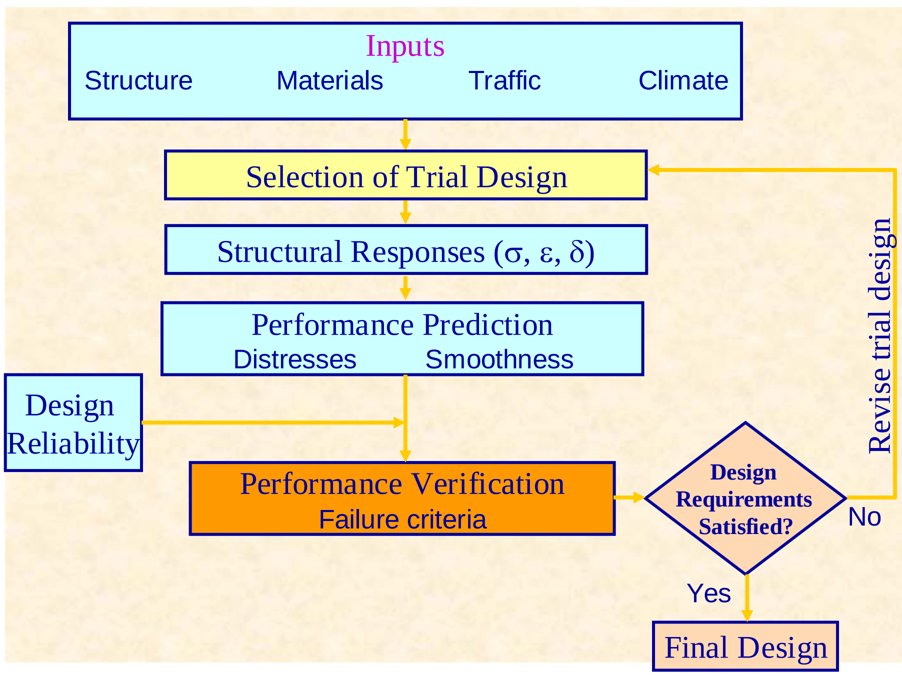

Figure 1. PCC mechanistic-empirical design framework (NCHRP 2004)

OBJECTIVE

The overall objective of this paper is to present the MEPDG design analysis results for selected in-service Portland cement concrete (PCC) projects in Kansas. Sensitivity analysis results for determining the sensitivity of the output variables due to variations in the key input parameters used in the design process are also presented.

TEST SECTIONS

Eight in-service PCC projects were selected for the MEPDG analysis. Three of these projects were the experimental sections chosen from the Kansas SPS-2 project located on Interstate route 70. Two other projects are located on Interstate Route 70, two on US-50, and one on route K-7. Table 1 tabulates the features of these sections. The SPS-2 sections were 500 ft long each, and the rest are one mile to several miles long.

All sections have 15 ft joint spacings with dowelled joints. Most have 1.5 in diameter steel dowels. Sections, where the PCC slab thickness is less than 10 inches, have 1.25 in diameter dowels. All sections have 12 ft lanes with tied concrete shoulders except the SPS-2 section 6. That section has a widened lane of 14 ft with tied PCC shoulder. The sections were constructed on stabilized base and on treated subgrade. Base stabilization was done with Portland cement. Depending upon the cement content and gradation, the bases were designated as Portland cement treated base (PCTB), bound drainable base (BDB), or lean concrete base (LCB). Base thickness ranged from 4 to 6 inches. The projects have primarily silty clay as subgrade. The top six inches of the natural subgrade were treated with lime or fly ash to reduce the plasticity and/or control the moisture during construction. PCC slab thickness, designed according to the 1986 or 1993 AASHTO design guide, ranged from 9 to 12 inches. The strength (modulus of rupture or compressive strength) values shown in Table 1 are the actual average values obtained during construction. The as-constructed International Roughness Index (IRI) values on these projects varied from 59 in/mile to 122 in/mile.

The annual average daily traffic (AADT) on these sections ranged from 2,080 for a US-50 Chase County project to 36,000 for the I-70 Shawnee County project. Very high percentage of truck traffic was observed on the US-50 projects, and the lowest percentage was on the I-70 Shawnee County project.

Table 1. Project features of the study sections

Project ID	Route	County	Year Built	Mile post Limit	PCC Thickness (in)	PCC 28-day Strength (psi)	Subgrade Soil Type	Initial AADT	% Truck	Initial IRI (in/mi)
K-2611-01*	I-70	Geary	1990	0 – 7	11	690	A-6	9,200	18	60
K-3344-01**	I-70	Shawnee	1993	9 – 10	10.5	473	A-7-6	36,000	5	96
SPS-2 (Sec-5)†	I-70	Dickinson	1992	20 – 22.61	11	945	A-6	11,970	22.3	122
SPS-2 (Sec-6)†	I-70	Dickinson	1992	20 –22.61	11	617	A-6	11,970	22.3	98
SPS-2 (Control)*	I-70	Dickinson	1992	20 – 22.61	12	647	A-6	11,970	22.3	95
K-3216-02***	US-50	Chase	1997	0 – 9	10	5,569	A-7-6	2,080	45.5	59
K-3217-02***	US-50	Chase	1997	9 – 19	10	4,362	A-7-6	3,480	40.5	68
K-3382-01**	K-7	Johnson	1995	12 – 15	9	537	A-7-6	13,825	7	81

*6” Portland Cement-Treated Base (PCTB)
** 4” Portland Cement-Treated Base (PCTB)
*** 4” Bound Drainable Base (BDB)
† 6” Lean Concrete Base (LCB)

MEPDG DESIGN INPUTS

Hierarchical Design Inputs

The hierarchical approach is used for the design inputs in MEPDG. This approach provides the designer with several levels of "design efficacy" that can be related to the class of highway under consideration or to the level of reliability of design desired. The hierarchical approach is primarily employed for traffic, materials, and environmental inputs (NCHRP 2004). In general, three levels of inputs are provided:

Level 1 **-** Level 1 is a "first class" or advanced design procedure. It provides for the highest practically achievable level of reliability and recommended for design in the heaviest traffic corridors or wherever there are dire safety or economic consequences of early failure. The design inputs also are of the highest practically achievable level and generally require site-specific data collection and/or testing. Example is the site-specific axle load spectra for traffic input.

Level 2 **-** Level 2 is the input level expected to be used in routine design. Level 2 inputs are typically user-selected, possibly from an agency database. The data can be derived from a less than optimum testing program or can be estimated empirically. Estimated Portland cement concrete elastic modulus from the compressive strength test results is an example of Level 2 input in the material input data category.

Level 3 **-** Level 3 typically is the lowest class of design and should be used where there are minimal consequences of early failure. Inputs typically are user-selected default values or typical averages for the region. An example would be the default value for the Portland cement concrete coefficient of thermal expansion for a given mix class and aggregates used by an agency.

For a given design, it is permissible to mix different levels of input.

MEPDG Design Features

Due to climatic variation and repeated traffic loads over the design life of a pavement, very high amount of uncertainty and variability exists in the pavement design and construction processes (NCHRP 2004). In the mechanistic-empirical design, the key outputs are the individual distress quantities. For jointed plain concrete pavements, MEPDG analysis predicts distresses, such as faulting, transverse cracking, and smoothness or IRI. Therefore, the reliability term has been incorporated in MEPDG to come up with an analytical solution, which allows the designer to design a pavement with an acceptable level of distress at the end of design life. Design reliability is defined as the probability that each of the key distress types and smoothness will be less than a critical level over the design period. Therefore, failure criteria are associated with this design reliability. The failure criteria and design reliability are also required inputs for the MEPDG analysis; although, the designer and the agency have the control over these values. The design can fail if the predicted distress is greater than the allowable amount or if the predicted distresses are unacceptable. In this study, the design reliability used for all projects was 90% and the corresponding failure criteria was 164 in/mile for IRI, 0.12 in for faulting, and 15% for slab cracking.

Inputs for Concrete Pavement Design Analysis

Input data used for the MEPDG analysis of concrete pavements are categorized as (a) general information, (b) traffic, (c) climate, (d) pavement structures, and (e) miscellaneous.

General Information

The general information inputs include design life, construction month, traffic opening month, pavement type (JPCP/CRCP), initial smoothness (IRI), etc. All pavement sections in this study were jointed plain concrete pavements (JPCP) and were analyzed for 20-year design life. Inputs, such as construction and traffic opening months, were required to compute the drying shrinkage and concrete strength gain.

Traffic

Traffic data is one of the key elements required for the MEPDG analysis. The basic required information is AADT for the year of construction, percentage of trucks in the design direction and on the design lane, operational speed, and traffic growth rate. The AADT in this study ranged from 2,080 to 36,000. The traffic growth in MEPDG can be linear or compound. Project-specific linear traffic growth rates varying from 2% to about 7% were used in this study. Truck percentages in the design direction varied from 47% (provided by LTPP for the SPS-2 sections) to 50%. For the 4-lane divided highways, 95% percent of trucks were assigned in the design lane based on the default level 3 input.

For this study, all other required traffic inputs, such as monthly and hourly truck distribution, truck class distribution, axle load distributions, and some other general traffic inputs, were derived from the design guide level 3 or default values.

Climate

Environmental conditions have significant effects on the performance of rigid pavements. The seasonal damage and distress accumulation algorithms in the MEPDG design methodology require hourly data for five weather parameters, such as air temperature, precipitation, wind speed, percentage sunshine, and relative humidity (NCHRP 2004). The design guide recommends that the weather inputs be obtained from weather stations located near the project site. At least 24 months of actual weather station data are required for computation. The design guide software includes a database of appropriate weather histories from nearly 800 weather stations throughout the United States. This database can be accessed by specifying the latitude, longitude, and elevation of the project location. The MEPDG software locates six weather stations closest to the site. In this study, project specific virtual weather stations were created by interpolation of climatic data from the selected physical weather stations. Specification of the weather inputs is identical at all three hierarchical input levels in MEPDG.

Pavement Structures

In this study, the baseline rigid pavement structure for design analysis is a four-layer JPCP consisting of a Portland cement concrete (PCC) slab over a stabilized base and a treated subgrade. The bottom most layer is the compacted natural subgrade. The inputs required for the PCC layer were layer thickness, material unit weight, Poisson’s ratio, coefficient of thermal expansion, cement type, cement content, water-cement ratio, aggregate type, modulus of rupture, modulus of elasticity, compressive strength, etc. PCC slab thickness in this study ranged from 9 to 12 inches.

All projects in this study have stabilized bases. The inputs required for these bases were layer thickness, mean modulus of elasticity, unit weight of the material, Poisson’s ratio, etc. Layer thickness ranged from 4 to 6 inches. The modulus of elasticity for the cement-treated bases was 500,000 psi. The lean concrete base modulus was taken as 2 million psi. All projects have 6-inch lime or fly-ash treated subgrade (LTSG/FASG) with an input modulus of 50,000 psi. Natural subgrade modulus was calculated by the

MEPDG software from a correlation equation involving the plasticity index and soil gradation. These properties were the inputs in this analysis process.

Miscellaneous

The thermo-hydraulic properties required as inputs into MEPDG are groundwater depth, infiltration and drainage properties (pavement cross slope (%) and drainage path length), physical/index properties (specific gravity of soil solids, maximum dry unit weight, etc.), hydraulic conductivity, thermal conductivity, heat capacity, etc. (Barry and Schwartz 2005). All projects in this study have 1.6% of pavement cross slope. Other parameters were derived from the design guide level 3 values or determined based on correlations. The recommended calibrated values of 1.25 BTU/hr-ft-°F and 0.28 BTU/lb-°F were used for thermal conductivity and heat capacity, respectively. Physical and index properties were derived based on the gradation of the unbound materials. Surface shortwave absorptivity and drainage path length were chosen based on the default inputs, and were 0.85 and 12 ft, respectively. County soil reports were used to estimate the ground water table depth for all sections. Project specific input parameters for the analysis are summarized in Table 2.

RESULTS AND DISCUSSIONS

Design Analysis

As mentioned earlier, key rigid pavement distresses predicted for JPCP from the MEPDG analysis are IRI, faulting, and percentage of slabs cracked.

Smoothness or IRI

In this study, MEPDG-predicted IRI values for the sections were compared with the KDOT-measured and LTPP DataPave (online database) values for 2003. The average IRI, obtained from the left and right wheel path measurements on the travel lane, was used in the comparison. The profile survey for the KDOT Pavement Management System (commonly known as Network Optimization System or NOS) was done on both eastbound and westbound directions for the I-70 Geary and I-70 Shawnee county projects and on northbound and southbound directions for the K-7 Johnson County project. For the SPS-2 sections, measured values were obtained from the LTPP database. Figure 2 shows the comparison between the predicted and measured IRI values for 2003 for all projects. The predicted IRI values for all Interstate sections are fairly similar to the measured values except for the I-70 Geary county project. SPS Section-5 has the highest predicted IRI of 130 in/mi and the I-70 Geary county project has the lowest. The predicted IRI for the Geary county project is 68 in/mi compared to the measured values of 103 in/mi and 102 in/mi for the eastbound and westbound direction, respectively. For the non-interstate sections, NOS-measured IRI values are much higher than the predicted values for both Chase county projects. The MEPDG predicted IRI of 88 in/mi for the K-7 Johnson county project is very similar to the measured values of 86 in/mi and 81 in/mi for the northbound and southbound directions, respectively.

Table 2. Input parameters for MEPDG rigid pavement design analysis

INPUT PARAMETERS	Input Value							
	(I-70 GE)	(I-70 SN)	SPS (Sec 5)	SPS (Sec 6)	SPS (Control)	(US-50-CS1)	(US-50-CS2)	(K-7 JO)
General Information								
Pavement construction Date	Oct, 93	Nov,90	July, 92	July, 92	July, 92	Dec, 97	Dec, 97	Sep, 95
Traffic								
Initial two-way AADTT	1656	1800	2670	2670	2670	946	1409	968
No. of lanes in design direction	2	2	2	2	2	1	1	2
Traffic growth factor (%)	1.2	3	3.5	3.5	3.5	2	2	6.7
Design lane width (ft)	12	12	12	14	12	12	12	12
PCC Layer								
PCC Layer thickness (in)	11	10.5	11	11	12	10	10	9
Material Unit Weight (pcf)	140	142	143	139.2	146	136.9	138.5	142
Cement Type	II	I	II	II	II	II	II	II
Cement Content (Lb/yd^3)	653.4	630	862	532	600	622.1	626.3	622.9
Poisson’s ratio	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
Co-eff of thermal. expansion (in./in./°F X 10^{-6})	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5
Water-cement ratio (w/c)	0.44	0.411	0.35	0.35	0.42	0.42	0.43	0.46
Base Material								
Base Type	PCTB	PCTB	LCB	LCB	PCTB	BDB	BDB	PCTB
Base Thickness (in)	6	4	6	6	6	4	4	4
Base material unit wt. (pcf)	135	135	135.4	135.4	135	135	135	135
Base Modulus (psi)	500,000	500,000	2,000,000	2,000,000	500,000	500,000	500,000	500,000
Treated Subgrade								
Subgrade type	N/A	LTSG	FASG	FASG	FASG	LTSG	LTSG	LTSG
Subgrade modulus (psi)	N/A	50,000	50,000	50,000	50,000	50,000	50,000	50,000
Unit weight (pcf)	N/A	125	125	125	125	125	125	125
Poisson’s ratio	N/A	0.20	0.15	0.15	0.15	0.20	0.20	0.20
Compacted Subgrade								
Subgrade soil type	A-6	A-7-6	A-6	A-6	A-6	A-7-6	A-7-6	A-7-6
Subgrade Modulus (psi) *	9,746	6,268	6,928	6,928	7,523	6,300	6,098	7,262
Plasticity index, PI	15.8	25.7	26	26	23	25.7	27.3	19.9
Percent passing # 200 sieve	71.8	93.3	78.1	78.1	76.9	92.5	91.9	94.3
% passing # 4 sieve	100	100	100	100	98	100	100	100
D_{60} (mm)	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001

* computed by MEPDG

Faulting

Figure 3 shows the comparison between the predicted faulting and the NOS- or LTPP-measured values. No faulting was observed for the SPS-2 Section-6 which has a widened lane of 14 ft. Good agreement was observed for the other SPS-2 section. On other projects, some discrepancies were observed between the predicted and measured faulting at few locations. However, both measured and predicted values in 2003 were negligible for all practical purposes. For example, the Shawnee county project was projected to show faulting of 0.009 inch in 2003. The discrepancies between the NOS-measured and MEPDG-predicted faulting at a few locations were partly due to the way faulting is interpreted in the NOS survey. During NOS reporting, measured faulting is coded as F1, F2, or F3 depending upon the severity of faulting. F1 describes the faulting of greater than 0.125 in but less than 0.25 in, and this is the only severity observed at few locations on some projects. Also, in NOS faulting is rated on a per mile basis and computed from the profile elevation data. No numerical value of faulting is reported by NOS. Thus, the MEPDG analysis showed minimal faulting and it was confirmed by actual observation.

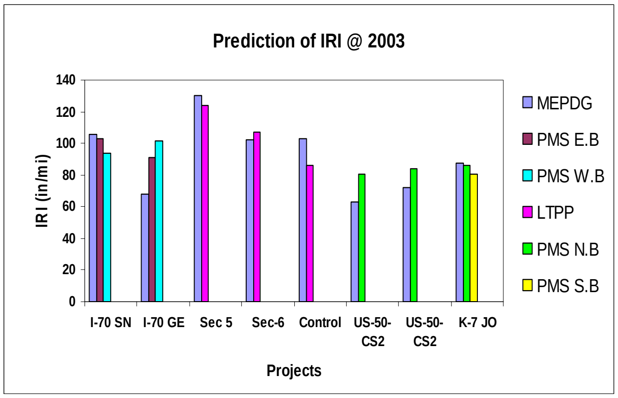

Figure 2. Predicted and measured IRI

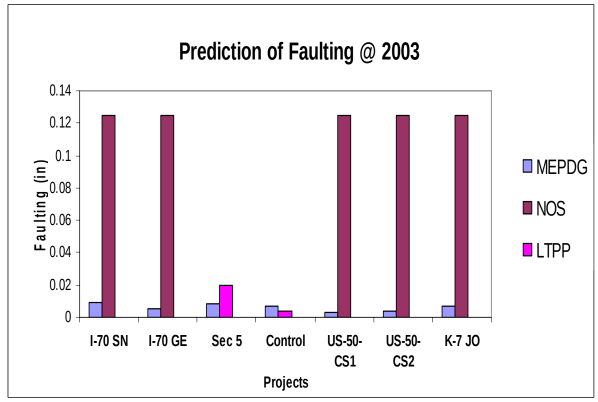

Figure 3. Predicted and measured faulting

Percentage of Slabs Cracked

One of the structural distresses considered for JPCP design in MEPDG is fatigue-related transverse cracking of the PCC slabs. Transverse cracking can initiate either at the top surface of the PCC slab and propagate downward (top-down cracking) or vice versa (bottom-up cracking), depending on the loading and environmental conditions at the project site, material properties, design features, and conditions during construction. The parameter indicates the percentage of total slabs that showed transverse cracking. Figure 4 shows the MEPDG-predicted percentage of slabs cracked values. With the project specific inputs, only Shawnee and Johnson county sections are showing some insignificant amounts of percentage of slabs cracked values of 1.3% and 0.6%, respectively. No measured values were available from the Kansas NOS condition survey report and LTPP database for comparison with the MEPDG predicted cracking. In NOS, no cracking survey is done on rigid pavements. In the LTPP survey, cracking is measured in terms of longitudinal and transverse crack lengths which can not be interpreted as percentage of slabs cracked. Of course, an average value can be computed. It is to be noted that none of the SPS-2 sections in this study showed any cracking up to 2003 in the MEPDG analysis.

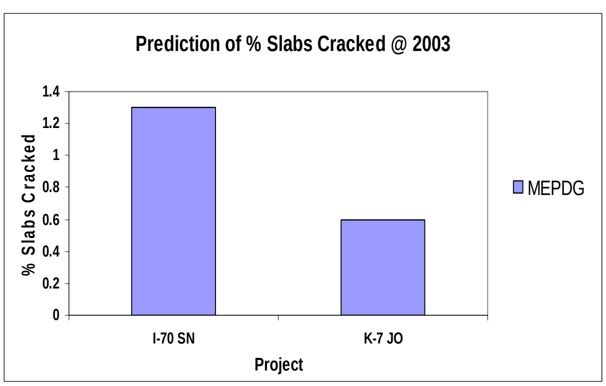

Figure 4. Predicted cracking

Sensitivity Analysis

In this part of the study, the sensitivity of the predicted performance parameters in the MEPDG analysis toward various input parameters has been determined for the I-70 Shawnee County project. The analysis was done for 20 years. The following input parameters were varied at the levels shown, and the predicted IRI, faulting, and percentage of slabs cracked were calculated. All other input parameters were project specific.

1. PCC Thickness (8 to 12 inch, at 0.5-inch increments)
2. AADT (2,080[Low], 12,526 [Medium], and 36,000 [High])
3. Truck (%) (5 [Low], 23.2 [Medium], and 47 [High])
4. Interaction of PCC Thickness and Truck Traffic (10.5 to 12 inch, 5% to 47%, respectively)

Thickness

PCC slab thickness is obviously the basic parameter in the design of rigid pavements. For the I-70 Shawnee county project, the design thickness was 10.5 inch. Thickness was varied from 8 to 12 inches at an increment of 0.5 inch. Figures 5, 6, and 7 show the variation of IRI, faulting, and cracking with thickness, respectively. Figure 5 shows that with increasing thickness, the predicted IRI after 20 years decreases. The predicted IRI decreased from 194 to 115 in/mi when the PCC slab thickness was increased from 8 to 12 inches. However, beyond 10 inches, the predicted IRI remained fairly the same irrespectively of increased thickness. Figure 6 shows that thicker pavement results in reduced faulting. Faulting decreased from 0.018 to 0.014 inches when the thickness was increased from 8 to 12 inches. However, this change in faulting is insignificant for all practical purposes. Figure 7 shows the change in percentage of slabs cracked with thickness. Cracking decreased from 90% to 0.3% when the thickness was changed from 8 to 12 inches. It appears that this parameter is extremely sensitive to thickness. Again, the results show that almost no benefit in terms of fatigue crack resistance can be obtained if the slab thickness is increased beyond 10 inches. The 1986 AASHTO design guide-derived thickness for this section was 10.5 inches.

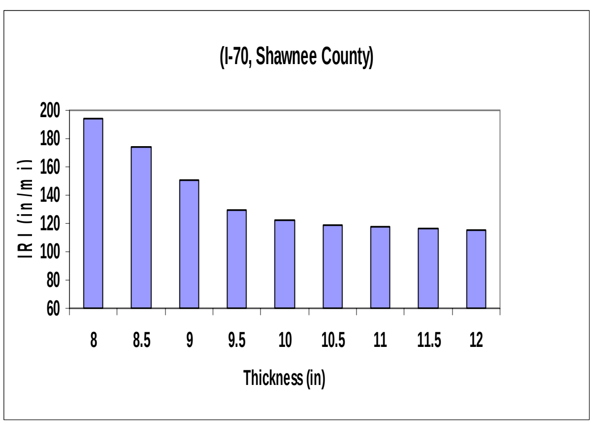

Figure 5. Change in IRI with thickness

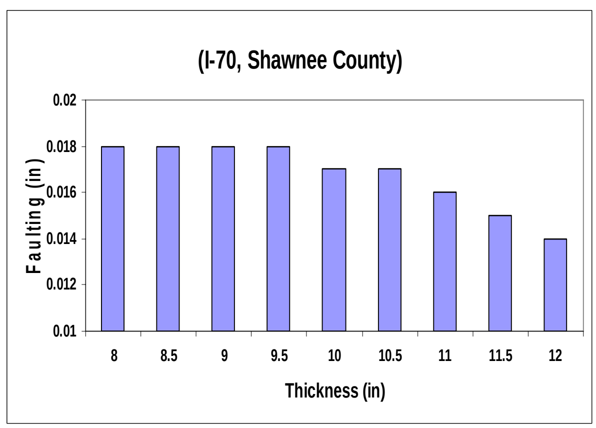

Figure 6. Change in faulting with thickness

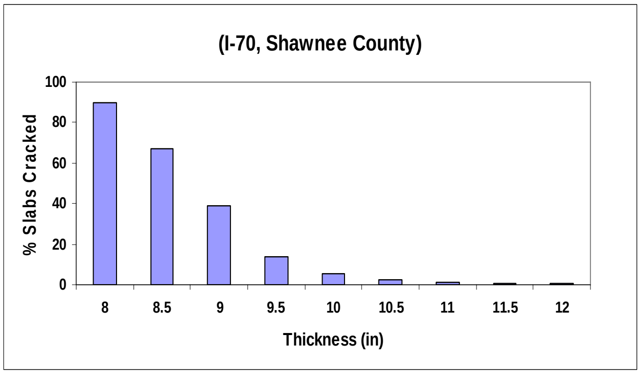

Figure 7. Change in cracking with thickness

AADT

The Annual Average Daily Traffic (AADT) was varied at three levels with a constant truck percentage (23.2%). Figures 8, 9, and 10 show that with increasing AADT, distresses increase significantly. For the three levels of AADT chosen, the predicted IRI ranged from 159 to 111 in/mi. Faulting and percentage of slabs cracked varied from 0.06 to 0.004 in and 23% to 0.2%, respectively.

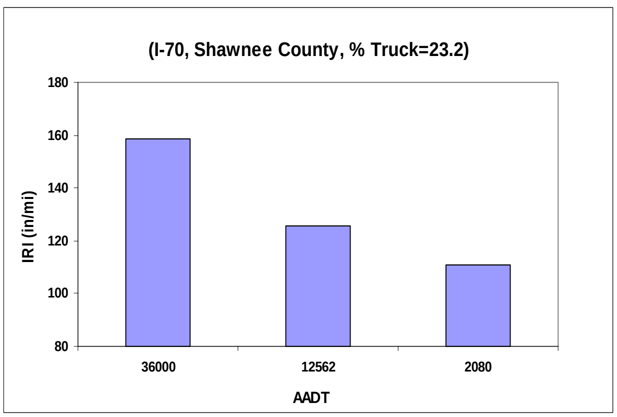

Figure 8. Change in predicted IRI with AADT

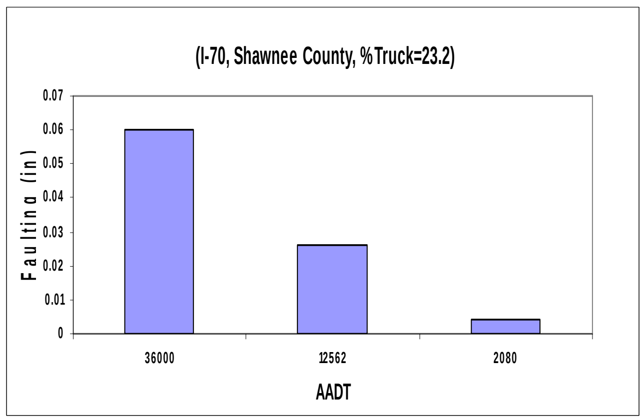

Figure 9. Change in faulting with AADT

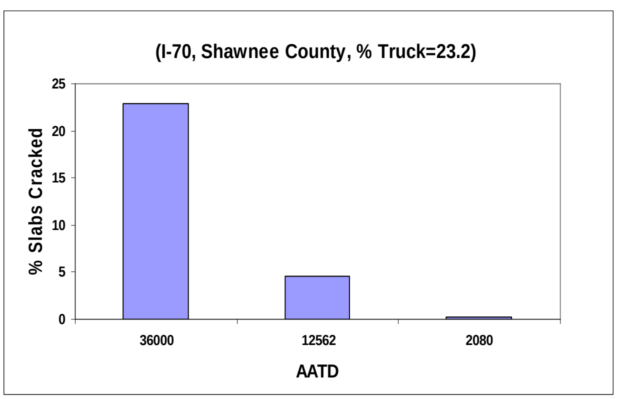

Figure 10. Change in cracking with AADT

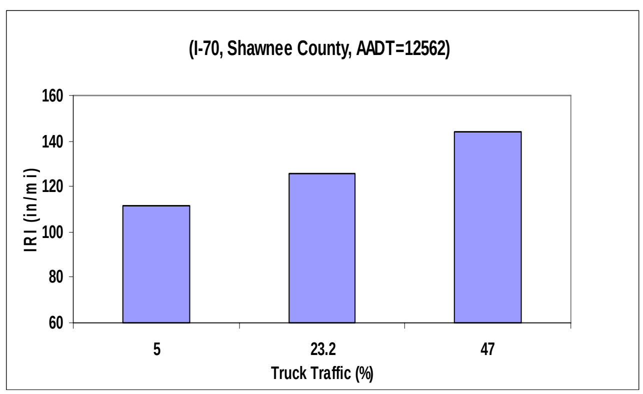

Figure 11. Change in IRI with % trucks

Truck Traffic

Figures 11, 12, and 13 show the variation of predicted distresses with different truck percentage at a constant AADT of 12,562, which is the average AADT level in this study. Figure 11 shows that with increasing percentage of trucks, IRI also increases significantly and ranges from 112 to 145 in/mi. Trend is similar for faulting and percentage of slabs cracked, as shown in Figures 12 and 13, respectively. Faulting varied from 0.006 to 0.047 in, whereas percentage of slabs cracked increased from 0.4% to 14%.

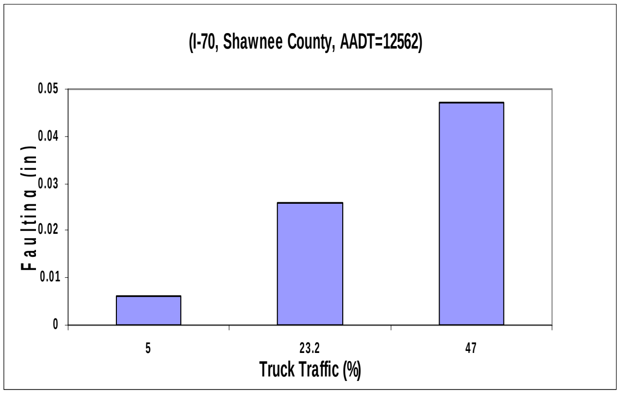

Figure 12. Change in faulting with truck traffic

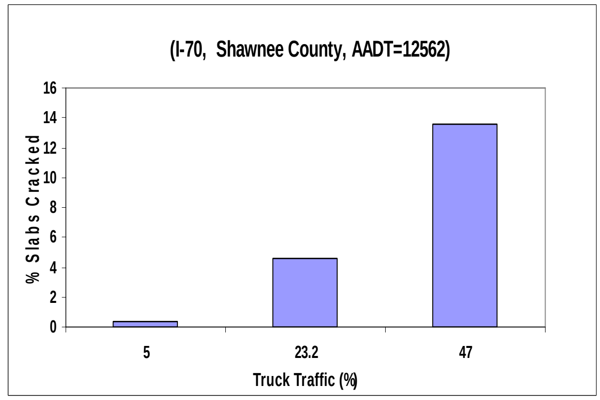

Figure 13. Change in cracking with truck traffic

Interaction of PCC Thickness and Truck Traffic

Truck traffic and PCC slab thickness were varied at three (5%, 20%, and 40%) and four (10.5 to 12 inch at an increment of 0.5 inch) levels, respectively. Thickness variation was done at each level of truck traffic. Figure 14 shows that if the thickness is changed for a lower truck percentage, variation in IRI is not significant compared to the higher percentages of truck traffic. It can be seen that for higher truck percentage, IRI decreases quite significantly with increasing PCC slab thickness. For 40% truck traffic, IRI decreased from 190 to 154 in/mi when the thickness was increased from 10.5 to 12 inches. Figure 15 shows that faulting is not very sensitive to the changes in truck traffic for the given thickness range. However, percentage of slabs cracked is quite sensitive, as shown in Figure 16. For 40% trucks, cracking decreased from 45% to 8% when the thickness was increased from 10.5 to 12 inches.

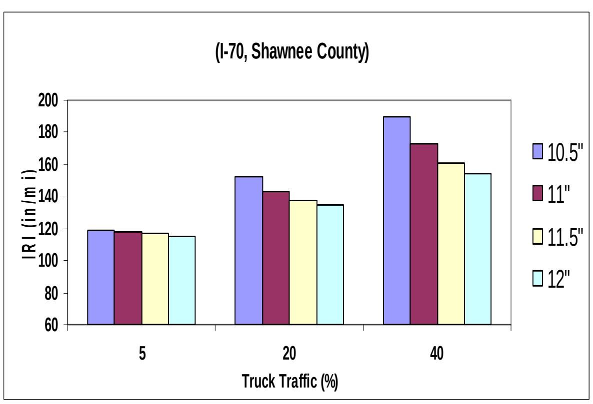

Figure 14. Change in IRI for % truck & thickness

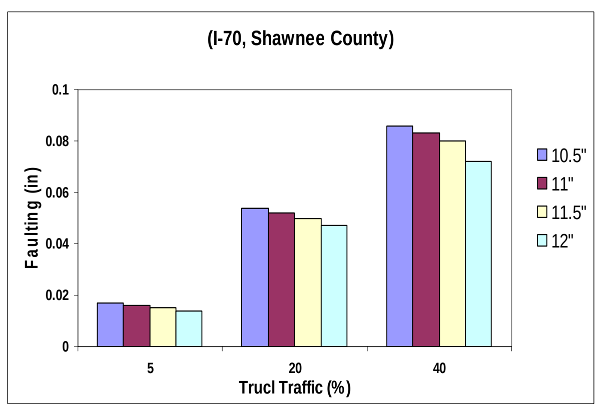

Figure 15. Change in faulting for % truck & thickness

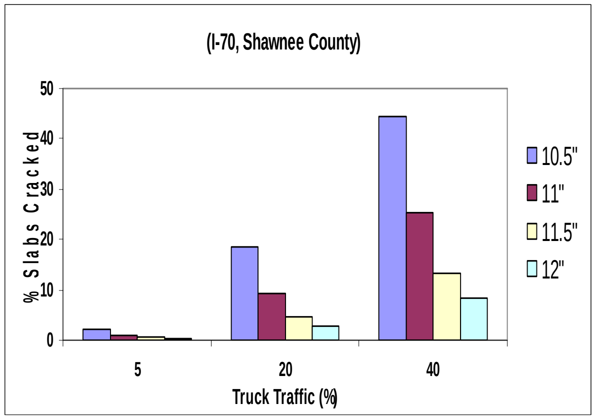

Figure 16. Change in cracking with % truck & thickness

CONCLUSION

This study presents the results of design analysis following the new mechanistic-empirical pavement design guide (MEPDG) for eight in-service concrete pavements in Kansas. The predicted distresses were compared with the measured values. Sensitivity analysis was also done for a project. Based on the study, the following conclusions can be made:

- For most projects in this study, the predicted IRI was similar to the measured values. MEPDG analysis showed minimal or no faulting, and it was confirmed by visual observation.
- IRI is the most sensitive output with respect to traffic.
- Percentage of slabs cracked increases significantly with increasing truck traffic and decreases significantly with increasing slab thickness. Faulting is the least sensitive parameter.

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Performance of SPS-2 Project in Kansas

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ABSTRACT

The Long-Term Pavement Performance (LTPP) SPS-2 experiment was designed to study structural factors, such as drainage, base type, concrete strength and thickness, and lane width, of rigid pavements. The SPS-2 experiment section in Kansas, constructed in 1992, is a jointed dowelled plain concrete pavement. The experiment consisted of 12 standard SPS-2 sections and 1 Kansas DOT control section. These sections have been monitored by the LTPP program since construction. Performance monitoring included measurements for ride quality (International Roughness Index [IRI]), faulting, cracking, and surface deflections. Performance parameters analyzed in this study included IRI, faulting, cracking (combined longitudinal and transverse crack lengths), and joint load transfer efficiency. The results show that the project has performed very well to date. Most sections are smooth and crack-free, with negligible faulting. The load transfer efficiency of the sections has been good too. The drainable sections with a permeable asphalt-treated base have performed the best. These sections were built smoother and remained so after 12 years of service. The section with low PCC slab thickness (8 inches) and low concrete design strength (550 psi) on a dense graded aggregate base has performed the worst. The combination of high slab thickness and high concrete strength tends to mask the effect of the base on pavement performance. The Kansas DOT control section with a thick slab (12 inches) over a dense graded portland cement-treated base has also performed very well.

Key words: concrete pavement—Long-Term Pavement Performance—SPS-2

INTRODUCTION

The Strategic Highway Research Program (SHRP) was initiated in 1987 to improve the performance of highway pavements and bridges. A part of this national research effort, the Long-Term Pavement Performance (LTPP) program was aimed at determining the effects of various designs and environmental and material performance by monitoring a large number of in-service pavement sections. The LTPP national database was established and is being updated periodically with the performance monitoring information collected from the selected pavement sections. The LTPP program was divided into two programs: General Pavement Studies (GPS), representing pavements that use materials and structural design practices in the United States and Canada from the 1960s to the 1980s, and Specific Pavement Studies (SPS), representing performance and specific structural factors of interest in the 1990s. The SPS-2 experiment entitled "Strategic Study of Structural Factors for Rigid Pavements" was developed to study the structural factors affecting the performance of rigid pavements (SHRP 1990). The experimental design was aimed at determining the effects of the following specific pavement design features: (1) in-pavement drainage system, (2) base type, (3) concrete strength, (4) pavement thickness, and (5) lane width. This paper analyzes the performance of the SPS-2 project in Kansas with respect to drainage and base type. Comparison with a control section designed by the Kansas DOT (KDOT) was also made.

KANSAS SPS-2 PROJECT LAYOUT AND TEST SECTIONS

The Kansas SPS-2 project, constructed in 1992, is located in the westbound driving lane of Interstate 70 near Abilene. The project reconstructed an existing concrete divided interstate highway (Johnson 1993). The roadway consists of two 12-ft lanes, a 10-ft tied outer shoulder, and a 4-ft inner shoulder in the westbound direction. The SPS-2 experiment includes 12 standard sections and 1 KDOT control section, shown in Table 1. Each section is 500 ft long, with transitions of varying length between the sections.

Table 1. Kansas SPS-1 test section details

Test section #	SHRP ID	Base*	Slab thickness (design) (in.)	Concrete strength (design) (psi)	Concrete strength (actual) (psi)	Lane width (ft)
KDOT control	State supple-mental	6" PCTB 6"Mod fly ash	12	600	617	12
KDOT 1	200209	4" PATB 4" DGAB	8	550	624	12
KDOT 2	200210	4" PATB 4" DGAB	8	900	924	14
KDOT 3	200211	4" PATB 4" DGAB	11	550	576	14
KDOT 4	200212	4" PATB 4" DGAB	11	900	865	12
KDOT 5	200208	6" LCB	11	900	855	12
KDOT 6	200207	6" LCB	11	550	584	14
KDOT 7	200205	6" LCB	8	550	702	12
KDOT 8	200206	6" LCB	8	900	829	14
KDOT 9	200202	6" DGAB	8	900	803	14
KDOT 10	200201	6" DGAB	8	550	606	12
KDOT 11	200204	6" DGAB	11	900	784	12
KDOT 12	200203	6" DGAB	11	550	595	14

* PCTB: dense graded portland cement-treated base; PATB: permeable asphalt-treated base; LCB: lean concrete base; DGAB: dense graded aggregate base

The sections incorporated two thicknesses (8 and 11 in.), two strength classes (design modulus of rupture of 550 and 900 psi at 14 days), three base types (permeable asphalt-treated base [PATB], lean concrete base [LCB], dense graded aggregate base [DGAB]), and two lane widths (12 ft and 14 ft). The KDOT control section consists of a 12-in. concrete slab over a 6-inch dense graded portland cement-treated base (PCTB). The design modulus of rupture for this section was 600 psi. Subgrade soils were silty clay. Since this project involved reconstruction of an existing concrete highway, the top subgrade layer was reworked and recompacted after incorporating the existing granular subbase and shoulder material. Construction was difficult due to the wet climate (Johnson 1993). Thus, a Class C fly ash was added to dry and stabilize the wet subgrade. As a result, all sections have a six-inch Class C fly ash-modified subgrade. The project was initially classified to be in the dry-freeze zone of the LTPP program. Later it was reassigned to the wet-freeze zone, since the annual precipitation at this site (32 inches/year) is greater than the 20 inches/year for the LTPP dry-freeze zone. The freezing index was 259 ^{o}C-days. The base year annual average daily traffic on this section was 13,750, with 21.4% trucks. The estimated 18-kip equivalent single axle load (ESAL) in the SPS-2 lane was 1,300,678, with a total of 26,013,550 18-kip ESALs over the 20-year design period.

Layout and Construction

The test sections were laid out starting at the west end of the project and according to the base type, as shown in Figure 1. Lane width and surface thickness were not considered. The KDOT control section was first, followed by those with PATB over DGAB (KDOT sections 1, 2, 3, and 4). The sections with LCB were placed next (KDOT sections 5, 6, 7, and 8). The last sections were those containing DGAB (KDOT sections 9, 10, 11, and 12). The construction of the test sections started in June 1992 and was completed in July 1992. The project was opened to traffic in early August of 1992 (Johnson 1993).

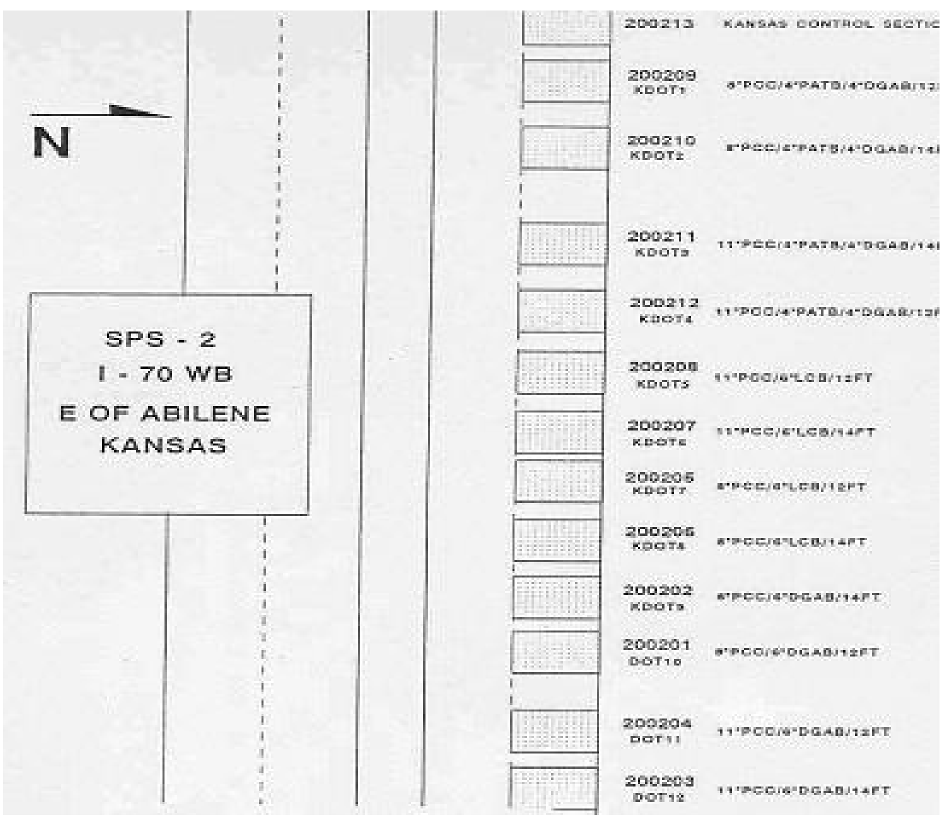


Figure 1. Layout of SPS-2 sections

Table 2 shows the mix design details. Type II cement was used in all mixes. The cement content varied from 279 lbs per cubic yard for LCB to 862 lbs for the 900-psi mixture. All mixtures, including the lean concrete base and paving concrete, used 70% natural sand and 30% crushed stone. Crushed limestone was used in the LCB and in the 550-psi concrete mix. The 900-psi mix had calcite-cemented sandstone for the coarse aggregate part to help increase the flexural strength. The design water-cement ratios for the LCB, 550-psi mix, and 900-psi mix were 1.0, 0.5, and 0.35, respectively (Johnson 1993). The mixes were air-entrained, and a water-reducing admixture was used in the 900-psi mix.

Table 2. SPS-2 mix design data

Feature	Lean concrete base	550-psi concrete	900-psi concrete
Strength requirement	500–750 psi comp. (7-day cure)	550 ± 60 psi flex. (14-day cure)	900 ± 100 psi, flex. (14-day cure)
% Air/air ent. admix	4–9%	2.6 oz.	7.7 oz.
Slump required	1”–3”	1¾” ± ¾”	1¾” ± ¾”
Cement type	II	II	II
Cement content (lbs/yd^3)	279	532	862
Water content (lbs/yd^3)	280	266	301
Coarse agg. content (lbs/yd^3)	929	891	1349
Fine agg. content (lbs/yd^3)	2169	2071	1347
Unit weight (lbs/ft^3)	135.4	139.21	143

The construction of the project was uneventful except for the PATB layer, which was placed too thick but then trimmed to the required thickness (Johnson 1993). Compaction of the PATB was problematic too. Post-construction coring and flexural beam tests showed that most sections had concrete that exceeded the design strength, as shown in Table 1. The strength gain over the design strength was most significant for the 550-psi mixture.

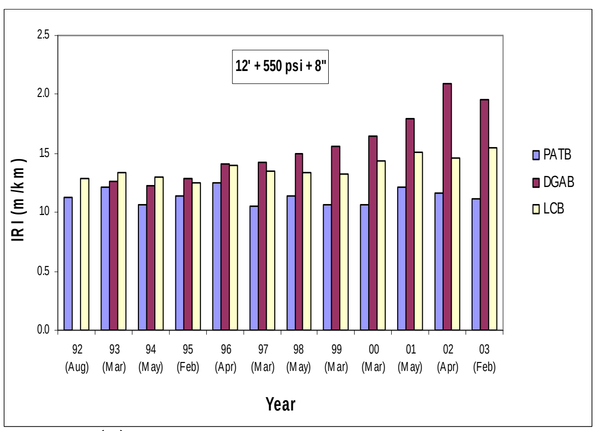
PERFORMANCE MONITORING, DATA ANALYSIS, AND RESULTS

The Kansas SPS-2 project has been monitored by the LTPP program since its construction. A Toledo Model 930 high-speed weigh-in-motion system was installed to provide automatic high-speed weighing, classifying, and traffic data collecting. The test sections have carried 6.6 million ESALs from 1992 to 2000. The truck traffic has been lower than that anticipated during the design process. Performance monitoring of these sections included annual measurements for ride quality (International Roughness Index [IRI]), faulting measurements since 1999, cracking measurements since 1993, and falling weight deflectometer (FWD) testing since 1993. All performance data used in this study were retrieved online from Datapave, release 18, of July 2004. This version of Datapave contains the following data elements for key performance measures: (a) IRI, every year between 1993 and 2003; (b) faulting for 1999, 2001–2003; (c) cracking for 1993, 1996, and 1999–2003; and (d) FWD deflection data for every other year between 1993 and 2003. From the wheel path FWD deflection data, joint load transfer efficiency (LTE) values were computed for all sections. Thus, performance parameters analyzed in this study included IRI, faulting, cracking (combined longitudinal and transverse crack widths), and joint LTE corresponding to the design features incorporated in the experiment.

Drainage

As mentioned, different base types were constructed to compare drained and undrained sections. The drained sections in this study are KDOT 1 (8" PCC/550-psi/12'), KDOT 2 (8" PCC/900-psi/14'), KDOT 3 (11" PCC /550-psi/14'), KDOT 4 (11" PCC/900-psi/12'). All drained sections were on 4-in. PATB and the PATB layer was over 4-in. DGAB. The undrained sections in the experiment include KDOT 5 (11" PCC /6" LCB/900-psi/12'), KDOT 6 (11" PCC/6" LCB/550-psi/14'), KDOT 7 (8" PCC/6" LCB/550-psi/12'), KDOT 8 (8" PCC/6" LCB/900-psi/14'), KDOT 9 (8" PCC/6" DGAB/900-psi/14'), KDOT 10 (8" PCC/6" DGAB/550-psi/12'), KDOT 11 (11" PCC/6" DGAB/900-psi/12'), KDOT 12 (11" PCC/6" DGAB/550-psi/14') and KDOT control (12" PCC/6" PCTB/600-psi/12').

Figure 2 (a) shows the IRI history of the sections with 12-ft lane width, 550-psi design strength, and 8-in. PCC slab thickness. The figure compares the ride quality of KDOT 1 (PATB), KDOT 10 (DGAB), and KDOT 7 (LCB). It is clear that the drained PATB section (KDOT 1) remained smoother than the undrained sections for the past 12 years. The IRI of this section in 2003 was only 70 inches/mi. This section was also the smoothest after construction. The DGAB section (KDOT 10) is progressively becoming rougher than the LCB section (KDOT 7) with common design features mostly due to cracking. The section had extensive cracking in 2000 (75 ft of longitudinal and transverse cracking) and was repaired. For the 12-ft lane width, thicker PCC slabs (11 inches), and higher slab strength (900-psi), the section with DGAB (KDOT 11) was the smoothest, PATB was next, and LCB was the roughest, as shown in Figure 2 (b). LCB performs the worst among all base types. Faulting on all sections until 2003 was minimal.



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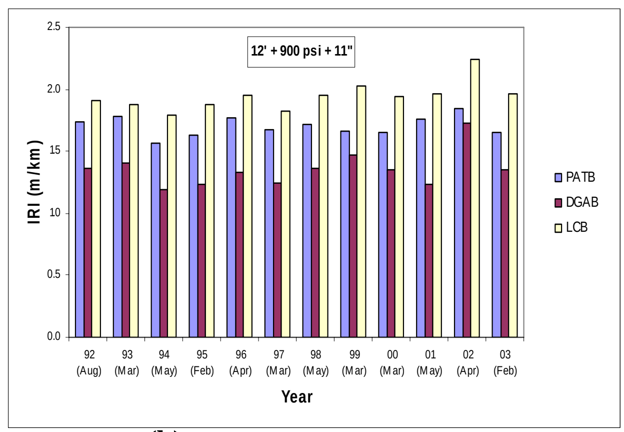

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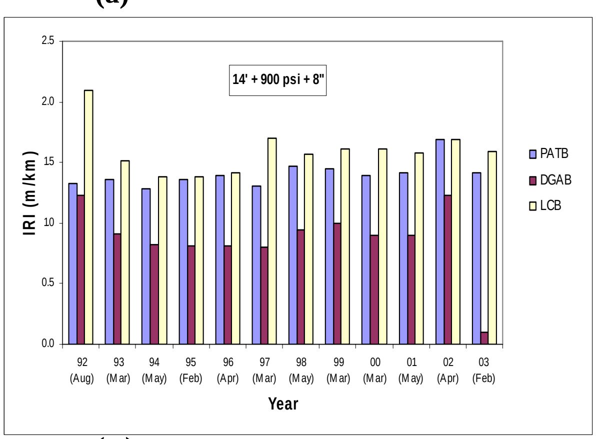

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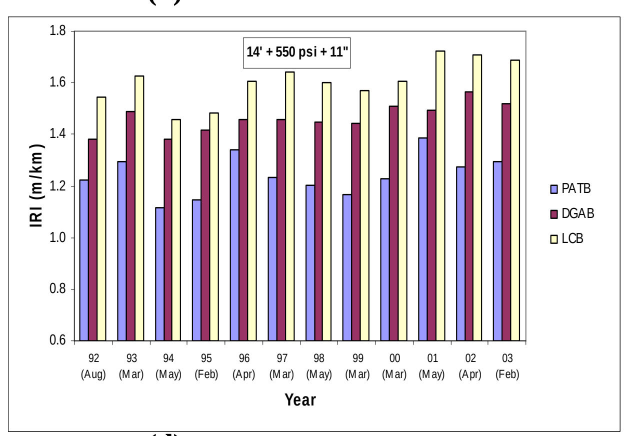


(d)

Figure 2. Change of IRI with time for the drained and undrained sections

Figure 2 (c) shows the IRI progression on the sections with a 14-ft lane width, 900-psi design strength, and 8-in. PCC slab thickness. The sections are KDOT 2 (PATB), KDOT 9 (DGAB), and KDOT 8 (LCB). The section with the DGAB base (KDOT 9) showed the lowest increase in roughness, whereas the section with LCB showed the largest. However, KDOT 9 (DGAB) had some longitudinal and transverse cracking. No cracking was observed on the other sections until 2003. The increased roughness with LCB is also evident for the section with a 14-ft lane width, 550-psi design strength, and 11-in. PCC slab thickness, as shown in Figure 2 (d). For this design feature, the section with PATB (KDOT 3) was initially the smoothest after construction, and remained so over time. For these design features, the DGAB section, but not the others, showed some cracking. Faulting on all sections was also minimal until 2003.

Figure 3 shows the typical LTE vs. time relationships for the sections with 12-ft lane widths, 550-psi design strengths, and 8-in. PCC slab thicknesses. The LTE values appear to be good for all sections. As expected, the sections with stabilized bases showed higher LTE values than the sections with DGAB. The trends are similar for other SPS-2 sections with different design features. Some fluctuations were observed in the LTE values over the years because of the time/temperature variations during FWD testing. For example, during the 1997 testing, some sections showed low LTE values. However, in 2003 these sections had LTE values greater than 80%.

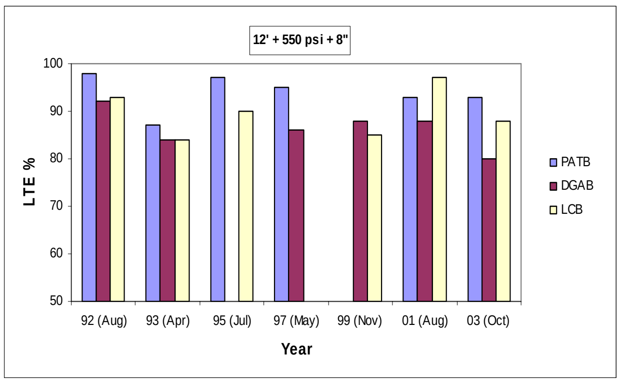


Figure 3. Change of LTE values with time

Base Type

PATB vs. LCB

Figures 4(a) through 4(d) compare IRI values for the sections with stabilized bases (PATB and LCB). The figures show that, for most design feature combinations, the PATB sections outperformed the LCB sections. The PATB sections were also built smoother initially. For the widened lane (14-ft) and high-strength sections, the IRI values are somewhat comparable, but for all other design features, the PATB sections are much smoother. It is noted that LTE values for some years are missing on some sections because no FWD tests at the joints were done on those sections during those years. On a few sections, LTE values were computed to be more than 100%, indicating locked joints, and were ignored during comparison.

Faulting is negligible on all sections. The PATB sections for the 12-ft lane width, 550-psi design strength, and 8-in. thickness and the 12-ft lane width, 900-psi design strength, and 11-in. thickness showed a very small amount of cracking (1.65 ft in 2003). For the 12-ft lane width, 550-psi design strength, and 8-in. thickness and the 14-ft lane width, 900-psi design strength, and 8-in. thickness, the PATB sections also showed higher LTE values with time than the LCB sections, in Figures 5(a) and 5(b), respectively. For other design features, both sections show comparable LTE values during recent years (Figures 5[c] and

5[d]). Based on the time-series trends of the performance measures in this study, the PATB sections outperformed those with LCB.

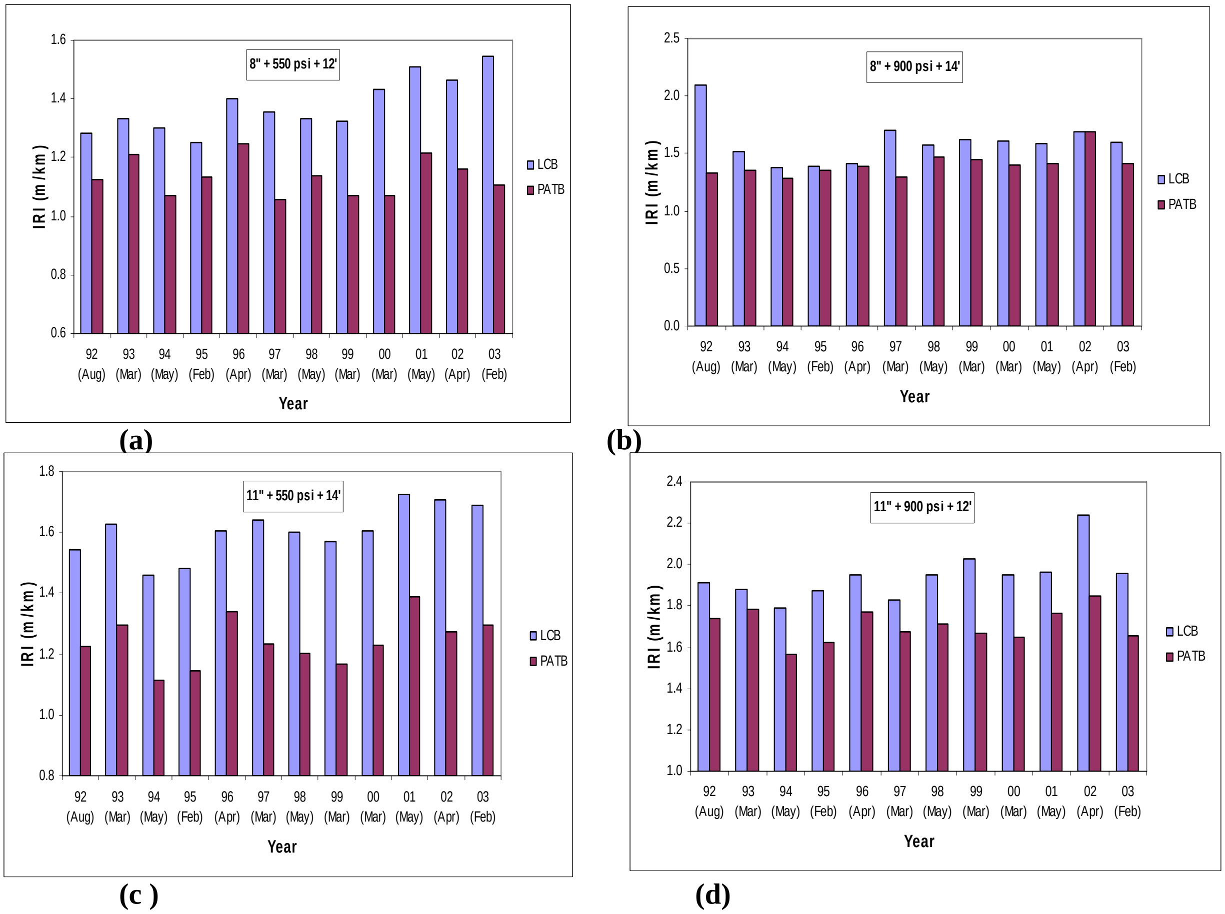
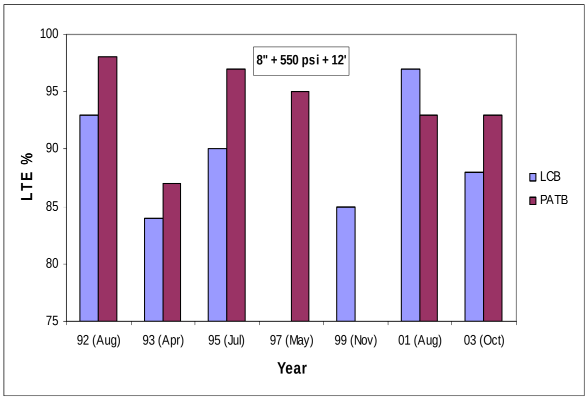
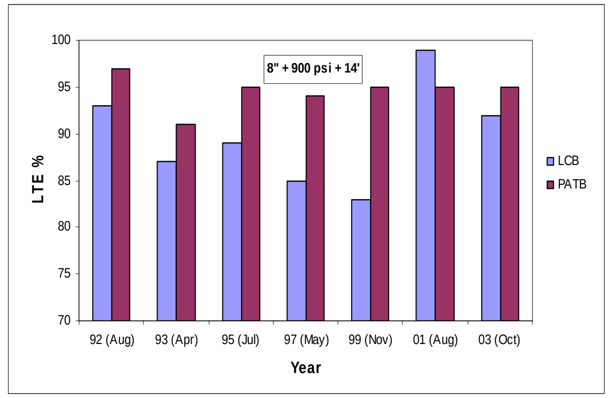


Figure 4. Change in IRI with time for the sections with stabilized base

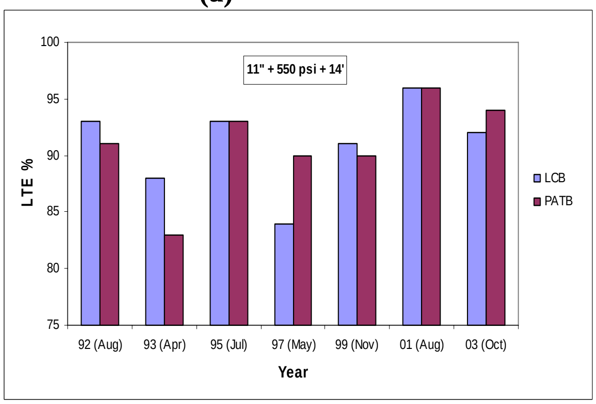
LCB vs. DGAB

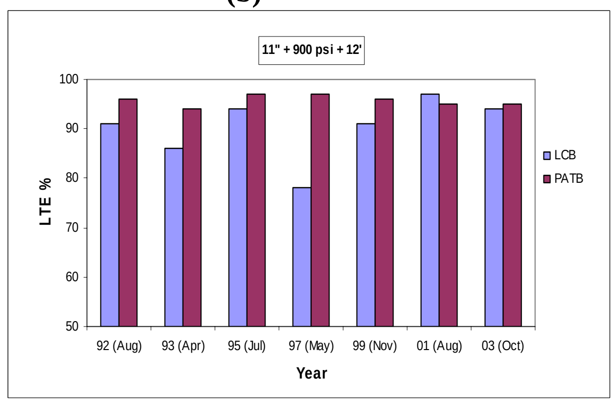
Figure 6 shows that for 12-ft lane width, 550-psi design strength, and 8-in. thickness, the DGAB section has become rougher more quickly than the LCB section. As mentioned, this DGAB section had a considerable amount of cracking and was repaired recently. For all other design features, the LCB sections have higher IRI values than the DGAB sections. The differences have recently become more prominent. Again, faulting has been very low on all sections, but since 2001, the section with DGAB and a widened lane (14 ft) has developed some negligible cracking. No cracking has been observed on the DGAB section with high strength and high thickness. It appears that the effect of the base layer is masked by higher PCC slab strength and thickness. The LTE values on the DGAB sections have been lower than those with LCB over the years (Figures 7(a) through 7(d)). However, the LTE values are generally good.





(a) **(b)**





(c) **(d)**

Figure 5. Change of LTE with time for sections with a stabilized base

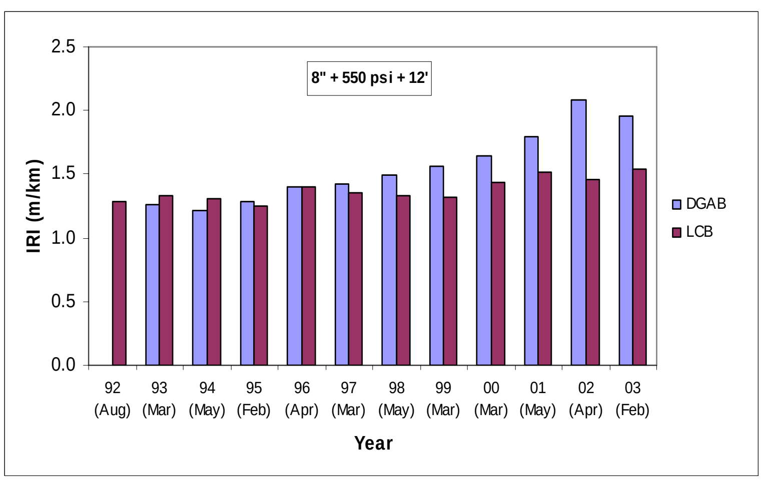
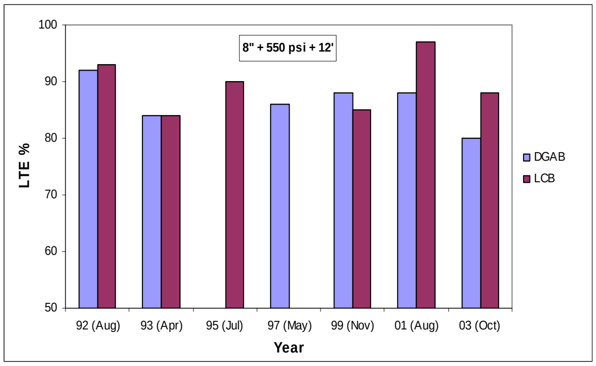
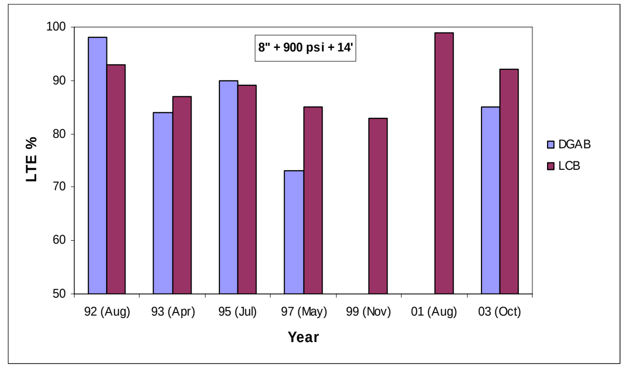


Figure 6. Comparison of IRI values for the sections with DGAB and LCB



(a)



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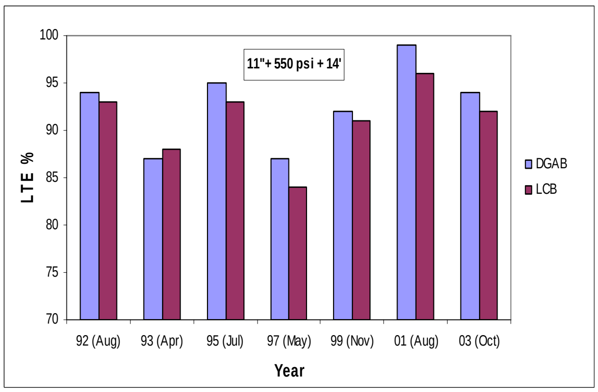

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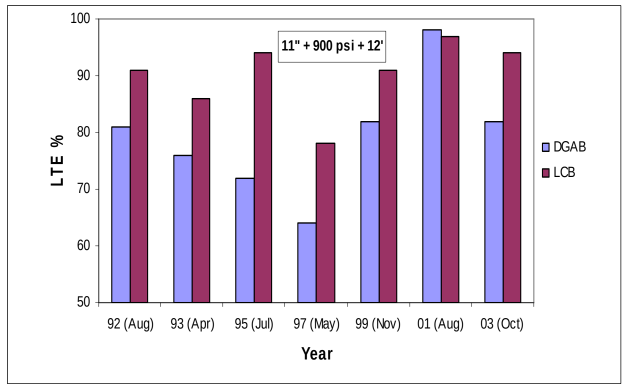

(d)

Figure 7. Comparison of LTE values with time for the sections with DGAB and LCB bases

Control section vs. DGAB

As mentioned, the KDOT control section consists of a 12-in. PCC slab over a 6-in. dense graded portland cement-treated base (PCTB). This section's performance has been compared with the four DGAB-based sections: DGAB-1 [KDOT 10], DGAB-2 [KDOT 9], DGAB-3 [KDOT 12], and DGAB-4 [KDOT 11].

Figure 8 shows the change of IRI values with time for the sections with a DGAB base and the KDOT control section with a stabilized base. It is evident that the section (KDOT 9) with the widened lane (14 ft) and 900-psi design strength has remained the smoothest over time. However, this section has also showed some cracking recently (Figure 9) and has been repaired.

The KDOT control section can be described as the second best in terms of smoothness or IRI. The KDOT 10 (DGAB-1) section with 8-in. PCC slab thickness and 550-psi design strength has shown the highest increase in roughness. As mentioned earlier, this section has shown some distresses (Figure 9) and repairs have been made.

The joint LTE values of different sections with DGAB bases have also been compared with the values for the KDOT control section with PCTB, as shown in Figure 10. As expected, the KDOT control section has shown very good (greater than 80%) LTE values with time. However, all sections have shown good LTE most of the time. The LTE values for sections KDOT 9 (DGAB-2) and KDOT 11 (DGAB-4) have shown very high fluctuations over time.

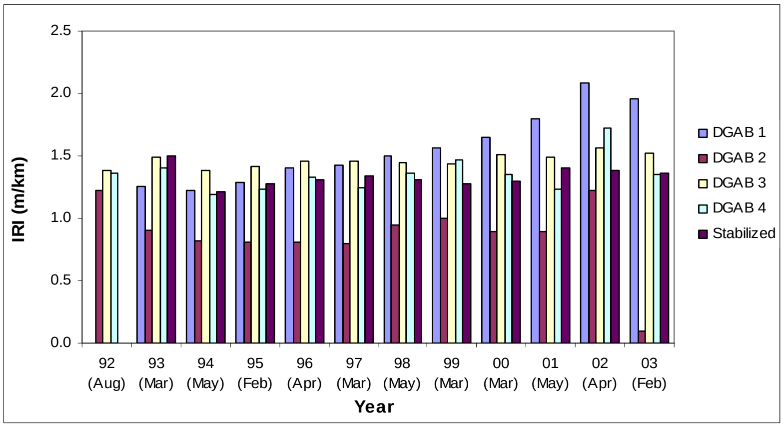

Figure 8. Change in IRI with time for the KDOT control section

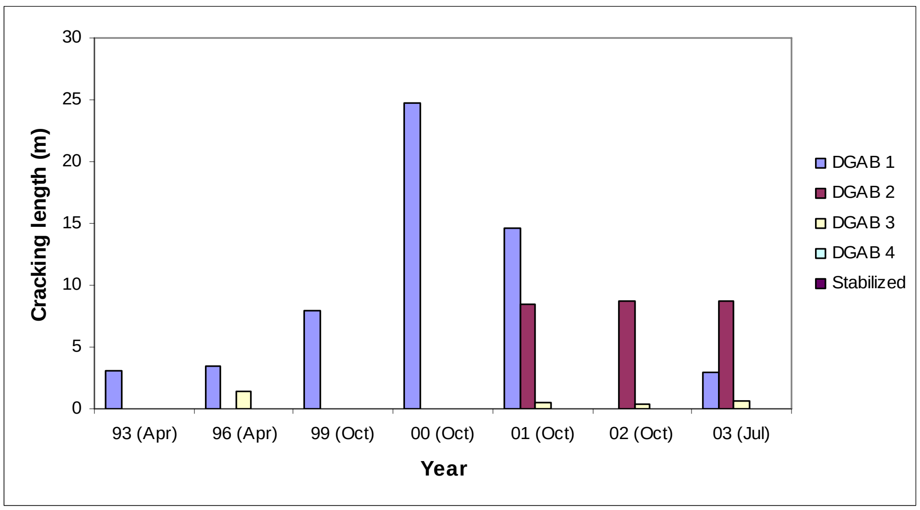

Figure 9. Cracking on the sections with DGAB bases

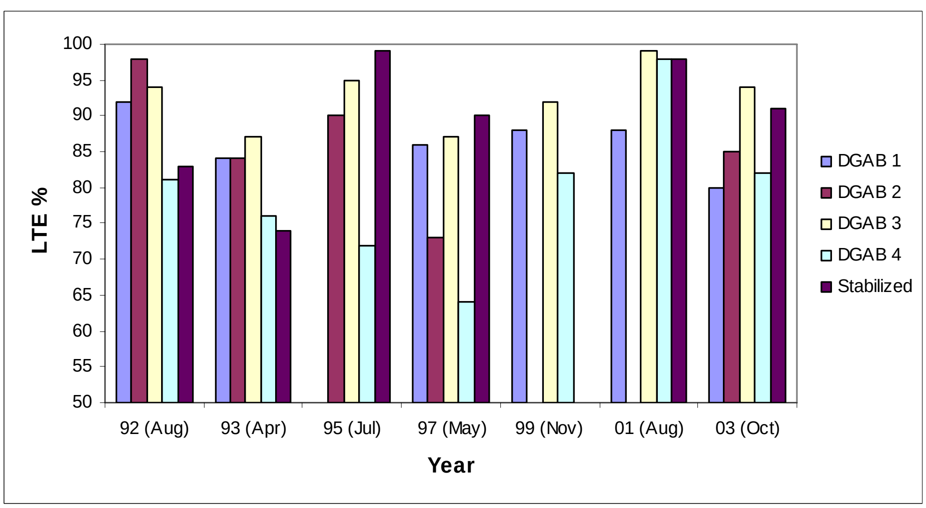

Figure 10. Change in LTE with time for the KDOT control section

LTPP EVALUATION OF KANSAS SPS-2 PROJECT

The LTPP program recently published a report on the initial evaluation and analysis of all SPS-2 projects using performance data up to 2000 (Jiang and Darter 2005). The study has found that the Kansas SPS-2 project is in good shape. The report presents the following findings for the SPS-2 project in Kansas:

(1) Designed vs. constructed performance is generally good. The key deviations from the LTPP guidelines are the following:

- Mean slab thickness values for five sections are more than 1/2 in. higher than the designed values.
- Mean 14-day flexural strength values for the lower strength concrete are more than 10% above the design values.
- Construction difficulties and deviations are relatively minor.

(2) Data availability is excellent overall.

- Site condition data are good, except for deficient traffic data and missing automatic weather station data.
- Only 66% of the cores have been tested.
- Good monitoring data, except for deficient faulting data, are available.

CONCLUSION

The 12-year performance of the Kansas SPS-2 project has been analyzed in this study with respect to the design features of drainage and base type. Based on this study, the following conclusions can be drawn:

- The Kansas SPS-2 project has performed very well to date. Most sections are smooth, crack free, and have negligible faulting. The LTE of the sections has been good too.
- The drainable PATB sections have performed the best. These sections were built smoother and remained so after 12 years of service.
- The LCB sections have not performed well in terms of smoothness, although they have maintained very good LTE.
- The section with low PCC slab thickness (8 inches) and low concrete design strength (550 psi) on DGAB has performed the worst.
- The combination of high slab thickness and high design strength tends to mask the effect of the base on pavement performance.
- The KDOT control section with a thick slab (12 inches) over dense graded portland cement treated base has also performed very well.

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Employment and Spatial Effects of Highways in Missouri

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ABSTRACT

The purpose of this paper is to determine if there is a relationship between highway development and the spatial pattern of employment growth in Missouri. For determining if there is a spatial pattern to the employment growth in the county, an OLS model was first run and a spatial error model was estimated and contrasted with a simpler model without considering spatial statistics relationships. My intention is to figure out how the region's highway affects employment. Results in model 1 related to Interstate highways suggest that Interstate highways in Missouri do not have positive effects on employment growth. In model 2 related to US highways, the results show that US highways have distance effects and distance decay. As for road factors with 2 lanes and 4 lanes in both models, variable of mileage of road with 2 lanes has a positive sign, but 4 lanes show negative sign. Spatial autoregressive coefficients (LAMDA) in both models are negative, implying there is negative spatial dependence between the counties.

Key words: employment growth—highway development—spatial pattern

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

State highway investment projects are often justified on grounds that such efforts will make positive economic impacts. In particular, road network improvements are known as a good means of bringing development to undeveloped areas, including rural area. In some manner, transportation routes have been promoted as vehicles for commerce in the U.S. For rural areas experiencing economic distress, such policies are often welcomed with open arms. Empirical evidence suggests that there is a close relationship between the presence of infrastructure (highways, etc.) and economic development. In general, however, evidence is less certain as to whether road investments play a role in the economic growth of rural areas specifically.

Figure 1 shows the trend of highway capital outlay in Missouri State during the years 1990 to 2003. The highway outlay of 2003 was 1,258,831 thousand dollars, down 3.2 percent from last year. Missouri had increased highway capital outlay by average 9.4% for 13 years. The outlay had been over 1,000,000 thousand dollars from 2000. The growth of highway capital outlay between 2001 and 2000 is the highest—23.3%, and the second highest growth is that from 1999 to 2000—16.3%. The lowest growth during the period is from 1994 to 1995, about –21%.

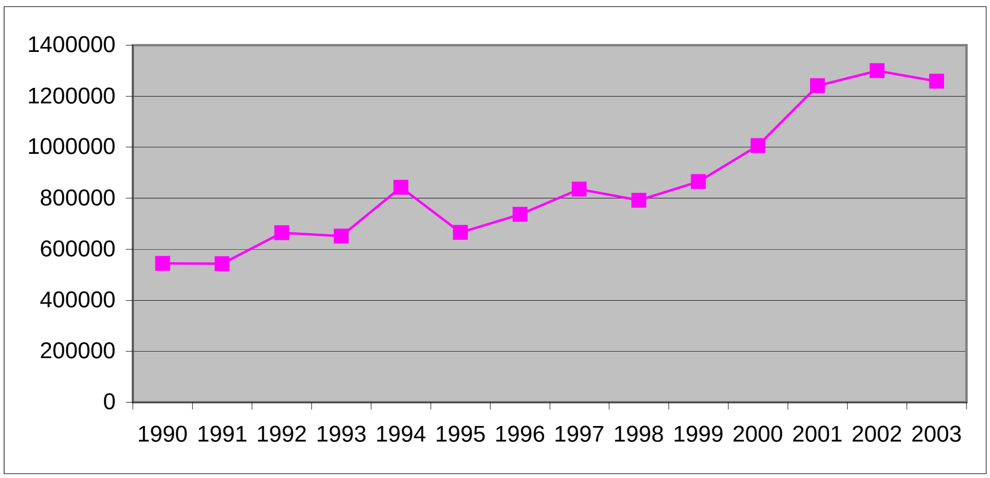

Figure 1. Missouri highway capital outlay (Highway Statistics Publication 1990-2003)

RESEARCH OBJECTIVES

Using geographic information system (GIS) techniques and spatial econometric methods, this paper analyzes a spatial pattern to the employment growth process in the region. OLS models were run and spatial effects were examined. A spatial error model was used in order to remove a spatial dependence. A spatial error model was adopted through diagnostic tests for the robustness of the results. Finally, maps of employment growth residuals were made to analyze whether the spatial patterns respond to the socioeconomic trends in the region from 1990 to 2000.

The model uses employment growth as the dependent variable. The purpose of this paper is to study how the region's highway affects employment. As we shall see, results suggest that highway's spatial spillover effects might exist on US highways but not on Interstate highways.

LITERATURE REVIEW

David A. Aschauer (1990), Alica H. Munnell (1990, 1992), and M. Ishaq Nadiri and Theofanis P. Mamuneas (1994) found that public capital has important contributions to output and economic growth. Researchers such as Douglas Holtz-Eakin (1994), and Teresa Garcia-Mila and Therese McGuire (1992) found that public capital has negligible effects on economic growth.

Munnell (1990a) shows that a 1% increase in public capital increases output by 0.34%. She suggests that it indicates a marginal productivity of public capital of roughly 60%. She estimated the marginal productivity of public and private capital. Munnell (1990b) looks at the relationship between public capital and measures of economic activity and the state level by using production function. In her research, public capital has a significant positive impact on the output. Her coefficient on public capital is 0.15, which is smaller than the 0.35 estimated by Aschauer (1989). Munnell suggests that public capital stimulates the productivity of private capital, and public capital is a substitute for private capital.

Kelejian and Robinson (1997) try to formulate an empirical production function model with spatial variables. They consider Cobb-Douglas production function and spatial correlation model. Their results show that the important factor to see if elasticity estimates in production function models involving infrastructure variables are significant or not is whether econometric problems are considered or not. They suggest that accommodating spatial correlation also reduces the magnitude and significance of the estimated productivity impacts.

Cohen and Paul (2002) reevaluate the "public capital hypothesis" for the U.S. manufacturing sector for 1986-1992. The results show that intra-state public infrastructure investment is significantly productive. Spatial spillovers complement the cost-saving impacts of within-state public infrastructure capital. They argue that output growth by the cost-depressing impact of infrastructure investment may stimulate capital investment and labor employment.

Morrison and Schwarz (1996) compute a direct cost-benefit measure of infrastructure investment and analyze the rates of return to investment. Their analysis is based on cost-side marginal products and productivity-growth measures. They measure the impacts of these effects on costs and thus on productivity. Their results show that the return to public capital is both positive and significantly different from zero.

Tortorice (2002) analyzes the role of government infrastructure in the production process. He finds its effect on the variable costs of private industries and overall TFP growth by using cost function. The results indicate that the changes in government capital stocks have a significant role in the production process of the economy. The elasticity of labor costs with respect to government capital is 0.02 and the elasticity of productivity with respect to government capital is 0.003.

Boarnet (1998) tests the existence of negative output spillovers from street-and-highway capital. His result shows that street-and-highway capital affects output in California counties, and that such infrastructure makes negative output spillovers across counties of similar urban character. This research suggests that infrastructure investment is productive for counties, but public capital also creates negative output spillovers.

Holtz-Eakin and Schwartz (1995) examine how state highways provide productivity benefits beyond the narrow confines of each state's borders. They use state level production function. They estimate basic production function ignoring spillover effects in state highways and including spillover effects. In the basic production function regressions, the results do not show empirical support for the notion that public infrastructure has significant cross-state effects on output and productivity. In the production function including full spillover effects, the results do not support the notion that a state's effective stock of highways depends upon the provision of highway by its neighbors.

Holl (2004) analyzes the impact of road infrastructure on the location of new manufacturing establishments in Spanish municipalities from 1980 to 1994. The empirical results suggest that road transport infrastructure is important, motorways affect the spatial distribution of new manufacturing establishments, and road infrastructure has a differential impact across manufacturing sectors.

Stephan (1997) provides an estimation of road infrastructure's impact on production in the manufacturing sector from an ex-post perspective and takes econometric issues such as autocorrelation, heteroskedasticity and cross-sectional correlation, and nonstationarity of data into account. He finds a strong positive and significant correlation between road infrastructure and the manufacturing sector's output.

Nadiri and Mamuneas (1996) identify the contribution of output demand, relative input prices, technical change, and publicly financed capital to total factor productivity growth, and provide a general framework for analyzing and measuring the contribution of highway capital to private sector productivity growth.

Canaleta et al. (1998) examine the impact of infrastructure on productivity in the various regions of Spain and provide an approximate estimation of the impact of various types of infrastructure on Spanish regional production costs in the agricultural, industrial, and services sectors in the period 1964-1991. They examine how industry and services show similar cost/public capital elasticity—slightly higher in the secondary sector and greater than in agriculture. The results show that public capital reduces private production costs. They try to reveal the presence of so-called “spillover effects.”

RESEARCH MODEL AND DATA

This research was executed for cross-sectional units (e.g., 114 counties and St. Louis city) in Missouri State. The models consist of two models: one is related to Interstate highways and another to US highways. OLS was run and spatial error model was used.

Model 1 related to Interstate highways:

$$\begin{aligned}EMPGRT = {} & \alpha_0 + \alpha_1 (EDRT) + \alpha_2 (PPLTDNST) + \alpha_3 (PVRT) + \alpha_4 (HHIM) + \alpha_5 (UEMPLYMT) + \alpha_6 \\ & (GSTR) + \alpha_7 (RVLTN) + \alpha_8 (AWPJ) + \alpha_9 (LANES2) + \alpha_{10} (LANES4) + DVRL + DVIS2 \\ & + DVIS6 + DVIS12 + \varepsilon_i \quad \ldots\ldots\ldots \qquad (1)\end{aligned}$$

Model 2 related to US highways:

$$\begin{aligned}EMPGRT = {} & \beta_0 + \beta_1 (EDRT) + \beta_2 (PPLTDNST) + \beta_3 (PVRT) + \beta_4 (HHIM) + \beta_5 (UEMPLYMT) + \beta_6 \\ & (GSTR) + \beta_7 (RVLTN) + \beta_8 (AWPJ) + \beta_9 (LANES2) + \beta_{10} (LANES4) + DVRL + DVUS2 \\ & + DVUS6 + DVUS12 + \varepsilon_j \quad \ldots\ldots\ldots \qquad (2)\end{aligned}$$

The dependent variable is EMPGRT that is the change rate of employment growth between 1990 and 2000. This paper uses employment of civilian labor force, published by the Economic & Policy Analysis Research Center (EPARC) in University of Missouri-Columbia. Table 1 gives explanations of the explanatory variables.

The demographic and socio-economic data such as educational achievement, population density, poverty status, household income, unemployment rates, gross sales tax receipts, real property valuation, and average wage per job were obtained from the Missouri QuickFacts from U.S. Census Bureau, Economic Research Service of USDA, from U.S. Bureau of Economics Analysis, U.S. Department of Labor, and State Tax Commission of Missouri.

Geographic information system (GIS) calculates distance between base cities and highways. SPSS program was used to run OLS and GeoDa program to do spatial model.

RESULTS

The preliminary results of model 1 (Table 2) don't match my expectation, according to transportation factors like LANES2, LANES4, DVIS2, DVIS6, and DVIS12.

In model 1, population density, real property valuation, dummy of rural county, and dummy variable for county with base city within a distance of 2.1 to 5.9 miles from nearest Interstate highways have statistically significant coefficients with expected signs.

Table 1. Independent variables

Variable	Definition	Scale	Expected effect	Data Source
EDRT	Change rate of percent of people, 25 and older, with High school degree or higher between 1990 and 2000	County	+	US Census
PLTDNTY	Change rate of person per square mile of land area between 1990 and 2000	County	+	US Census
PVTY	Change rate of number of people below poverty level between 1989 and 1999	County	-	US Census
HHIM	Change rate of household income between 1989 and 1999	County	+	US Census
UEMPLYMT	Change rate of unemployment Rate between 1990 and 2000	County	-	US Census
GSTR	Change rate of gross sales tax receipts between 1990 and 2000	County	+	US Census
RVLTN	Change rate of real property valuation between 1990 and 2000	County	+	US Census
AWPJ	Change rate of average wage per job between 1990 and 2000	County	-	US Census
LANES2	Change of mileage of road with 2 lanes between 1990 and 2000	County	+	MO Dept. of Transportation
LANES4	Change of mileage of road with 4 lanes between 1990 and 2000	County	+	MO Dept. of Transportation
DVRL	Dummy to reflect rural county	County	-	US Census
DVIS2	Dummy variable for county with base city within a distance of 2 miles from nearest Interstate highways	County	+	GIS
DVIS6	Dummy variable for county with base city within from a distance of 2.1 miles to 5.9 miles from nearest Interstate highways	County	+/-	GIS
DVIS12	Dummy variable for county with base city within from a distance of 6 miles to 11.9 miles from nearest Interstate highways	County	-	GIS
DVUS2	Dummy variable for county with base city within a distance of 2 miles from nearest US highways	County	+	GIS
DVUS6	Dummy variable for county with base city within from a distance of 2.1 miles to 5.9 miles from nearest US highways	County	+/-	GIS
DVUS12	Dummy variable for county with base city within from a distance of 6 miles to 11.9 miles from nearest US highways	County	-	GIS

Among the variables that are significant, population density, real property valuation, and average wage per job in model 1 have positive signs, implying that an increase would induce an increase in employment growth in the region. Positive average wage per job means that the counties that have high average wage per job would have higher employment.

Dummy variables of rural county and county with base city within 2 miles and from 2.1 to 5.9 miles from nearest Interstate highways have negative signs in model 1, meaning that rural area and counties within 5.9 miles from nearest Interstate highways have negative employment growth.

On the other hand, the results of model 2 (Table 3) about transportation variables coincide with the expected sign. In model 2, population density, real property valuation, and mileage of road with 2 lanes show statistically significant coefficients that matched expectation.

Table 2. OLS regression results of model 1

Variable	Coefficient	Std.Error	t-value	Probability
CONSTANT	0.017	0.063	0.276	0.783
EDRT	-0.114	0.190	-0.603	0.548
PPLTDNST	0.204	0.097	2.107	0.038
PVRTY	0.083	0.090	0.917	0.362
HHIM	-0.115	0.192	-0.601	0.549
UEMPLYMT	-0.008	0.052	-0.154	0.878
GSTR	0.017	0.013	1.311	0.193
RVLTN	0.253	0.047	5.340	0.000
AWPJ	0.232	0.104	2.235	0.028
LANES2	0.001	0.000	1.418	0.159
LANES4	0.000	0.001	-0.393	0.695
DVRL	-0.049	0.029	-1.701	0.092
DVIS2	-0.075	0.029	-2.542	0.013
DVIS6	-0.058	0.034	-1.698	0.093
DVIS12	0.081	0.103	0.784	0.435

R-squared: 0.667

Table 3. OLS regression results of model 2

Variable	Coefficient	Std.Error	t-value	Probability
CONSTANT	-0.068	0.070	-0.960	0.339
EDRT	-0.015	0.188	-0.080	0.937
PPLTDNST	0.215	0.098	2.189	0.031
PVRTY	0.168	0.084	1.988	0.050
HHIM	0.092	0.182	0.504	0.615
UEMPLYMT	0.014	0.052	0.268	0.789
GSTR	0.021	0.013	1.548	0.125
RVLTN	0.236	0.048	4.916	0.000
AWPJ	0.197	0.105	1.876	0.064
LANES2	0.001	0.000	2.046	0.043
LANES4	0.000	0.001	-0.348	0.729
DVRL	-0.019	0.028	-0.671	0.504
DVIS2	0.025	0.029	0.843	0.401
DVIS6	0.026	0.035	0.746	0.457
DVIS12	-0.159	0.110	-1.443	0.152

R-squared: 0.654

Among the variables that are significant in model 2, population density, poverty rate, real property valuation, average wage per job, and mileage of road with 2 lanes have positive signs, implying that an increase would bring an increase in employment growth.

Anselin and Rey (1991) tried to figure out how the Moran I and Lagrange multiplier tests are used by different situations, different sample sizes, alternative spatial structure, and under the nonstandard error distributions. Their results are very sensitive to the properties of the tests by using spatial weights matrix. They suggest that the Lagrange multiplier tests are the most powerful in deciding the choice between a spatial error model and a spatial lag model.

Kelejian and Robinson (1992) show a large sample test for spatial autocorrelation in terms of error terms of regression models. Their results indicate that omitted independent variables may bring spatial autocorrelation in the error terms. Nass and Garfinkle (1992) introduce the localized autocorrelation diagnostic statistic (LADS). The LADS is an error diagnostic. They suggest that LADS is a good method to diagnose localized residuals and to identify omitted variables in models.

For this project, Anselin's methods were used in analyzing and quantifying spatial effects. GeoDa has two tests for diagnostics for spatial dependence: Moran's I and Lagrange Multiplier test.

Moran's I statistics is used (Moran 1948; Cliff and Ord 1981) to estimate and test hypotheses. Moran's I measures spatial autocorrelation in regression residuals.

$$I = \frac{n}{\sum_i \sum_j w_{ij}} \cdot \frac{\sum_i \sum_j w_{ij}(y_i - \mu)(y_j - \mu)}{\sum_i (y_i - \mu)^2} \tag{3}$$

where w_{ij} the elements of the spatial weight matrix, W, μ the mean of all y observations, and $i, j = 1,\ldots,n$. A positive and significant value of Moran's I indicates positive spatial correlation, showing that counties have levels of employment (high or low) similar to their neighboring counties. A negative and significant value for Moran's I indicates negative spatial correlation, showing that counties have levels of employment unlike neighboring counties, and a low value may be surrounded by high values in nearby counties.

Moran's I test on the OLS yields insignificant negative values in model 1, -0.115603 ($z = -1.5289187$ and $p < 0.1262846$), and in model 2, -0.106429 ($z = -1.3734012$ and $p < 0.1696278$). These show an insignificant negative spatial relationship (Anselin 1988, 1995), indicating the needs for another test for spatial dependence.

Lagrange Multiplier was used for another test. Lagrange Multiplier tests on the OLS in model 1 show that robust LM (lag) is an insignificant positive value, 1.5345774 ($p < 0.2154268$), and robust LM (error) is a significant positive value, 4.8478735 ($p < 0.0276802$). The latter indicates that spatial dependence exists and spatial error model is better than spatial lag model for spatial model because LM-error test has high value and lower probability.

Lagrange Multiplier tests on the OLS in model 2 show that robust LM (lag) is an insignificant positive value, 0.4452569 ($p < 0.5045960$), and robust LM (error) is a significant positive value, 2.7379851 ($p < 0.0979883$). The latter indicates also that there is spatial dependence and spatial error model should be used because LM-error test has high value and lower probability. Therefore, spatial error model was used for spatial analysis considering spatial effects in both models.

The residuals of employment growth were mapped from the OLS model in order to determine if there was a spatial pattern. GeoDa calculates residuals. A residual map gives an indication of systematic over- or under-prediction in counties, indicating there is spatial autocorrelation. OLS residuals make a standard deviation map for the residuals.

This map suggests that similarly colored areas tend to be located in similar locations, indicating positive spatial autocorrelation (Anselin 2005). Also, it means a tendency to over-predict (negative residuals) in the outlying areas and a tendency to under-predict (positive residuals) in the core, implying the possible presence of spatial heterogeneity (Anselin 2005). A negative standard deviation means that the predicted values exceeded the actual value.

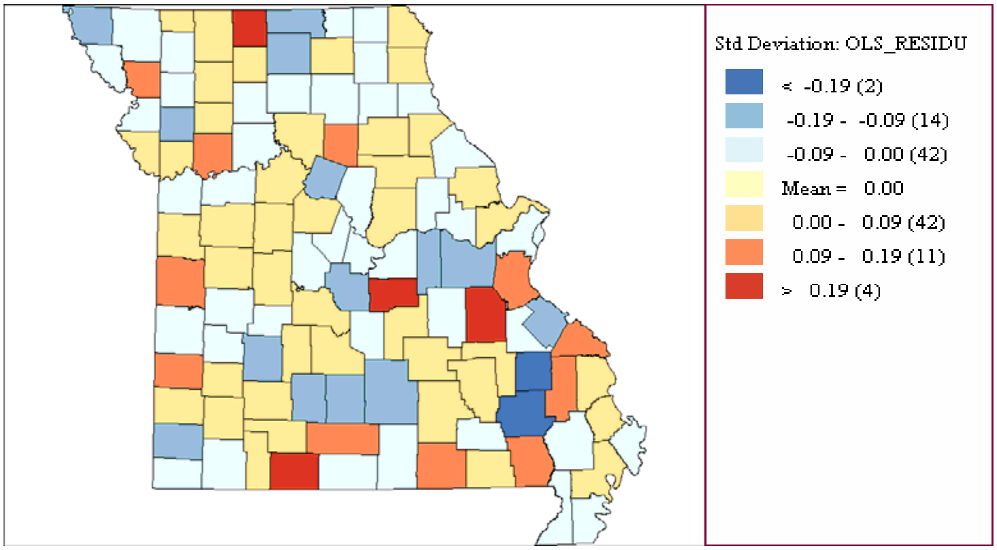

Figure 2. Mapping the OLS residuals of employment growth for model 1

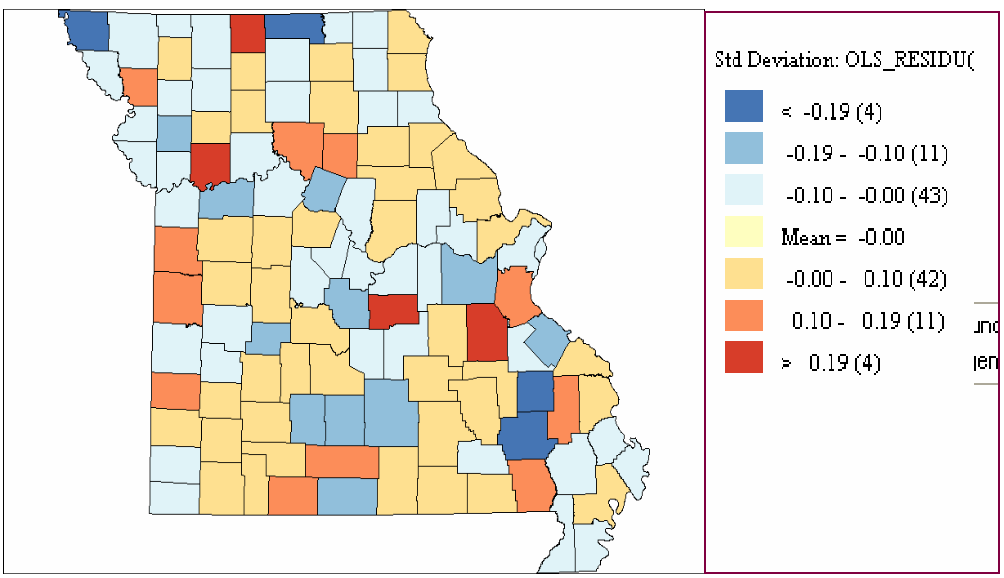

Figure 3. Mapping the OLS residuals of employment growth for model 2

Figure 2 and 3 have almost same portion in the OLS model that over-predicts and under-predicts employment growth. There is some clustering.

There exists spatial dependence when the dependent variable or error term at each location is correlated with the dependent variable or the error term at other locations (Anselin 2001; Islam 2003). The spatial error model reflects errors that are not homoskedastic and uncorrelated. In the spatial error model, OLS remains unbiased, but it is no longer efficient.

The spatial errors model is shown in equation (4), where the disturbances exhibit spatial dependence. Anselin (1988) provides a maximum likelihood method for this model.

$$
\begin{aligned}
y &= X\beta + u \\
u &= \lambda W\mu + \varepsilon \\
\varepsilon &\sim N(0; \sigma^2 I_n)
\end{aligned}
\tag{4}
$$

where y contains an n*1 vector of dependent variables and X represents the usual n*k data matrix containing explanatory variables. W is a known spatial weight matrix and the parameter λ is a coefficient on the spatially correlated errors analogous to the serial correlation problem in time series models. The spatial autoregressive parameter λ thus displays the strength of correlation between the disturbance term u and the weighted average of the disturbance terms of neighboring counties Wu. The parameters β reflect the influence of the explanatory variables on variation in the dependent variable y.

Checking the order of the Wald (W), Likelihood Ratio (LR), and Lagrange Multiplier (LM) statistics on the spatial autoregressive error coefficient in model 1, it was found that W = 7.04 (the square of the z-value of the asymptotic t-test), LR = 5.42, and LM = 4.85. This corresponds to the expected order (W > LR > LM) (Anselin 1988, 2005) and indicates that this Maximum Likelihood Estimation (MLE) is good.

In model 2, it was found that W = 6.45, LR = 4.76, and LM = 3.10. This corresponds also to the expected order (W > LR > LM) (Anselin 1988, 2005) and indicates that this Maximum Likelihood Estimation (MLE) is good too.

The spatial autoregressive coefficient (LAMDA) in model 1 is estimated as -0.403, and is highly significant ($p < 0.0079871$). As in the OLS result, population density, real property valuation, and average wage per job in model 1 have significant positive signs.

Dummy variables of rural county and for county with base city within 2 miles and from 2.1 to 5.9 miles from nearest Interstate highways have significant negative signs. As for LANES2 and LANES4 in model 1, variable of mileage of road with 2 lanes has a positive effect, but 4 lanes show negative sign, indicating that the investment in a 4-lane road can actually cause a decline in employment growth among counties.

Like the OLS results, results of spatial error model in DVIS2, DVIS6, and DVIS12 do not reveal also distance effects between employment growth and Interstate highways in Missouri. Therefore, unlike other researches, results in model 1 suggest that Interstate highways in Missouri do not have positive impacts on employment growth.

Table 4. Spatial error model for model 1 - maximum likelihood estimation

Variable	Coefficient	Std.Error	z-value	Probability
CONSTANT	0.0185865	0.0510535	0.3640591	0.7158140
EDRT	-0.1326345	0.1457388	-0.9100834	0.3627784
PPLTDNST	0.2018393	0.0847898	2.380466	0.0172907
PVRTY	0.0823759	0.0781049	1.054684	0.2915699
HHIM	-0.1460195	0.1703483	-0.8571819	0.3913442
UEMPLYMT	-0.0011165	0.0472104	-0.0236512	0.9811307
GSTR	0.0167383	0.0113513	1.474574	0.1403272
RVLTN	0.2626993	0.0421149	6.23768	0.0000000
AWPJ	0.2417196	0.0896859	2.695177	0.0070352
LANES2	0.0004833	0.0003182	1.518902	0.1287873
LANES4	-0.0004981	0.0008109	-0.614143	0.5391207
DVRL	-0.0495095	0.0238689	-2.074228	0.0380580
DVIS2	-0.0745099	0.0251698	-2.96029	0.0030736
DVIS6	-0.0613473	0.0302371	-2.028876	0.0424708
DVIS12	0.1033375	0.0927709	1.113899	0.2653226
LAMBDA	-0.4032255	0.1520105	-2.652617	0.0079871

R-squared: 0.691911

Table 5. Spatial error model for model 2 - maximum likelihood estimation

Variable	Coefficient	Std.Error	z-value	Probability
CONSTANT	-0.0541582	0.0570368	-0.9495316	0.3423503
EDRT	-0.0666669	0.1457682	-0.4573486	0.6474205
PPLTDNST	0.2203311	0.0870867	2.53002	0.0114056
PVRTY	0.1880665	0.0714325	2.632788	0.0084688
HHIM	0.1012832	0.1569291	0.6454075	0.5186630
UEMPLYMT	0.0364700	0.0455579	0.80052	0.4234095
GSTR	0.0184717	0.0118377	1.560416	0.1186616
RVLTN	0.2417855	0.0431125	5.608244	0.0000000
AWPJ	0.1905960	0.0903894	2.108609	0.0349782
LANES2	0.0007108	0.0003237	2.195959	0.0280948
LANES4	-0.0004253	0.0008408	-0.5057739	0.6130154
DVRL	-0.0169307	0.0234479	-0.7220547	0.4702607
DVUS2	0.0291454	0.0258978	1.125399	0.2604201
DVUS6	0.0292870	0.0307224	0.9532773	0.3404496
DVUS12	-0.1578530	0.0968193	-1.630387	0.1030197
LAMBDA	-0.3860620	0.1520762	-2.538609	0.0111295

R-squared: 0.677203

The spatial autoregressive coefficient (LAMDA) in model 2 is -0.386 and is significant ($p < 0.0111295$). Population density, poverty rate, real property valuation, average wage per job, and mileage of road with 2 lanes are statistically significant coefficients. As for LANES2 and LANES4 in model 2, variable of mileage of road with 2 lanes has also a positive sign, but variable of 4 lanes gives negative impact. Although dummy variables for county with base city within 2 miles and from 2.1 to 12 miles from nearest US highways are insignificant, these variables show that US highways have distance effects and distance decay.

Spatial autoregressive coefficients (LAMDA) in model 1 and 2 have negative signs in the spatial econometric regressions, indicating negative spatial dependence between the counties. The diagnostics for spatial dependence in OLS suggest a spatial error correlation, implying that the spatial error model is better than the spatial lag model according to the information criteria. Therefore, we might expound the spatial correlation in the data as obstacles apparent to disturbance terms correlated negatively spatially. Incoherencies of data might be one reason for such spatial obstacles in the data.

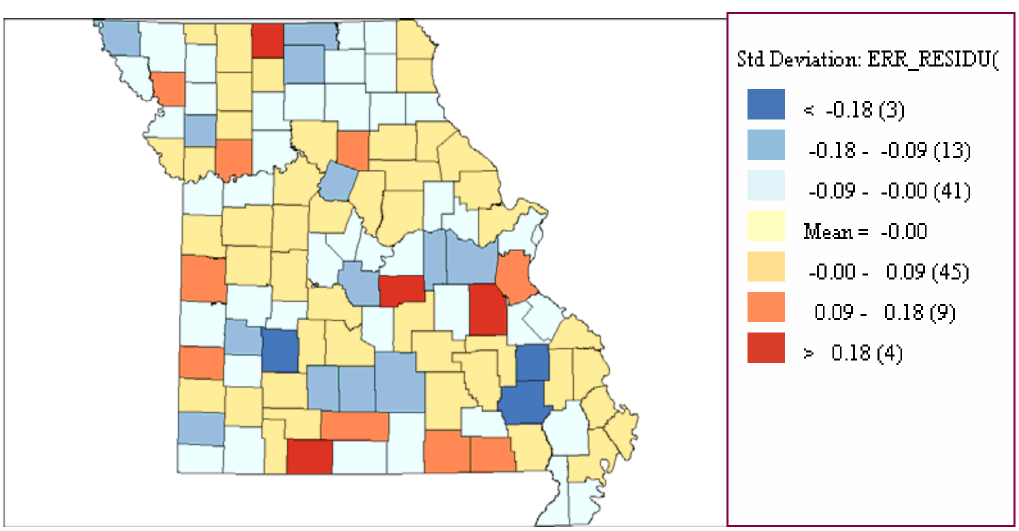

Figure 4. Mapping residuals of employment growth from the spatial error model for model 1

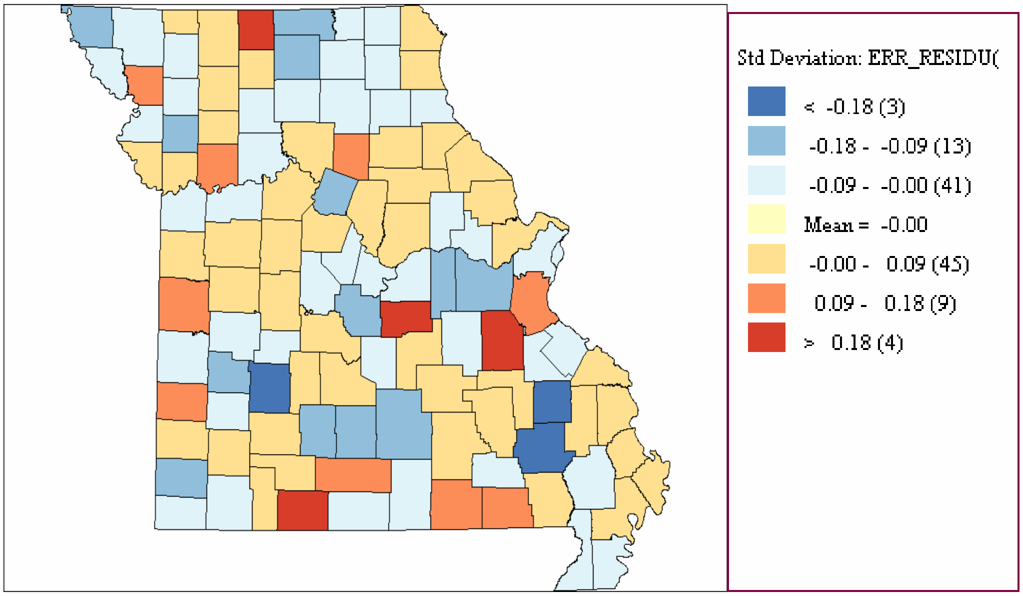

Figure 5. Mapping residuals of employment growth from the spatial error model for model 2

Maps of the spatial error model residuals (Figures 4 and 5) show that the spatial patterns of employment growth are a little different to the OLS model. Across Missouri, in both model 1 and model 2, the portion of under-predicted and over-predicted areas in employment growth is almost 50%.

CONCLUSION

Though this research analyzes only Missouri State, the results may have expanded adaptation. Economic development advocates and public officials often advance the notion that more highways will automatically lead to more development. The results of this research may help in developing consistent regional policies that ensure greater efficiency of highway capital and better evaluate user benefits. It appears that highways contribute to accessibility and can improve regional growth and development. However, the relationship between highway investment and economic development is multifaceted and highly complex.

In this analysis, results in model 1 related to Interstate highways suggest that Interstate highway in Missouri doesn't have positive effects on employment growth. In model 2 related to US highways, the results indicate that US highways have distance effects and distance decay.

As for roads with 2 lanes and 4 lanes in both models, variable of mileage of road with 2 lanes has a positive sign, but 4 lanes show negative sign. It indicates that the investment of 4-lane road may induce a decline in employment growth among counties.

Negative spatial autoregressive coefficients (LAMDA) in both models imply negative spatial dependence between the counties. There might be the spatial correlation in the error term correlated negatively spatially. Incoherencies of data might cause spatial problem in the data.

It is expected that this research helps identify and quantify potential spatial relationships between highway investments and economic growth. Still, this paper has explored a limited number of econometric and spatial econometric specifications, and further investigation may prove profitable. At the current stage of my research into spatial effects of highway in Missouri, there are some questions to reflect on. The potential interaction between highway and new establishment of firms could be another good research topic. It is necessary to research the differences between using population centroids and cities and the distance decay due to being far away from highways.

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Sensitivity Study of Design Input Parameters for Two Flexible Pavement Systems Using the Mechanistic-Empirical Pavement Design Guide

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ABSTRACT

Many agencies use the *AASHTO Guide for Design of Pavement Structures* to design their pavement systems. The limitation inherent in this method is the empirical nature of the decision process, which was derived from a road test conducted almost 45 years ago in Ottawa, Illinois. The newly released Mechanistic-Empirical Pavement Design Guide (MEPDG), based on NCHRP Study 1-37A, has adopted a mechanistic-empirical pavement design procedure, in which pavement distresses are calculated through calibrated distress prediction models based on material properties laboratory test results and local climatic conditions. The calibrated distress prediction models are based on the critical pavement responses mechanistically calculated by a structural model and coefficients determined through national calibration efforts using the Long-Term Pavement Performance (LTPP) database. The MEPDG requires many parameters to map the calibrated distress prediction models with traffic, environment, and material properties. The present study was conducted to evaluate the relative sensitivity of MEPDG input parameters to asphalt cement concrete (ACC) properties, traffic, and climatic conditions based on field data from two existing Iowa flexible pavement systems. The sensitivities of five MEPDG performance measures (longitudinal cracking, alligator cracking, thermal cracking, rutting, fatigue cracking, and smoothness) were studied by either varying a single input parameter or by varying two input parameters at a time. The findings of this study, presented in this paper, will provide pavement designers a better understanding of those design parameters that affect certain pavement distresses the most and need careful consideration during the design process.

Keywords: flexible pavements—mechanistic-empirical pavement design guide—pavement analysis and design—sensitivity study

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INTRODUCTION

The newly released Mechanistic-Empirical Pavement Design Guide (MEPDG) (based on NCHRP Study 1-37A) predicts pavement distresses using calibrated distress prediction models, based on critical pavement responses computed by a structural model (like JULEA for flexible pavements) and coefficients determined through national calibration efforts using the Long-Term Pavement Performance (LTPP) database. A mechanistic-empirical pavement design procedure requires many input parameters related to traffic, climate, and material conditions and presents the predicted pavement distresses with time series. In such situations, the pavement designers should consider the traffic and climatic conditions in the pavement construction area and decide the thickness of each layer and the material properties that satisfy the distress criteria in the pavement design life by running the program several times with varying input parameters.

Recent studies (Mohamed and Witczak 2005a; Mohamed and Witczak 2005b) reported the sensitivities of some of key input parameters for the permanent deformation model and the fatigue model in conventional flexible pavement. In these studies, design input parameters were grouped into three different levels based on the severity (low, medium and high), and the magnitude of the input parameter in question was changed in the medium stage. Though these studies provided general information on input parameter sensitivities in the MEPDG, they focused only on conventional three-layer flexible pavement structures (an HMA surface, granular base, and subgrade), which are not currently used for interstate and state roads in Iowa.

This paper presents the results of a study conducted on the relative sensitivity of MEPDG input parameters to the properties of asphalt cement concrete (ACC), traffic, and climate using field data from two existing Iowa flexible pavements (US-020 in Buchanan County and I-80 in Cedar County). The sensitivities of five MEPDG performance measures were studied either by varying a single input parameter or by varying two input parameters at a time for these pavements.

MECHANISTIC-EMPIRICAL FLEXIBLE PAVEMENT DESIGN

A major limitation of the current AASHTO pavement design procedure is the empirical nature of the thickness decision process, which is derived from a road test conducted almost 45 years ago at a single location in Ottawa, Illinois. This empirical approach cannot be applied to current pavements systems with increased traffic volumes, different climate conditions, different pavement construction areas, etc. In recognition of the limitation of current AASHTO guide, the AASHTO Joint Task Force on Pavements initiated an effort to develop an improved pavement design guide based on mechanistic-empirical principles (NCHRP 1-37A 2004). The product of this effort is the newly released MEPDG, based on NCHRP Study 1-37A.

The major components of the MEPDG are the input system, mechanistic pavement analysis model, transfer functions, and an output system that consists of predicted pavement distresses. A new feature in the MEPDG, absent from the current AASHTO design guide, is the availability of hierarchical input levels. Users have the option to choose to any one of them for design. Depending on the accuracy of the input parameter, three levels of input are provided, from level 1 to level 3. However, it should be recognized that irrespective of the input design level, the computational algorithm for the design procedure is the same. In addition, a mix of levels can be used for a given design project. To decide on a suitable input level, a designer should recognize which input parameter would be important for the results.

The mechanistic pavement analysis model used in the MEPDG for flexible pavements is the multilayer elastic program JULEA. In JULEA, the critical stress, strain, and displacement due to traffic and material parameters are generated for the designer's selected pavement structure. These critical responses generated in JULEA are used to calculate the incremental damage accumulations on a monthly basis over the entire design life. The incremental damage accumulations are adjusted through the transfer function coefficients, developed through national calibration efforts using the LTPP database to provide the predicted pavement distress in time series as output. In this procedure, it should be noted that even though the pavement responses are generated with a mechanistic approach, an empirical approach (calibrated transfer function coefficient) is still used to provide the predicted distress measures. The LTPP database used to develop the calibrated transfer functions did not include the Iowa locations.

Compared to the current AASHTO guide, another important feature of the MEPDG for flexible pavement is that the new guide does not yield a design thickness as output. The MEPDG predicts performance measures as output for selecting layer thickness and material properties. These predicted performance measures can be assessed to established performance criteria. Thus, the designer must continue to adjust thickness and material input parameters until the predicted performance measures satisfy the established performance criteria.

OBJECTIVE

The primary objective of this study was to evaluate the input parameters related to ACC material properties, traffic, and climate that significantly or insignificantly influence the predicted MEPDG performance model for Iowa's flexible pavement systems. To achieve this objective, the sensitivities of five MEPDG performance measures (longitudinal cracking, alligator cracking, thermal cracking, rutting, fatigue cracking, and smoothness) were conducted by either varying the magnitudes of a single input parameter or by varying the magnitudes of two input parameters in a representative pavement structure. The findings of this study can provide pavement designers a better understanding of the design parameters that should be defined more carefully in the design process and those that most affect certain pavement distresses.

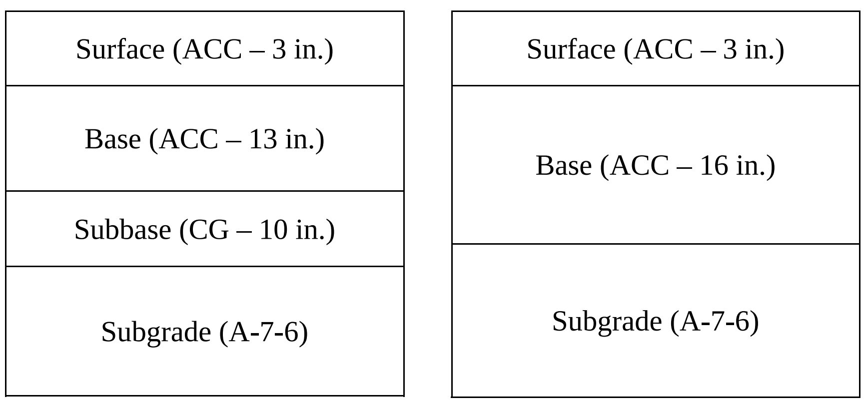
FLEXIBLE PAVEMENT STRUCTURE

Two flexible pavement structures were considered in this study. Though these pavements are ACC overlays of ACC pavements, the pavement structures could be represented as thick ACC-layer pavement structures normally used on interstate and state roads in Iowa. Table 1 summarizes the information for these existing pavements.

Table 1. Summary for two existing flexible pavements

County		Buchanan	Cedar
Route		US 020	I 80
Mile post		266.7–269.24	257.66–265.76
Construction years		Construction: 1971 1st resurfacing: 1989	Construction: 1962 1st resurfacing: 1976 2nd resurfacing: 1990
Surface	**Type**	ACC	ACC
	Thickness	Construction: 3 in. 1st resurfacing: 2 in. (mill: 1.5 in.)	Construction: 4.5 in. 1st resurfacing: 3 in. 2nd resurfacing: 4 in. (mill: 1.5 in.)
Base	**Type**	ACC	ACC
	Thickness	13 in.	16 in.
Subbase	**Type**	Crushed gravel (CG)	N/A
	Thickness	10 in.	N/A
Subgrade	**Type**	A-7-6	A-7-6

Based on the information summarized in Table 1, the two standard flexible structures used in this study were diagrammed, shown in Figure 1.



(a) US-020 in Buchanan County **(b) I-80 in Cedar County**

Figure 1. Standard flexible pavement structures

INPUT DESIGN PARAMETERS FOR FLEXIBLE PAVEMENTS

To investigate the effect of a particular pavement input parameter, the other input parameters are held constant. The design input parameters were divided into two groups: fixed input parameters and varied input parameters. While one design parameter was being examined, a standard value was assigned for the other design parameters. The ranges of magnitude for the varied input parameters were selected based on the recommendations of MEPDG and engineering judgment. Twenty-three key input parameters were selected as varied input parameters for the flexible pavement structure. Table 2 lists the fixed input parameters and standard values for the two flexible pavement structures used in this study.

Table 2. Fixed project input parameters

Fixed input parameter	Standard value
General information	
Design life (years)	20
Base/subgrade construction month	Sept. 2004
Pavement construction month	Sept. 2004
Traffic open month	Oct. 2004
Type of design	Flexible pavement
Site/project identification	
Location	Buchanan/Cedar
Analysis parameter	
Initial IRI (in./mi)	40
Terminal IRI (in./mi)	172 (maximum criteria)
ACC surface down cracking (ft/mi)	1000 (maximum criteria)
ACC bottom up cracking (%)	25 (maximum criteria)
ACC thermal fracture (ft/mi)	1000 (maximum criteria)
Permanent deformation—total pavement (in.)	0.75 (maximum criteria)
Permanent deformation—ACC only (in.)	0.25 (maximum criteria)

Traffic

The traffic input parameters required in the MEPDG are truck traffic volume, truck traffic movement information (speed, lateral wander, and axle load distribution), and truck traffic load character related to tire, axle, and wheel. This is more traffic information than what is required by the current AASHTO guide.

Traffic input parameters were expected to reflect real traffic condition in two locations (US-020 in Buchanan County and I-80 in Cedar County) so that the monthly adjustment factors (non-varied input parameters) in Table 3 and the vehicle class distribution (varied input parameters) in Table 4 could be obtained from the Iowa DOT database (Iowa DOT 2004; 2003). Five cases of vehicle class distribution were investigated for sensitivity analysis of vehicle class distribution. Each vehicle class distribution has a different shape of distribution. Tables 5 and 6 show the fixed traffic input parameters and varied traffic input parameters.

Table 3. The monthly traffic adjustment factors (fixed input parameter)

Month	MAF for Buchanan	MAF for Cedar
Jan.	0.82	0.78
Feb.	0.86	0.79
Mar.	0.90	0.90
Apr.	0.96	0.93
May	1.04	1.03
Jun.	1.06	1.13
Jul.	1.08	1.17
Aug.	1.11	1.23
Sep.	1.07	1.05
Oct.	1.08	1.05
Nov.	1.04	1.00
Dec.	0.99	0.92

Table 4. Vehicle class distributions (varied input parameters)

Veh.	Buchanan					Cedar				
Class	Case1[a]	Case2[b]	Case3[c]	Case4[d]	Case5[e]	Case1[a]	Case2[b]	Case3[c]	Case4[d]	Case5[e]
4	1.5	0.5	5.3	5.2	0.1	1.2	0.5	5.3	5.2	0.1
5	12	3.2	8.5	38.9	0.6	4.1	3.2	8.5	38.9	0.6
6	5.7	2	6.5	35.8	0.8	2.2	2	6.5	35.8	0.8
7	1.1	0.2	4.8	10.2	0.6	0.4	0.2	4.8	10.2	0.6
8	19.7	45	15.6	5.6	6.8	27.5	45	15.6	5.6	6.8
9	53.9	49	38.9	3.5	9.2	57.9	49	38.9	3.5	9.2
10	3.9	0.1	7	0.2	25.8	1.8	0.1	7	0.2	25.8
11	0.8	0	4.5	0.3	36.4	1.7	0	4.5	0.3	36.4
12	0.7	0	4.5	0.2	16.5	1.6	0	4.5	0.2	16.5
13	0.7	0	4.4	0.1	3.2	1.6	0	4.4	0.1	3.2

a: real vehicle distribution; b: high-medium class concentrated vehicle distribution; c: low-medium class concentrated vehicle distribution; d: low class concentrated vehicle distribution; e: high class concentrated vehicle distribution

Table 5.Fixed traffic input parameters

Fixed input parameter	Standard value
Traffic general	
Number of lanes in design direction	2
Percent of trucks in design direction (%)	50
Percent of trucks in design lane (%)	90
Traffic volume adjustment factors	
Hourly truck distribution	Default
Traffic growth factor	4 – composite
Axle load distribution factors	Default
General Traffic inputs	
Mean wheel location (in.)	18
Design lane width (ft)	12
Axle configuration	
Average axle width (ft)	8.5
Dual tire spacing (in)	12
Axle spacing—Tandem, Tridem, Quad axle	51.6 , 49.2 , 49.2
Wheel base	
Average axle spacing (ft)	12 , 15 , 18
Percent of trucks	33 , 33 ,34

Table 6.Varied traffic input parameters

Varied input parameter	Standard value	Investigated value
Traffic General		
Initial two-way AADTT	1168 for Buchanan/ 10928 for Cedar	100,1000,5000,10000,250 00
Operational speed (mph)	60	3,25,45
General traffic input		
Traffic wander standard deviation (in.)	10	7,13
Axle configuration		
Tire pressure—single and dual tire(psi)	120/120	90/90,110/110,130/130,15 0/150

Climate

Climate input parameters for pavement design locations can be generated by choosing climate data from a specific weather station or by interpolating climatic data for a given location. Two new climate files, Buchanan County and Cedar County, were generated to determine standard input values. To investigate the effect of climate, Burlington in southern Iowa and Estherville in northern Iowa were chosen as investigated input values. The climate may influence the asphalt binder property more than the aggregate property, so the study should investigate the interacted sensitivity for different climates and different binder grades. Two types of PG grade binder, PG 58-28 and PG 64 -22, were assigned in different climate areas, summarized in Table 7.

Table 7. Varied climate input parameter

Varied input parameter	Standard value	Investigated value
Climate		
Climate data file	Buchanan file/Cedar file	Burlington (relatively warm area in Iowa) Estherville (relatively cold area in Iowa)
Asphalt material property		
Asphalt binder grade	PG 58 – 28	PG 64 – 22

Material properties

The materials used in this study can be divided into three major groups: ACC material, unbounded material (aggregate), and subgrade material. Most properties of ACC required in the MEPDG were investigated. However, for unbounded material and subgrade material, strength-based properties were investigated with the Integrated Climate Model input analysis. The standard material property values were used to reflect actual field pavement properties in Buchanan and Cedar County. Table 8 shows the varied material input parameters used in this study. Table 9 shows the unbound and subgrade material properties used.

SENSITIVITY RESULTS

The results of the MEPDG software runs for the evaluated input parameters provided numerous charts and tables. Therefore, due to space limitations, it is difficult to discuss each evaluated input parameter and provided distress in this paper. The results of the MEPDG software runs are summarized. The sensitivities of five MEPDG performance measures were conducted by either varying a single input parameter or by varying two input parameters at a time. Each performance measure was graded at three levels (very sensitive, sensitive, insensitive). The trends in performance measure magnitude changes (increasing or decreasing) with respect to input parameter magnitude changes are presented in Table 10. The relative sensitivity of input parameters can be changed with the scale used in analysis, so that the scale in the examined chart should be fixed according to Table 11. Examples of different performance measures are illustrated in Figure 2. A large difference among output trends with respect to input parameter changes indicates an increased sensitivity, while little or no difference indicates insensitivity. It is also noted that the relative sensitivity of one input parameter could be influenced by the other input parameter when varying two parameters at a time.

Table 8. Varied material input parameters used in sensitivity analysis

Varied input parameter	Standard value	Investigated value
Asphalt layer		
Asphalt surface thickness (in.)	3	4,5,6,7,8
Asphalt base thickness (in.)	13 (Buchanan)/16 (Cedar)	5,10
Type of asphalt base material	ACC	CG, A-1-a, A-2-4, A-2-7
Surface ACC aggregate gradation	NMS ¾ in. gradation - Cuml.% retain. ¾ in.: 0 - Cuml.% retain.3/8 in.: 22 - Cuml.% retain.#4: 48 - % passing #200: 3	NMS ½” - Cuml.% retain. ¾ in.: 0 - Cuml.% retain.3/8 in.: 15 - Cuml.% retain.#4: 41 - % passing #200: 4
Base ACC aggregate gradation	NMS ¾ ” gradation - Cuml.% retain. ¾ in.: 0 - Cuml.% retain.3/8 in.: 25 - Cuml.% retain.#4: 56 - % passing #200: 3	NMS ½” - Cuml.% retain. ¾ in.: 0 - Cuml.% retain.3/8 in.: 13 - Cuml.% retain.#4: 42 - % passing #200: 4
Asphalt binder	PG 58 – 28	PG52-22,PG52-28,PG52-34 PG58-22,PG58-34 PG64-22,PG64-28,PG64-34
Initial volumetric properties (Vbe/ Va/ VMA, %)	11/7/18	12/8/20,13/7/20,11/6/17 12/5/17,12/4/16,11/3.5/14
Poisson’s ratio	0.25	0.35,0.45
Thermal conductivity asphalt (BTU/hr-ft –F”)	0.67	0.5, 0.7,1
Heat capacity asphalt (BTU/lb-F”)	0.23	0.1,0.3,0.5
Unbound layer (ν =0.35, K_O = 0.5)		
Subbase thickness (in.)	10	3,6,9,12
Type of subbase material	CG	A-1-a, A-2-4, A-2-7
Subgrade layer (ν =0.35, K_O = 0.5)		
Type of subgrade material	A-7-6	A-1-a,A-2-4,A-5
Thermal cracking		
Aggregate coefficient of thermal extraction (/ºF)	2.8 x 10^-6	10^-7,10^-4

Table 9. Unbound and subgrade materials (varied input parameter)

Classification	CG	A-1-a	A-2-4	A-2-7	A-5	A-7-6
Modulus (Ksi)	42	40	32	24	20	8
PI (%)	1	1	2	15	1	40
% Passing #4	30	20	80	90	90	99
% Passing #200	10	3	20	20	80	90
D60 (mm)	2	8	0.1	0.1	0.05	0.01

Table 10. Notation used in sensitivity study

Notation	Meaning
↑↑ / ↓↓	Very sensitive (↑↑ = more increasing / ↓↓ = more decreasing output trend) on varying two input parameters
↑ / ↓	Sensitive (↑= increasing / ↓= decreasing output trend) on varying two input parameters Sensitive (↑= increasing / ↓= decreasing output trend) on varying one input parameter
↔ (↑) / ↔ (↓)	Insensitive (↔ (↑) = increasing / ↔ (↓) = decreasing output trend) on varying input parameter
↔	Insensitive on varying input parameter

Table 11. Scales in different distress charts for sensitivity study of two flexible pavements

Distress	Minimum	Maximum	Unit	Recommended distress target
L.C (ft/mi)	0	10000	1000	1000
F.C (%)	0	26	2	25
T.C (ft/mi)	0	2000	200	1000
A.C rutting (in.)	0	1	0.2	0.25
T. rutting (in.)	0	1.6	0.2	0.75
I.R.I (in./mi)	0	220	20	172

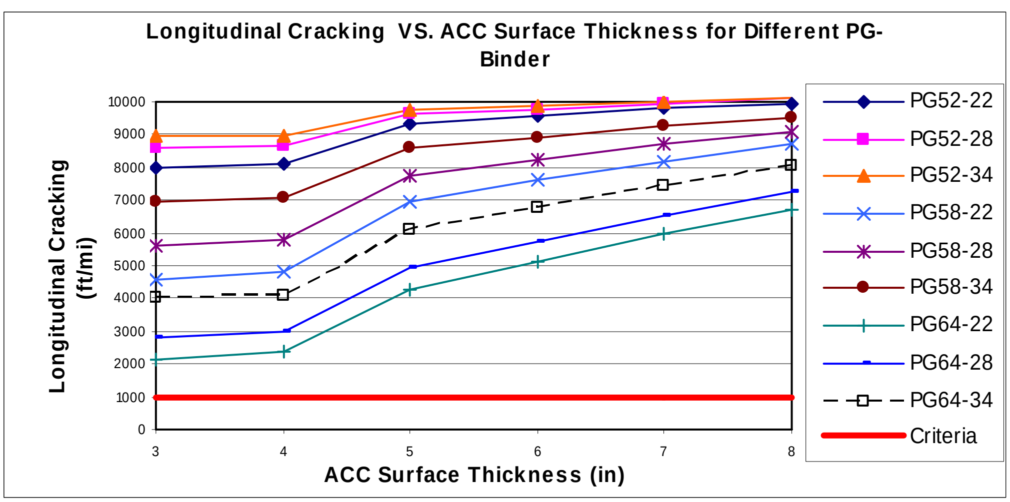


Figure 2. (a) Sensitive and different magnitude output trends for evaluating two input parameters (PG ↑ ➢ Long. crk.↓↓ for thin surface ACC layer and PG ↑➢ Long. crk. ↓for thick surface ACC layer)

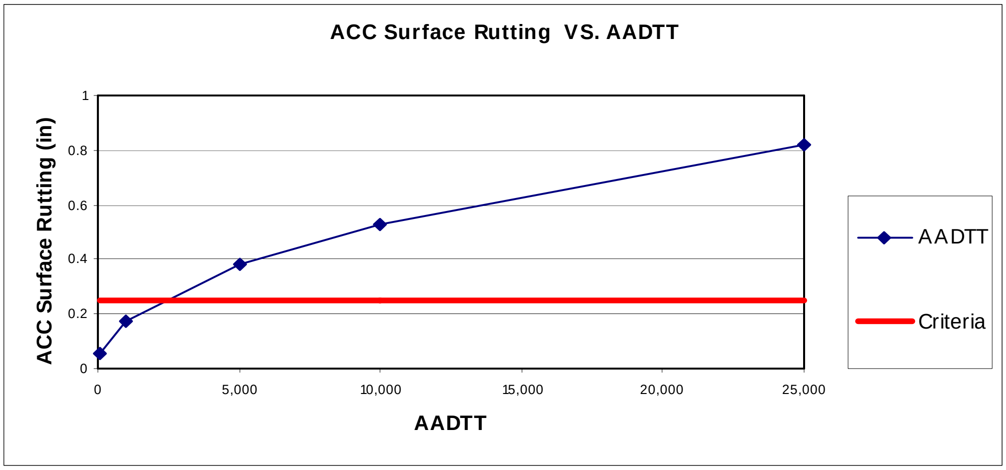


Figure 2. (b) Sensitive and different magnitude output trends for evaluating one input parameter (AADTT ↑ ➤ ACC surface rutting ↑)

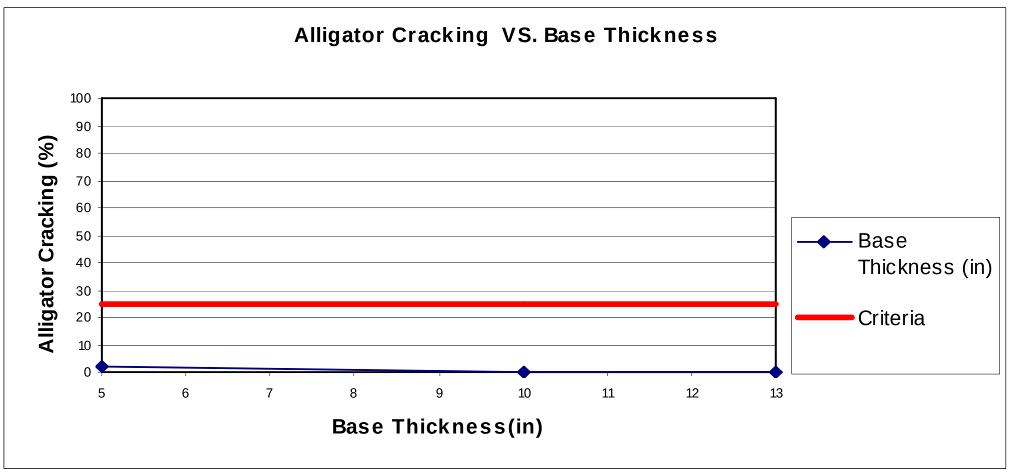


Figure 2. (c) Insensitive and different magnitude output trends for evaluating input parameter (base thickness ↑ ➤ allig. crk. ↔ (↓))

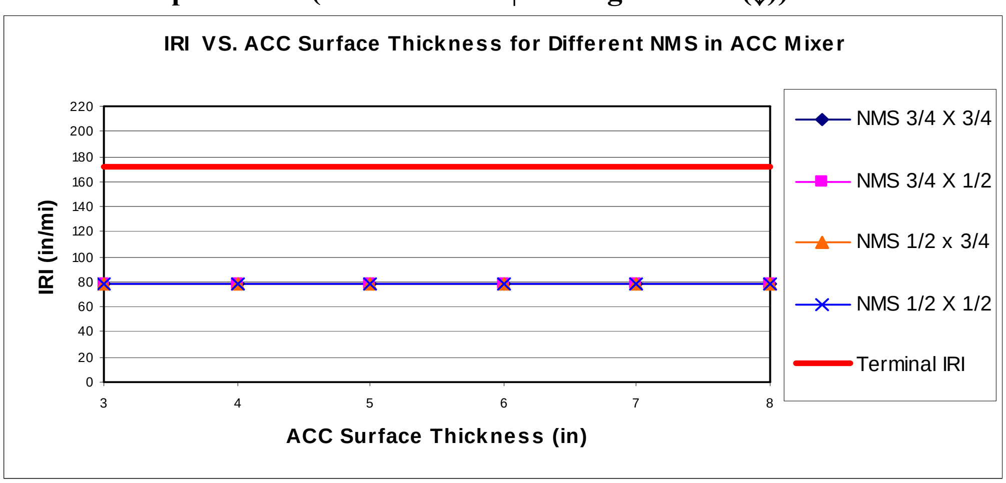


Figure 2. (d) Insensitive and no difference magnitude output trends for evaluating input parameter in legend (ACC surface thickness ↑➤ IRI ↔)

Figure 2. Examples of different sensitive output trends

Twenty-three input parameters were investigated in this study. Table 12 summarizes the results. Interestingly, there is no input parameter that is sensitive to all of the MEPDG performance measures in this study. This indicates that it would be quite difficult to obtain the optimum pavement structure that would be able to resist all distresses.

Most investigated input parameters were sensitive to longitudinal cracking, while most were insensitive to alligator cracking (19 of 23 input parameters were listed as “relatively sensitive” for longitudinal cracking, while 3 of 23 input parameters were listed as “relatively sensitive” for alligator cracking). The reason for this might be related to the relatively thick ACC layer pavement structure evaluated in this study. This is verified in that changing the base type and thickness results in relative sensitivity to alligator cracking. Considering that flexible pavement structures in Iowa have relatively thick ACC layer pavements, a pavement designer in Iowa should be more concerned about longitudinal cracking than alligator cracking. Transverse cracking was more influenced by 5 of 23 input parameters related to material properties and climate. Thus, there is reasonable agreement that transverse cracking might be due to material properties and not to structure. Rutting was found to be a more sensitive distress by most input parameters (14 of 23 input parameters were listed as being quite sensitive for rutting). It is also interesting that the ACC surface rutting was found to be more sensitive to the variance of input parameters. Though 6 of 20 input parameters were listed as sensitive for international roughness index (IRI), IRI was not very sensitive for most input parameters. It may be due to the nature of the IRI model that alligator cracking and transverse cracking contribute more to the IRI value.

The interaction between two parameters, 7 of 23 input parameters, was investigated. Among them, four of seven input parameters were listed as interactive for longitudinal cracking while most input parameters were listed as non-interactive for other performance measures. These results indicate that a pavement designer using the MEPDG for flexible pavement design should recognize the interactive effects among input parameters to obtain the predicted performance measures for satisfying the design criteria.

The differences in trends of performance measures between the two flexible pavements were also investigated. Five of twenty-three input parameters show differences for projected longitudinal cracking measurements. Only 1 of 23 input parameters shows differences in the projected alligator cracking and projected rutting measurements. These differences might be attributed to the differences in AADTT and the component materials in the structures of the two investigated flexible pavements.

Table 12. Summary of sensitivity analysis results for two flexible pavements

Flexible Pavement Input Parameters			Predicted Performance measures								
			Long. Crk. (Top down)	Allig. Crk. (Bottom up)	Therm.Crk. (Transverse)	Rutting					IRI
						ACC Surface	ACC Base	Subbase	Subgrade	Total	
Traffic		**AADTT ↑**	↑	↔(↑)	↔	↑	↑	↔	↑	↑	↔
		Traffic Speed ↑	↓	↔	↔(↓)	↓	↔(↓)	↔	↔(↓)	↓	↔
		Traffic Distribution ↑at thin ACC surface in thick ACC layer	↑/↑↑*	↔	↔	↔(↑)/↑*	↔	↔	↔	↔(↑)/↑*	↔
		Traffic Distribution ↑at thick ACC thick. in thick ACC layer	↑	↔	↔	↔(↑)/↑*	↔	↔	↔	↔(↑)/↑*	↔
		Traffic Wander ↑	↓	↔	↔	↔(↓)	↔	↔	↔	↔(↓)	↔
		Tire Pressure ↑at thin ACC in thick ACC layer	↑/↑↑*	↔	↔	↑	↔	↔	↔	↑	↔
		Tire Pressure ↑at thick ACC in thick ACC layer	↑	↔	↔	↑	↔	↔	↔	↑	↔
Climate		**MAAT ↑ (North → south)**	↑	↔	↓	↑	↔	↔	↔	↑	↓
		ACC E* ↑ (PG ↑) at low MAAT (North)	↓	↔	↑	↓	↔	↔	↔	↓	↑
		ACC E* ↑ (PG ↑) at high MAAT (South)	↓	↔	↑	↓	↔	↔	↔	↓	↑
Material	**A.C Mix. Property**	**NMS ↓ (E*↑) at thin ACC surface in thick A.C layer**	↓	↔	↔	↔(↓)	↔	↔	↔	↔(↓)	↔
		NMS ↓ (E*↑) at thick ACC surface in thick A.C layer	↓↓/↓*	↔	↔	↔(↓)	↔	↔	↔	↔(↓)	↔
		PG ↑ (E*↑) at thin ACC surface in thick A.C layer	↓/↓↓*	↔	↑	↓	↔	↔	↔	↓	↑
		PG ↑ (E*↑) at thick ACC surface in thick ACC layer	↓↓/↓*	↔	↑	↓	↔	↔	↔	↓	↑
		Volumetric (Va ↓ → E*↑)	↓	↔(↓)	↔	↓	↔	↔	↔	↓	↔
		Volumetric (Vbe ↓ → E*↑)	↑	↔(↑)	↓	↓	↔	↔	↔	↓	↓
		Unit Weight ↑	↔(↓)/↓*	↔	↔(↓)	↔(↓)	↔	↔	↔	↔(↓)	↔
		Poisson Ratio ↑	↓	↔	↔	↓	↔	↔	↔	↓	↔

Table 12. Continued

Flexible Pavement Input Parameters			Predicted Performance measures								
						Rutting					
			Long. Crk. (Top down)	Allig. Crk. (Bottom up)	Therm.Crk. (Transverse)	ACC Surface	ACC Base	Subbase	Subgrade	Total	IRI
Material	A.C Thermal Property	Thermal Conductive↑ at thin ACC surface in thick ACC layer	↔(↑)	↔	↔(↓)	↔(↑)	↔	↔	↔	↔(↑)	↔
		Thermal Conductive↑ at thick ACC surface in thick ACC layer	↔(↑)	↔	↔(↓)	↔(↑)	↔	↔	↔	↔(↑)	↔
		Heat Capacity ↑ at thin ACC surface in thick ACC layer	↓	↔	↔(↓)	↓	↔	↔	↔	↓	↔ (↓)
		Heat Capacity ↑ at thick ACC surface in thick A.C layer	↓	↔	↔(↓)	↓	↔	↔	↔	↓	↔ (↓)
	Base	Mr ↑ (Type of Base)	↔(↓)	↓	↔	↔	↓	↔	↔(↓)	↔(↓)	↓
		Mr → E* (Unbound → Bound)	↑	↓	↓	↓	↓	↔(↓)	↔(↓)	↓	↓
	Subbase	Mr ↑ (Type of Subbase)	↔(↑)	↔	↔	↔	↔	↔	↔	↔	↔
	Subgrade	Mr ↑ (Type of Subgrade)	↑	↔(↓)	↔	↔	↔	↔	↓	↓	↔
Layer Thick.	ACC Surface	Thickness ↑	↑	↔(↓)	↔	↔(↑)	↔(↓)	↔(↓)	↔(↓)	↔(↓)	↔
	ACC Base	Thickness ↑	H<10 ↓ H>10 ↑	↔(↓)/↓*	↔	↓	↔(↓)	↔	↔(↓)	↓	↔
	Subbase	Thickness ↑	↑	↔(↓)	↔	↔	↔	↔(↑)	↔(↓)	↔(↓)	↔
Others	Aggr.	Aggregate Thermal Coefficient ↑	↔	↔	↔	↔	↔	↔	↔	↔	↔

* US 20 in Buchanan County/I 20 in Cedar County

CONCLUSIONS AND RECOMMENDATIONS

The relative sensitivity of MEPDG input parameters related to the properties of ACC, traffic, and climate were investigated in two existing Iowa flexible pavement structures. Most of the input parameter variations could be represented in Iowa conditions. Based on the observations of this study, the following conclusions were drawn:

- The MEPDG requires many more input parameters than the current AASHTO guide. These input parameters are connected to each other inside the software and provide predicted performance measures. Thus, increasing layer thickness, which is the general design approach in the current AASHTO guide to reduce the distress, is not the only solution in the MEPDG.
- Few input parameters used in this study affect all the predicted performance measures. However, binder PG grade, volumetric properties, climate, AADTT, and type of base generally influenced most of the predicted performance measures.
- The predicted longitudinal cracking performance measure was influenced by most input parameters. A reasonable design concept to reduce longitudinal cracking should be considered in relatively thick pavement designs.
- Alligator cracking was not a critical distress in the relatively thick pavement structures used in this study.
- The input parameters related to material properties and climate were especially sensitive to the predicted transverse cracking performance measures.
- ACC surface rutting dominated total rutting in the relatively thick pavement structures used in this study.
- IRI was not sensitive for most input parameters. This might be due to the nature of the IRI model. Alligator cracking and thermal cracking are the primary contributors to the IRI value.

To supplement the conclusions made in this sensitivity study, remaining research efforts related to the MEPDG include the following:

- The predicted pavement measures were not validated through the MEPDG against the recorded measurements in the DOT PMIS database in this study. This research approach is strongly recommended for local calibration.
- The input parameters in the MEPDG are interconnected and provide different severities for each performance measure. The optimizing input parameters (especially layer thickness and material parameters), which can be satisfied with the criteria for all of the predicted performance measures, will be useful to pavement designers. This research will provide a valuable guideline to pavement designers using the MEDPG for flexible pavement design.

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Local Adoption of Safety Conscious Planning through Technology Transfer

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ABSTRACT

The Safety Conscious Planning (SCP) model, a comprehensive safety system, had been selected as the statewide network to adopt in New Jersey because it promoted the reduction of crashes that affect the security and congestion of the entire New Jersey transportation infrastructure. The intended benefit of this implementation effort was realized when funding opportunities, resources, and technical support were able to reach county and local municipalities, where over 60% of the roadway fatalities occur annually. Another gain had been the collective empowerment of a partnership being applied to resolving regional safety issues. Also, SCP facilitated the involvement of local elected officials, working together with safety professionals, to organize local safety networks within their own communities.

Key words: local safety agencies—metropolitan planning organizations—safety conscious planning—safety networks—technology transfer

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PROBLEM STATEMENT

After examining the status of congestion, security, and safety issues on the national and state levels, the determination was made to implement a comprehensive safety network that would support the reduction of over 42,000 transportation fatalities that occur on our nation's roadways. Safety is also an integral part of the security and congestion considerations of the transportation network. Therefore, the Safety Conscious Planning (SCP) Model had been selected as the network for New Jersey to adopt. The SCP model also supports the reduction of crashes that impact the security and congestion of local roadways, where a majority of the crashes and fatalities occur in New Jersey.

Nationally, the SCP model has been recognized as an essential part of transportation that needs to be considered by local, regional, and state agencies. There are many ways that safety can be incorporated into the transportation planning process, specifically through long-range and short-range planning procedures. Project needs are identified as improvements of the system infrastructure, while non-project needs are improvement of the operation/management of the system. Furthermore, SCP procedures can be used by other agencies to establish safety as a priority beyond the planning venue, in order to support the "Vital Few Goals" of the FHWA.

RESEARCH OBJECTIVES

As part of the 1998, 2002, and 2004 Federal Certification Reviews, the FHWA's New Jersey Division Office identified the need for a statewide planning process to be in place for congestion mitigation, public safety, and disaster relief. The federal process has promoted stakeholder partnerships for producing maximum results by avoiding the needless replication of efforts. Another important issue for establishing a partnership is that resources and technical support need to reach the county and local levels to affect change. Therefore, this congestion, safety, and security study has been structured as an innovative technology transfer project with several phases that follow the Transportation Research Board recommendations for managing technology transfer. This process included characterizing the audience, characterizing the information, comparing technology transfer methods, applying them effectively, and continually modifying the technology transfer process until successful.

A comprehensive literature review of congestion, security, and safety technology transfer applications was initially conducted to determine the most comprehensive network for adoption in New Jersey. Since safety is an integral part of the security and congestion consideration of the transportation network, the SCP model had been selected as the network for implementation on the state, county, and local levels (see Figure 1).

This objective was accomplished by utilizing the technology transfer application process that included the following:

- Characterization of the group being served (size, homogeneity, affiliation, access to resources)
- Characterization of information (knowledge/skill, complexity)
- Comparison of technology transfer methods (e.g., manuals, implementation packages, workshops, applications)

Later, the innovative technology adoption process of awareness, attitude formation, trial/decision, and confirmation served as a framework for the modification and development of a three-tier system to accommodate state, regional, and local safety needs in New Jersey.

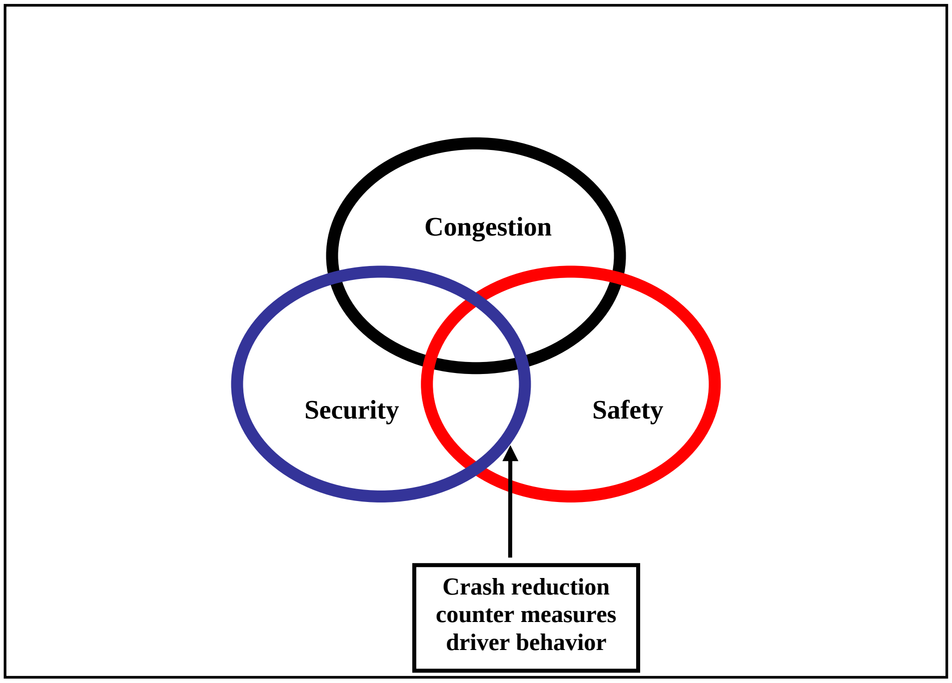

Figure 1. Commonalities between congestion, safety, and security

RESEARCH METHODOLOGY

Initially, characterization of the group being served was employed for the selection of SCP. The needs of the group affected the level of information to be delivered to the audience. Therefore, the SCP model had been examined to determine whether enhancements were required for reaching a diversified statewide audience. In this study, characterization of the group being served had been pre-determined by results from the Federal Certification Reviews. Specifically, the FHWA, New Jersey Division Office, acknowledged the need for a statewide planning process to be in place for congestion mitigation, public safety, and disaster relief. The role of the metropolitan planning organizations (MPO) was defined, by legislation, to address congestion mitigation and environmental planning and safety in New Jersey.

Next, characterization of information required an examination of commonalities between the issues of congestion, safety, and security. Safety appeared in all areas as a component or solution. Reduction of nonrecurring incidents (roadway crashes) was the predominant priority to be addressed, while safety countermeasures (i.e., signage, signals, traffic conditions) were identified as remediation tools. Safety assumed the role of an independent variable, with congestion and security posturing as dependent variables. Therefore, improvement of safety directly benefited the conditions related to congestion and security of the infrastructure.

Lastly, comparison of technology transfer methods identified SCP (Meyer 2004) as a process for integrating safety into established organizations. It required planners and the leadership to examine the presence of safety elements in the vision statement, goals, and performance measures used for accomplishing the goals. Data and safety analysis tools were to be used in conjunction with evaluation criteria. Products had been examined relative to transportation safety, while safety was a priority factor. Therefore, it had been important to identify the functions that were internally based and those that would have merit for external application, especially when adapting the SCP model for local use. All functions involving key stakeholders in the planning process qualified as internal, with a differentiation between organizational and operational system approaches.

Table 1. Internal and external components of SCP

Functions	Application
• Vision statement • Goals (one to two) • Safety related performance measures • Safety related data use in problem identification	Internal-organizational
• Presence of safety analysis tools for impacts • Evaluation criteria assessed merits of strategies that contain safety • Products of the process include some actions that focus on transportation safety • Safety is a priority factor in the prioritization process • Systematic monitoring process exists for collecting data on safety system performance	Internal-operational
• All key safety stakeholders are involved in the planning process	External

Meyer (2004) noted that the intent of the SCP model had been to integrate safety into all aspects of the planning process. This approach was effective, for use on state and county levels, with the planning profession, newly formed organizations or committees, and those in leadership roles who had the power to drive the vision of an organization (e.g., elected officials.) Therefore, the meaning of safety was defined with a new proactive importance in the organization.

However, the typical understanding of safety had been project-based and reactive to crashes that have already occurred, which created a dichotomy for local safety professionals. One solution was to approach safety from both directions so that the elected official assumed the leadership role of effecting internal change within an organization, while the safety professional maintained the responsibility of forming an external system (e.g., committee, task force, network) that served in an advisory capacity for the community, county, region, or state.

The existing SCP model had been modified for the creation of a statewide partnership between the MPOs, the New Jersey Department of Transportation (NJDOT), and other state-level transportation and safety professionals. Tier 1 included the preparation and development of a survey for state, county, and local transportation agencies to obtain feedback on the importance of transportation safety in their organizations. Also, a Safety Conscious Planning Working Group (SCPWG) was formed and met several times to identify potential partners, plan forum activities, and invite participants to a statewide event and offer input to three MPOs and the NJDOT. A series of safety technical resources were developed and distributed to transportation organizations during several workshops held in conjunction with the project.

As part of Tier 2, regional (county-level) forums were developed for elected officials and safety professionals that served a purpose similar to the abovementioned statewide event. County and local public sector representatives had been invited to provide input to the MPO representatives on their safety needs. The purpose of Tier 2 was to assess the existing level of safety within local agencies, target local elected officials and internally drive their organization's commitment to safety, and enable safety practitioners to participate in the development of an external partnership network.

Tier 3 had been accomplished through outreach services to local elected officials. Their role was to drive safety planning internally with the support of planners, while safety professionals (police officers) organized the local community network. An extensive description of the adoption process is provided in the following section.

Table 2. New Jersey SCP (external) partnership network model

Level	Activities
Tier 1: Statewide SCP Network	• MPO and NJDOT Partnership Network. • Development of educational activities
Tier 2: Regional Forum and Safety Task Force	• Identify existing local ad hoc and enforcement groups of the network • Educate representatives on the role of MPOs • Create partnership between four county representatives and the Delaware Valley Regional Planning Commission • Provide updates on safety issues to the Regional Safety Task Force representatives that communicate the information to local agency representatives • Expand services provided to locals and counties through newsletter, training, and resource distribution
Tier 3: outreach to local officials	• Adoption of SCP within the local organization • Creation of SCP local systems

Initially, a survey was prepared and distributed to state, county, and local transportation agencies that provided baseline feedback on the importance of roadway safety to their organizations. Next, the SCPWG met several times to identify potential partners, plan forum activities, and invite participants to a statewide event in order to provide feedback to three MPOs and the NJDOT. Several safety publications, reports, newsletters, handouts, and checklists were developed and distributed to transportation organizations during several workshops held in conjunction with the project.

Tier 1: Establishment of the New Jersey Statewide SCP Network

Representatives from the New Jersey Division Office of the Federal Highway Administration, the NJDOT, the three MPOs, and New Jersey CAIT-LTAP met to develop the New Jersey version from the sample survey used during the Michigan Safety Conscious Planning Forum (FHWA 2003). After an intense review, the survey instrument was mailed and made available online to county and municipal planning, engineering, public works, and law enforcement departments.

Over 95% of the 305 respondents were from municipal organizations, with 62% being answered by police traffic officers. Many individuals represented predominantly suburban municipalities. Their responses served to guide the regional MPOs to include traffic safety issues in future regional plans.

Participants were asked to rank order (1= lowest and 10= highest) a listing of their current top interests and future safety concerns. The results confirmed that reducing fatalities (7.7), impaired drivers (7.4), and aggressive driving (7.2) were the primary safety interests. Future safety concerns also yielded the same results from the respondents, who are predominantly police traffic officers. Interestingly, the engineering-related issues (consequences of leaving the road, head on collisions, and keeping vehicles on the roadway) received the lowest ranking scores. Also, truck travel did not appear to be an immediate or future concern on local roadways.

Sixty-one percent of the respondents mentioned that their agencies did have programs that addressed safety issues. Also, more than one type of safety program was sponsored per organization, with the most popular being safety belt checks and enforcement efforts that target impaired drivers on local roadways. Again, very few organizations sponsored engineering-related safety programs.

Table 3. Percentages of organizations with current local safety programs

Type	Yes	Unknown	Type	Yes	Unknown
Aggressive driving	13	87	Improve design intersection	3	97
Commercial drivers	2	98	Intersection safety	5	95
Fatigued drivers	2	98	Vehicle on roadway	1	99
				1	99
Fatalities	2	98	Consequences of leaving road		
Impaired drivers	25	75	Head-on and across -med. cr.	1	99
Influencing driver	6	94		15	85
behavior			Ped/bike and motorcycles		
Older/Young drivers	4	96	Safety belts	27	73
Construction of safety	3	97			
projects			Truck travel	2	98
Designing safer work	10	90			
zones			Other	23	77

Many agencies were involved in local safety initiatives. Fifty-six percent of the respondents were collecting data for use in addressing safety issues, but only 39% were actually conducting data analysis as part of their safety procedures. The second ranking responses for addressing safety concerns were through education of personnel. The bottom-ranked initiatives were development of a safety plan (21%) and conducting research on safety (16%).

Only 27% of the respondents indicated that their agencies participated in a long-range safety planning process, which is slightly higher (6%) than what was previously indicated about how their agencies address safety concerns. Significantly few respondents (10%) stated that they had been involved in the development of the state or MPO long-range transportation plan. This finding was not surprising, because most local agencies had not been directly involved in the state or MPO planning process.

Municipal police traffic officers worked directly with roadway safety issues as part of their daily routine and, therefore, served as a great resource for the MPOs and the NJDOT. It appeared that engineering issues were not of interest to the survey respondents, but that may have been due to the limited involvement of police traffic officers with data analysis and engineering technology. Also, the New Jersey Safety Conscious Planning Initiative (NJ SCPI) needed to include municipal police traffic officers as partners in the process because they were intimately familiar with understanding the events that lead to fatalities on local New Jersey roadways. They were the source point of data collection.

Statewide SCP Partnership

Several organizations were targeted to provide leadership for the NJ SCPI. With support from national consultants, the New Jersey Safety Conscious Planning Forum was held on May 2004 with the anticipation of increasing statewide safety awareness; developing an understanding of the planning process; establishing strategies for quantifying the safety component; providing an inventory of financial, technical, and human resources; providing best practices for assisting MPOs and other agencies in establishing technical advisory committees; and obtaining feedback from discussion groups on the four Es: engineering, education, enforcement, and encouragement.

Several goals and action items required resources and funding to accomplish them. Since the Safety Management Task Force, under the direction of the NJDOT Division of Traffic Engineering and Safety, was creating a comprehensive data-driven plan for all statewide safety agencies, the decision has been made to have safety goals addressed at the state level, where an opportunity exists to gain support and recognition. In December 2004, the list was formally presented to the Safety Management Task Force for further consideration and adoption, if applicable. Program participants also received a copy of the Forum Final Report as guidance on potential safety projects for consideration by their agencies.

MPO Safety Status Review

The three MPO organizations representing New Jersey exhibited varying degrees of internal and external integration of safety consciousness within their operations. First, the size of the North Jersey Transportation Planning Authority (NJTPA), one of the largest metropolitan planning organizations in the country, had prompted this organization to seek national guidance on the integration of safety into their system, while contracting a consultant to analyze their corresponding crash data and then releasing identified safety projects to the appropriate agencies for further action. Next, the South Jersey Transportation Planning Organization (SJTPO) Traffic Safety Alliance operated as a fully integrated transportation safety agency that offers technical support and services to the local municipalities of the counties that they serve. Lastly, the Delaware Valley Regional Planning Commission (DVRPC) did not have a formal safety network in the state, but had been very much involved with local outreach on projects that included congestion mitigation, corridor planning, and incident management.

Consequently, the DVRPC representatives partnered with New Jersey CAIT-LTAP to establish a local safety conscious planning network in their region. Since the SJTPO Traffic Safety Alliance had already integrated SCP into a formalized network, their representatives also provided input to the DVRPC on creating a similar network. An agreement was then reached to showcase Traffic Safety Alliance activities during the DVRPC-sponsored regional forums and during internal meetings with the staff members at the agency. Lastly, the NJTPA had deferred scheduling countywide forums until after the selection of 25 safety projects that would be completed in early 2005.

Tier 2: DVRPC Countywide Forums and Regional Safety Task Force Plans

Tier 2 of the SCP implementation plan focused on sponsoring county forums to gain local input on organizing a Regional Safety Task Force. The DVRPC held a series of regional safety forums for Burlington, Mercer, Gloucester, and Camden counties. Forum goals differed from the statewide program because the intent was to promote awareness and showcase the Traffic Safety Alliance as a local best practice. Over 40 participants were in attendance at each forum. Municipal police and MPO representatives yielded the highest participation level, which was followed by county employees and university members.

The feedback, obtained from the forums, identified existing safety partners for inclusion in the DVRPC Transportation Safety Task Force Plan. A comparative analysis of each county was then conducted to further determine the agency and type of safety services present in the counties. Also, Meyer’s guidebook was consulted to identify further potential partners for the task force.

County Profiles

The county profiles revealed that comprehensive traffic safety programs (CTSPs) existed in all of the counties under the jurisdiction of DVRPC, except for Mercer County. Some organizations were even fortunate enough to reap the benefits of having two safety associations located in their communities (e.g., Camden County). Several agencies were involved in providing local support in the form of coordinating safety campaigns, checkpoints, hot spot identification, and other safety activities.

Camden County was the most heavily populated region, while Gloucester County remained predominantly rural with only one quarter of the land being developed. The transportation network for this entire area was impacted by Philadelphia traffic and Wilmington, Delaware. Intermodal transportation issues had been important to all counties; however, Mercer County addressed air traffic and Camden County was involved with the ferry system as additional means of transportation for residents.

A total of 114 municipalities existed in this four-county area. This large number had limited the ability of the DVRPC to create a partnership involving local transportation agencies. Alternatively, the focus was redirected toward the creation of a county-based regional safety task force that would be better suited for working productively with the DVRPC in adopting SCP as a comprehensive, data driven, and collaborative safety network. Before that could happen, a county-wide safety system had to be organized in Mercer County, which included local transportation agencies.

Tier 3: Outreach to Local Elected Officials

The final stage of implementation, Tier 3, targeted local elected officials. In New Jersey, municipal safety professionals were usually the police traffic officers who had predominantly attended these SCP forums and were educated about the importance of establishing proactive approaches to local safety issues. Locally, they had been responsible for completion of all NJTR-1 crash reports, but these safety specialists did not have the power to affect visions and goals of the organization that drove the emphasis on safety in their represented communities. Frequently, the new information learned at seminars had stayed in the department and was not addressed at the next highest level for further action.

As a solution, the forum proceedings, Meyer’s checklist, and a website for obtaining a downloadable copy of the local SCP kit was sent to each mayor and their local safety professional. Also, a cover letter addressed the need for these officials to support the reduction of roadway crashes on local roads. Local professionals had the opportunity to gain greater internal support as proactive safety leaders, while also becoming connected to the regional network where resources, support, and guidance were obtained. Lastly, elected officials had become informed of the SCP concept and the use of assessment tools (SCP checklist) to integrate safety internally and reach out to their community for support in the development of a community-based safety network.

The final stage of adopting SCP was through the innovation adoption process, including the continuation of the MPO network and the development of a curriculum for county personnel (see Table 4.) The first stage of awareness had been discussed as part of the statewide and county forums. The next stage, attitude formation, involved the user becoming proactive in seeking additional information and forming attitudes shared with fellow network members. At this level, training became available to reinforce the attitudes. A specialized SCP professional was being developed and piloted for county employees to work directly with the municipalities on safety programs. Lastly, the confirmation stage enabled SCP to be measured and linked to the reduction of crashes and fatalities for the represented communities.

Table 4. Technology transfer innovation adoption process

Stages	Actions
Awareness	• Series of forums to complete the regional network
Attitude Formation	• Use of municipal status profiles to document regional needs for local adoption of SCP • Participation of county-based network for the establishment of the MPO task force
Trial/Decision	• Offering technical support for local adoption of SCP
Confirmation	• Evaluation of improved transportation systems (e.g., additional funded projects, lower transportation crashes) as a result of the safety network

KEY FINDINGS

Following the procedures for implementing a SCP network, an interview was conducted with the Mercer County Engineering Department to assess their existing safety needs based on the SCP checklist. This interview revealed important information about the level of involvement that Mercer County has with the NJDOT and the DVRPC. The planning director served as a voting member of the DVRPC committees, so their organization had a strongly established relationship within the region. The DVRPC had provided traffic counts for approximately 50 locations in their jurisdiction. Additionally, NJDOT has proactively approved three projects for funding along with providing technical support for one high-crash location.

Furthermore, the county worked closely with the local police in helping to solve problems that occur on their roadways. Municipal assistance was provided on projects of intercommunity travel (i.e., milling) when one agency paid for the materials and the other performed the milling process required on a resurfacing project. All bridge inventories were part of the county jurisdiction. Cost-benefit analysis had been used for prioritizing projects, instead of spending large amounts funding studies. A yearly meeting was held to make the determination on which projects will be funded in the future.

When developing local partnerships, it was important to gain a better understanding of the current safety needs of the organization, in order to establish an indicator of the readiness for adoption of SCP (see Table 5). Since municipal police have been responsible for traffic safety, the presence of a traffic unit, traffic officer, and/or police department served as a general indicator of the agency's commitment to roadway safety. Mercer County was fortunate, in that each municipality did have a dedicated traffic unit or officer, with the exception of one agency that used state police to patrol local roadways. Also, six local citizens' safety committees had already been established within the county, while two mayors were serving on task forces to help reduce local truck traffic in their communities.

A final comparison of the Mercer County local network to the entire region was conducted to determine potential safety readiness tendencies between counties and within the municipalities (see Table 5). This comparison revealed that Mercer County municipalities maintained more police traffic units than elsewhere and that a greater number of local safety committees had been established there as well. Further research needed to be done to determine whether or not community wealth influences the importance of community safety.

Table 5. Comparison of safety agencies between counties

Safety status	Burlington	Camden	Gloucester	Mercer
Police departments	26	28	23	3
Police departments with traffic units	6	2	1	8
No police/other police patrol	2	2	0	2
Unknown	6	5	0	0
Community and internal safety groups	5	6	5	9
Sponsored professional safety organizations	2*	2	2	1

* Regional forum results

During the Mercer County Police Traffic Officers Association meetings, the abovementioned data was reviewed, along with a commitment to sponsor the SCP network in their county. The Mercer County traffic engineer was sanctioned as the liaison on the DVRP Regional Task Force. Additional information was presented on reducing transportation crashes and fatalities through sponsoring best practices, obtaining technical support for municipalities through this alliance, and implementing appropriate community-based activities. The police membership committed to the following:

- Volunteering as a safety network where county and local agencies may obtain additional support of funding and resources
- Educating local officials in driving safety through policy and planning
- Supporting the distribution of tools for adopting local safety network in their communities

CONCLUSIONS

It is important to maintain SCP as the trademark for this comprehensive, data driven, and collaborative effort at all levels of government, because it connotes a new way of thinking, instead of the safety-business–as-usual. Fortunately, the SCP model had provided the local elected officials with the leadership opportunity to champion transportation safety causes on the administrative level, while at the same time leaving enough flexibility in the model to support designated safety professionals, thus producing a comprehensive safety system that reaches local public sector agencies in New Jersey. The most important outcome of this project was to direct additional safety resources to local communities, where transportation crashes and fatalities predominantly occur, in a systemized and effective order.

ACKNOWLEDGMENTS

During the past several years, Lawrence Cullari and Patricia Leech (FHWA, NJ Division) and Dr. Nick Vitillo (NJDOT, Research) have recognized and equally promoted technology transfer services of the New Jersey CAIT-LTAP throughout New Jersey. More recently, Dr. Ali Maher, director of CAIT, and Joseph Orth, director of CAIT-LTAP, have received support from Patricia Ott, director of the NJDOT Division of Traffic Engineering to develop the New Jersey Transportation Safety Resource Center being developed to support local transportation safety efforts throughout the state. The LTAP staff members are also recognized for their work on the project and coordination of the many meetings and forums that were held in conjunction with this project.

Rosemarie Anderson, DVRPC, was instrumental in sponsoring the countywide forums and the local pilot project in her region. Gregory Sandusky and George Filak, Mercer County Engineering Department, are acknowledged for their efforts in supporting the development of a safety network in Mercer County.

The SCP Committee members are recognized for their tireless efforts in promoting SCP within their own organizations and throughout the state. These individuals have been responsible for providing input on the sponsorship of the statewide forum, development of county forums, and distribution of safety resources for program participants.

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Design Methodology for the Modified Beam-in-Slab Bridge System

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ABSTRACT

Researchers at the Iowa State University (ISU) Bridge Engineering Center have developed the modified beam-in-slab bridge (MBISB) system as an alternative replacement for use on low-volume roads. The system consists of longitudinal steel girders with a concrete arched deck cast between the girders. Composite action between the concrete and steel is obtained by using an alternative shear connector, also developed at ISU. Other than the nominal transverse reinforcement required for the ASC, the MBISB requires only minimal additional reinforcement.

After an extensive laboratory testing phase, two demonstration bridges were constructed and field tested to determine the properties of the design. The demonstration bridges, MBISB 1 (L=50 ft, W=31 ft) and MBISB 2 (L=70 ft, W=32 ft), were constructed by in-house forces using standard construction equipment. The resulting structures saved the bridge owner slightly more than 20% over the costs of conventional designs. Test results indicated that the MBISB design exceeded strength and serviceability requirements, and the experimental lateral load distribution factors were comparable to AASHTO LRFD values.

The experimental data were corroborated through analytical modeling, leading to the development of a design methodology for MBISBs ranging in length from 40 ft to 80 ft. An HS-20 truck was used as the design vehicle and all applicable AASHTO LRFD design criteria for steel girder/concrete deck bridges were satisfied. Design tables based on the desired bridge length, width, material strengths, and deck thickness were developed, along with design aids that include a sample design and standard plan sheets. The design tables are included in Volume 2 of the final report for Iowa DOT Project TR-467.

Key words: beam-in-slab bridge— composite action—low-volume road bridge—replacement alternative

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PROBLEM STATEMENT

In Iowa, county governments are charged with maintaining and replacing off-system bridge structures, a majority of which are found on low-volume roads (LVRs). Approximately 30% of the more than 19,700 bridge structures located on Iowa's off-system roads are either structurally deficient or functionally obsolete (FHWA 2004). Due to limited resources and the costs associated with maintaining an aging and deteriorating bridge population, county engineers have expressed an interest in innovative methods to extend available replacement funds.

Many counties in Iowa employ full-time bridge crews to maintain and repair deficient structures. Due to the advanced levels of deterioration and functional obsolescence, replacement of the deficient structure is often the most cost-effective solution. The MBISB system is an alternative replacement design developed by the Iowa State University Bridge Engineering Center (BEC) to address the replacement of LVR bridges with spans of 40 ft to 80 ft.

RESEARCH OBJECTIVE

The objective of the MBISB research was to develop an alternative, low-cost design for use on LVRs when replacing deficient bridges with spans in excess of 40 ft. The resulting MBISB design is constructible by in-house forces, requires no specialized equipment, and reduces costs by approximately 20% when compared with conventional systems.

DEVELOPMENT OF THE MBISB SYSTEM

The development of the MBISB system grew from the beam-in-slab bridge (BISB) system, a successful alternative design limited to spans of approximately 50 ft due to prohibitive stresses and deflections resulting from the bridge's own weight. The original BISB design consists of simply supported W12x79 girders spaced 24 in. on center. Steel confining straps are welded to the bottom flanges at the longitudinal quarter points to provide confinement during the placement of the concrete. A stay-in-place plywood formwork floor rests on the top of the bottom flanges, leaving space between the web and the formwork to allow for the concrete to be in contact with the bottom flange. The void between the girders is filled with unreinforced concrete, which is struck off even with the top flange; a typical cross-section of a BISB is presented in Figure 1.

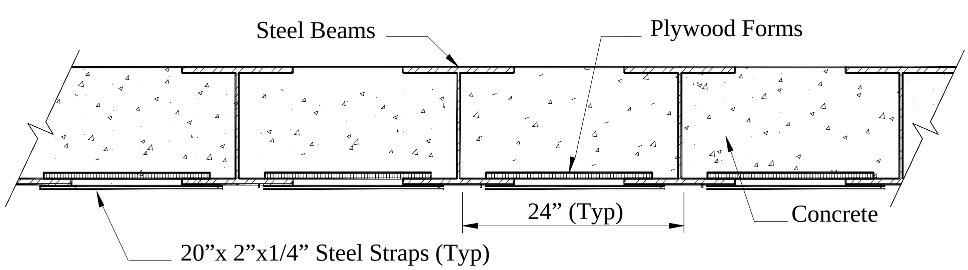

Figure 1. Typical cross-section of a BISB

To span longer distances, the self weight of the original BISB needed to be reduced and the structural efficiency of the system needed improvement. Two modifications were incorporated into the MBISB to achieve the desired effects: an alternative shear connector (ASC) to create composite action between the concrete and steel and a transverse arch to remove ineffective concrete from the cross-section.

Alternative Shear Connector

The ASC, a mechanism for developing composite action, consists of 1 1/4-in.–diameter holes that are either torched or cored in the web of the longitudinal W sections on 3-in. centers, 1 1/2 diameters below the bottom surface of the top flange for the length of the member. Reinforcement (#4 or #5, f_y = 60 ksi) is installed transversely through every fifth hole to provide confinement of the concrete shear dowels, which are formed in the holes when the deck concrete is placed. A series of static and cyclic push-out and flexural beam tests were performed in the laboratory to validate the ASC design; results indicated strength and fatigue properties that meet or exceed the performance of traditional shear studs (Klaiber et al. 1997; 2000).

Transverse Arch

As noted, the self weight of the cross section needed to be reduced so longer spans could be obtained. Recognizing that the concrete below the neutral axis is structurally ineffective and that its removal has minimal effect on the flexural rigidity of the system, a transverse arch cross-section was developed to remove a majority of the ineffective concrete. Two formwork designs, a removable/reusable system and a stay-in-place system, were developed with the arched formwork resting upon the bottom flanges of the longitudinal girders, similar to the original BISB system.

Due to the transverse arch, the mode of resistance of the deck was expected to change from flexure to punching shear, provided the arch was adequately confined. Previous research has indicated that when a bridge deck is adequately confined, the applied loads are resisted by internal arching action, which significantly reduces the amount of internal reinforcement required (Mufti et al. 1993).

RESEARCH METHODOLOGY

Laboratory Phase

Single Bay Specimens

A series of laboratory tests were performed to evaluate the applicability of the two modifications in a full-scale bridge. Continuing the preliminary research performed by Klaiber et al. (2000), two 14.5-ft–long single bay specimens (Specimens 1 and 2) were constructed; a typical cross-section is shown in Figure 2. The purpose of the single bay specimens was to determine the capacity and failure mode of the transverse arch. Each specimen was subjected to a single simulated wheel load, resulting in splitting/punching shear failures of the arched deck at loads 3.5 and 5.8 times greater than a factored 45-kip HS-20 wheel load (Konda 2004). A photograph of the typical testing set up as well as the geometric configuration of the single bay specimens is shown in Figure 3.

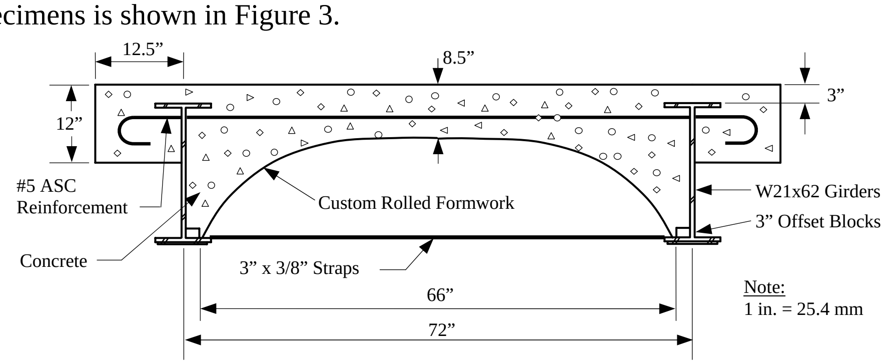

Figure 2. Typical cross-section of a single bay arched specimen

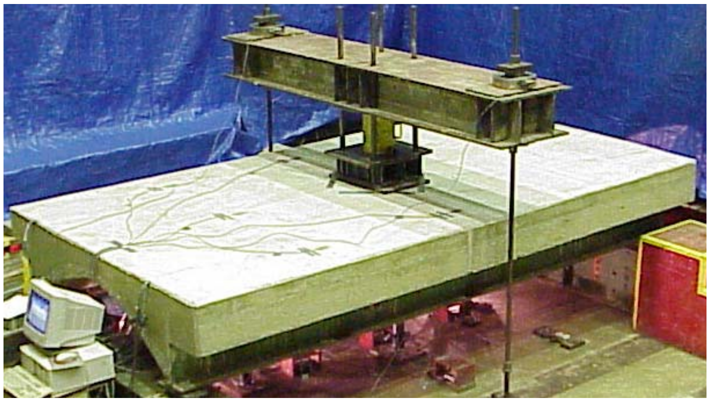

Figure 3. Geometric and loading configuration of Specimen 1

Model Bridge

The single bay specimens provided information on the ultimate capacity of the transverse arched deck, but did not describe the structural behavior of the modified system when integrated into a full-scale bridge. A three-bay model bridge was constructed to determine the lateral load distribution characteristics, flexural capacity, and the deck punching shear capacity. The 30.5-ft–long by 20-ft–wide model bridge was constructed using four W21x62 girders spaced on 72in. centers; the resulting cross-section is shown in Figure 4.

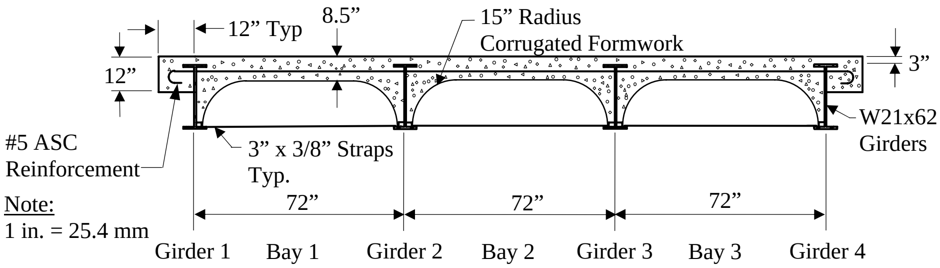

Figure 4. Cross-section of the model bridge

Due to the arching action developed within the deck, the reinforcement was reduced to that required to complete the ASC and stiffen the free ends of the bridge. Crack control reinforcement (a transverse layer of #3, f_y = 60 ksi reinforcement on 15-in. centers) was placed in a portion of the deck to determine whether it was necessary to prevent spalling or excessive cracking. A custom-rolled formwork system, which removed approximately 52% of the concrete from the cross-section, was used to form the transverse arched deck. After adequate curing of the concrete, the formwork was removed and cleaned for future use. The formwork and reinforcement used in the model bridge are shown in Figure 5.

Figure 5. Installed modifications prior to the deck placement

After adequate curing, the model bridge was subjected to both service and then ultimate loadings. The load response was quantified by strain and deflection instrumentation installed at the quarter, middle, and three-quarter span locations on the girders and the concrete deck. Due to equipment constraints, all loads were applied through 12-in. x 12-in. load pads, simulating wheel loads. A total of fifteen 45-kip service-level loads were applied at the quarter, middle, and three-quarter span locations to determine the lateral load distribution characteristics of the bridge. After completion of the service-level tests, an ultimate flexural test was completed, which was followed by five deck punching shear tests.

Lateral load distribution factors were determined by calculating the percentage of the total applied service-level moment that was carried by each effective section; the experimental distribution factors were determined based on the recorded girder strains at the midspan of the model. Since the cross-section of the model was similar to a slab/girder bridge, AASHTO LRFD bridge specification lateral load distribution design values for a single-lane loading were compared to the experimental results (AASHTO 1994). For the interior girders, the maximum experimental value of 45% matched the AASHTO design value. The experimental value for the exterior girder was slightly larger (56% vs. 54%). Due to the severity of the single-point load placed directly over the exterior girder, a conservative experimental value was not expected. Based on the comparative results, it was decided that AASHTO LRFD lateral load distribution values for a slab/girder bridge are applicable for the design of similar MBISBs.

The ultimate flexural loading mentioned earlier was applied about the midspan of the model via a four-point loading configuration. Three of the four girders yielded under a total loading of 302 kips, causing a maximum midspan deflection of 4.01 in. The eccentric positioning of the load pads caused a torsional effect, resulting in cracking of the deck, backwall, and both overhangs. In spite of the cracking and large displacements, the deck gave no indication of an impending catastrophic failure; thus, internal arching action was maintained within the deck slab while the longitudinal members experienced a ductile failure.

Five punching shear tests were completed on the distressed deck, three in the interior of the specimen and two near the free end, to determine the localized capacity and failure mode of the distressed deck. Load was applied through 12-in. x 12-in. load pads for all the tests, with punching shear failure occurring in all cases. The failure loads ranged from 117 kips to 158 kips, which are at least 2.6 times greater than a factored HS-20 wheel load (AASHTO 1994). The mode of failure and magnitude of load necessary to cause failure indicated that there was adequate lateral confinement for internal arching to develop, which resists the applied simulated wheel loading.

Demonstration Bridges

Results from the single bay specimens and the model bridge testing indicate the modifications have more than adequate strength to resist service-level loads. Two demonstration bridges utilizing the previously described modifications were designed, constructed, and field tested to further quantify the structural behavior of the MBISB system and verify the design process. The design and construction of the demonstration bridges is described in the following sections.

Construction of Demonstration Bridges

MBISB 1

The first demonstration bridge, MBISB 1, (L = 50 ft, W = 31 ft) followed closely the original BISB design and consisted of 16 W12x79, f_y = 50 ksi steel sections spaced on 24-in. centers. Holes for the ASC were torched in the webs of the sections, as previously described. Reinforcement (#4, f_y = 60 ksi) was placed through every fifth ASC hole to confine the concrete shear dowels. The transverse arch was formed by a stay-in-place formwork system fabricated from a section of 24-in. diameter, 16-gage corrugated metal pipe (CMP); the transverse arch reduced the self weight of the cross-section by approximately 20%. As in the original BISB system, the concrete deck was placed and struck off evenly with the top flanges.

MBISB 2

The second demonstration bridge, MBISB 2, (L = 70 ft, W = 32 ft), was designed using the AASHTO LRFD bridge specifications and has the outward appearance of a typical slab/girder bridge with six W27x129, f_y = 50 ksi steel girders (AASHTO 1994). However, by including the two modifications, the structure was readily constructed by in-house county forces without the need for specialized equipment. Prior to arriving at the construction site, the girders were cambered and the ASC holes cored in a fabrication shop. After the girders were placed, they were laterally aligned with threaded transverse tension rods, which were attached to the bottom flanges with friction clips. Two lines of diaphragms were installed to provide compression flange bracing, and 12-in.–thick reinforced concrete backwalls were placed at each end of the structure.

The transverse arches between the longitudinal girders were formed using a recoverable/reusable custom-rolled formwork system that was developed during the laboratory phase. The custom-rolled formwork consists of two individual pieces constructed from the same material as CMP, which were bolted together to form a single 24-in.–wide (nominal) section. The individual sections were bolted together into groups of four and five sections off-site and then installed in the bridge as shown in Figure 6.

When compared to traditional Iowa DOT bridge decks, the reinforcement required in MBISB 2 was reduced by approximately 70%. The deck reinforcement was limited to the transverse ASC reinforcement (#5's on 15-in. centers, f_y = 60 ksi) and temperature and shrinkage reinforcement (#4's on 12-in. centers, longitudinal, and #3's on 15-in. centers, transverse). An overhang (12 in. x 12 in.) was necessary on each edge of the bridge to develop the ASC reinforcement; the overhang formwork was also used to set the final deck elevation. Contrary to the BISB design and MBISB 1, the longitudinal girders were embedded with a minimum of 3 in. of concrete placed over the top flanges.

Figure 6. Installed custom-rolled formwork in MBISB 2

Construction Costs

MBISB 1 and MBISB 2 cost approximately \$50/ft^2 and \$52/ft^2 to construct, respectively, including the costs of the sub- and superstructure, labor, and equipment. When compared to conventional designs for these sites, the county realized a cost savings of approximately 20%.

Field Testing of the Demonstration Bridges

Both demonstration bridges were field tested to determine the lateral load distribution, service level stresses, and overall flexural stiffness when subjected to service-level truck loadings. The trucks, weighting between 50 kips and 56 kips, traveled across the bridge at approximately 2 mph (i.e., quasi-static tests) in the five lanes indicated in Figure 7 to create maximum effects in the interior and exterior girders. Prior to the quasi-static tests, static load tests were performed, in which two trucks were placed on the bridge simultaneously.

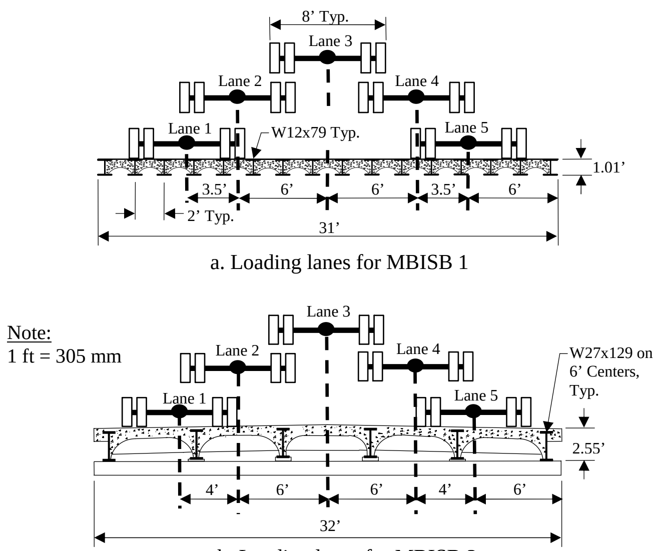

Figure 7. Layout of the loading lanes for field testing

Structural response was measured by strain and deflection instrumentation installed at the quarter and midspan locations to develop response profiles at the two sections. Additional strain instrumentation was installed near the abutments to determine the level of restraint present in the connections between the abutments and the girders.

Field Test Results

MBISB 1

A maximum midspan deflection of 0.73 in. was recorded during the static load test when the two trucks occupied Lanes 2 and 4 (see Figure 7). The maximum recorded deflection was slightly less than the suggested AASHTO service-level live load deflection limit (i.e., L/800) which is 0.74 in. A maximum midspan tensile strain of 154 microstrain (4.5 ksi, assuming Young's Modulus = 29,000 ksi) occurred when two trucks (in Lanes 2 and 4) occupied the bridge. When combining the measured live load and self weight stresses, a maximum tensile stress of 11.5 ksi resulted, a value which is less than 1/4 of the steel yield stress. Based on the strains measured close to the abutments, there was minimal end restraint present (i.e., the bridge is simply supported). Lateral load distribution factors for MBISB 1 were determined by applying the techniques similar to those used in the laboratory model bridge. Based on the experimental results, the lateral load moment distribution design factors for a single lane loading were determined to be 12% for both the interior and exterior girders. This value was not directly compared to the AASHTO LRFD design values because the cross-section of MBISB 1 does not correlate with the presented design information (AASHTO 1994).

MBISB 2

A maximum midspan deflection of 0.5 in. was recorded for both the interior and exterior girders during the static load tests when the trucks first occupied Lanes 2 and 4 and then Lanes 1 and 5, respectively. The maximum deflection measured during the rolling tests was 0.47 in., which occurred in the exterior girder during the Lane 1 loading. All measured deflections were less than 50% of the suggested 1.04-in. (L/800) limit (AASHTO 2004).

Strain profiles were developed at the quarter and midspans and were used to confirm composite action and develop the lateral load distribution factors; a typical midspan strain profile when the truck is in Lane 1 is presented in Figure 8. A maximum midspan tensile stress of 5.54 ksi was determined during the static loading. A total tensile stress of 26.2 ksi results when the effects of the live load and the self weight are combined. Similar to MBISB 1, there was no significant evidence of restraint at the abutments, which indicated a simply supported structure. The resulting experimental values for both single and multiple lane loadings were calculated and compared to AASHTO LRFD design values for steel girder/concrete slab bridges; the results, presented in Table 1, have been adjusted to reflect similar multi-presence and traffic use factors. When compared to the field test values, the AASHTO distribution factors are conservative and thus are considered applicable to the MBISB.

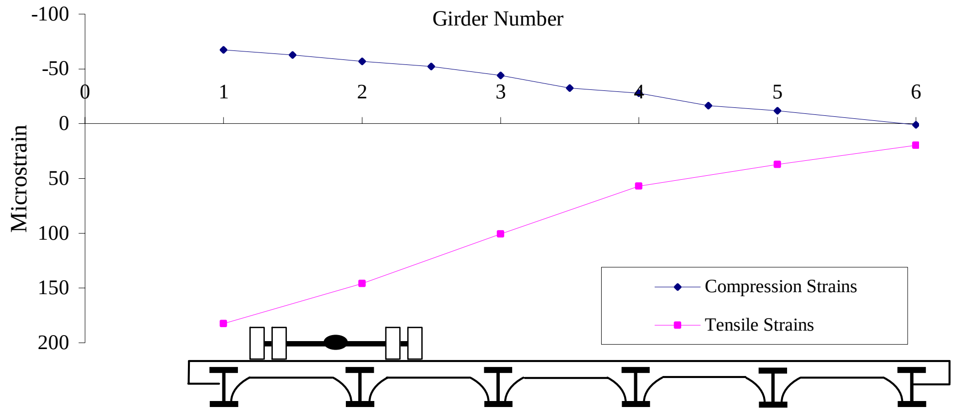

Figure 8. Midspan strain profile for MBISB 2, Lane 1 loading

Table 1. Experimental and AASHTO moment distribution factors for MBISB 2

	DISTRIBUTION FACTORS			
Girder	One Lane (%)		Two Lane (%)	
	Experimental	AASHTO LRFD Specification	Experimental	AASHTO LRFD Specification
Interior	32	36	43	49
Exterior	36	46	41	56

MBISB DESIGN METHODOLOGY

When the structural behavior of the laboratory model bridge and MBISB 2 under service-level loads were compared to AASHTO LRFD design values for a slab/girder bridge, the specification values were conservative, but deemed applicable for future designs. Thus a design methodology, similar to that of a standard composite slab/girder bridge was developed for the MBISB system. A detailed discussion of the addressed limit states and design parameters is provided in Volume 3 of the final report for Iowa DOT project TR-467 (Wipf et al. 2004a; AASHTO 1994).

To facilitate the use of the MBISB system, a series of designs were created using the methodology developed for use by county engineers and consultants in need of a replacement LVR bridge with a span of up to 80 ft. The designs provide the engineer with design information for the MBIBS system based on the following six parameters:

- Bridge length
- Bridge width
- Girder spacing
- Steel yield strength
- Concrete compressive strength
- Depth of cover

Designs were created for nine different lengths, increasing in 5-ft increments from an initial length of 40 ft. Two overall bridge widths, 26 ft and 32 ft, were selected so that two lanes of traffic or large agricultural equipment could be accommodated. Girder spacing is determined by dividing the predetermined bridge width by the number of girders used. For each girder spacing, the five lightest W sections that satisfied the strength limit state for an HS-20 design truck and lane load are included so that recycled girders, if available, can be readily evaluated for a variety of design configurations. A concrete compressive strength of 4 ksi is conservatively specified for the MBISB deck. Yield strengths of the longitudinal girders are specified as either 36 ksi or 50 ksi, which are readily available structural steels. A yield strength of 60 ksi is specified for the deformed reinforcing bars used in the concrete deck.

With the establishment of the previously stated boundary conditions, a series of designs were evaluated and the output arranged in a tabular format. An example of such an output for a 65-ft–long, 32-ft–wide MBISB, with a steel compressive strength of 50 ksi and 3 in. of concrete placed above the top flanges of the longitudinal members, is presented in Table 2. A complete listing of the designs available is presented in Volume 2 of the final report for Iowa DOT project TR-467 (Wipf et al. 2004b). In addition to the design outputs, a detailed example and explanation of the use of the design information is included. With the information provided in the design output for the selected bridge geometry, the user can readily compare gross material quantities for estimation purposes and determine the design option that best address a given site.

Table 2. Design output example for a 65 ft long, 32 ft wide MBISB

Number of girders @ spacing	Section	Radius of formwork (in.)	Volume of concrete (yd^3)	Weight of steel (kips)	Interior camber (in.)	Exterior camber (in.)	Number of diaphragms	Diaphragm spacing B (ft)	Service level deflection (in.)	Optional defl. control	Water sliding force (kips)
6 @ 6 ft	W30X116	23.5	91	45.94	4.25	3.75	2	21	0.642	Y	29.6
	W30X124	23.5	91	49.1	4	3.5	2	21	0.615	Y	29.8
	W27X129	20.5	86	51.08	4.25	3.75	2	21	0.752	Y	26.9
	W24X131	17.5	79	51.88	4.5	4	1	0	0.964	Y	23.6
	W30X132	23.5	92	52.27	3.75	3.25	2	21	0.594	Y	29.9
7 @ 5 ft	W30X108	23.5	97	49.9	4	3.5	2	21	0.61	Y	29.4
	W27X114	20.5	90	52.67	4.25	3.75	2	21	0.737	Y	26.6
	W30X116	23.5	98	53.59	3.75	3.25	2	21	0.582	Y	29.6
	W24X117	17.5	83	54.05	4.5	4	1	0	0.935	Y	23.4
	W30X124	23.5	98	57.29	3.5	3	2	21	0.557	Y	29.8
8 @ 4.29 ft	W27X102	20.5	95	53.86	4.25	3.75	2	21	0.718	Y	26.4
	W27X114	20.5	95	60.19	4	3.5	2	21	0.676	Y	26.6
	W24X117	17.5	87	61.78	4.25	3.75	1	0	0.855	Y	23.4
	W21X122	15	80	64.42	4.5	4	1	0	1.09	N	20.7
	W27X129	20.5	96	68.11	3.5	3	2	21	0.623	Y	26.9
9 @ 3.75 ft	W24X103	17.5	91	61.18	4.5	4	2	21	0.85	Y	23.6
	W24X104	17.5	90	61.78	4.25	3.75	1	0	0.851	Y	23.2
	W21X111	15	82	65.93	4.5	4	1	0	1.073	N	20.5
	W24X117	17.5	91	69.5	4	3.5	1	0	0.791	Y	23.4
	W21X122	15	83	72.47	4.25	3.75	1	0	1.007	N	20.7
11 @ 3 ft	W21X93	15	89	67.52	5	4.5	2	19	1.06	N	20.6
	W21X101	15	88	73.33	4.5	4	1	0	0.991	N	20.4
	W18X106	12	80	76.96	5	4.5	1	0	1.33	N	17.7
	W21X111	15	88	80.59	4.25	3.75	1	0	0.938	Y	20.5
	W18X119	12	80	86.39	4.75	4.25	1	0	1.213	N	18

Material and section properties: f_y = 50 ksi, f'_c = 4 ksi, Cover = 3 in., L/800 = 0.975 in.

CONCLUSION

Results from the laboratory and field evaluations verified that the modifications, the ASC and the transverse arch, can be successfully incorporated into a full--scale bridge, resulting in the MBISB system. Laboratory testing of the MBISB system provided evidence that the deck resisted the simulated wheel loads through arching action and provided a minimum reserve capacity in excess of 70 kips when subjected to a simulated HS-20 wheel loading. Results from the service loadings indicated that the experimental lateral load distribution factors present in the model bridge were similar to the AASHTO

LRFD design values. When subjected to ultimate flexural loads, the girders yielded prior to the occurrence of a localized punching shear failure, resulting in a ductile failure.

The two demonstration bridges provided verification of the applicability of the MBISB design by exceeding performance standards while being constructible by county forces at costs less than conventional designs. The structural behavior of MBISB 2 was determined and compared to the AASHTO LRFD design values for a slab/girder bridge, which were both conservative but applicable for the design of the MBISB system.

A design methodology for future MBISBs was then developed by applying the AASHTO LRFD limit states for a slab/girder bridge while incorporating the modifications for composite action and formwork for the deck. A series of designs were developed and included in a design manual to assist engineers in the design of future MBISBs.

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The opinions, findings, and conclusions expressed herein are those of the authors and not necessarily those of the Iowa DOT or the Iowa Highway Research Board.

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Deployment of Statewide Transit Intelligent Transportation Systems through the Iowa Rural Transit ITS Consortium

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ABSTRACT

This paper will present an overview and analysis of a regionally based, statewide transit intelligent transportation system that incorporates modern management software with automatic vehicle location and GPS capability. It will discuss problems, solutions, and perceived opportunities for the new system.

Note: This research was still in progress at the time of publication; contact the author above for more information.

Key words: automatic vehicle location—GPS—intelligent transportation systems

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Content of Existing Statewide Bicycle Maps

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ABSTRACT

A statewide bicycle map is a map of an entire state which shows available cross-state routes. A bicyclist could get sufficient information on road type, conditions, traffic information, and other details while planning a trip. This paper summarizes the observations made on existing statewide bicycle maps—a study of what other states do for their statewide bicycle maps. It was observed that each state has a unique way of presenting its bicycle resources to the bicyclist. Some states such as Ohio and New Jersey use what they call a "Guide," which includes the map along with a booklet on rules and regulations, places of interest, etc., while some states, such as Pennsylvania, present the map in the form of a manual. Tennessee has a unique way of presenting routes on cards with written driving directions on backside of each card. A web search was performed to determine what the various states do. Web pages of all state DOTs and tourism departments were examined. It was observed that 23 states have bicycle maps available for the public, and 17 maps of these are also available online. Each state followed its own way of presenting the data on the bike map. The contents of the maps varied considerably. A note was made of the information each state includes on its bike map. The content items included defined routes, information on parks and areas of interest, and Annual Average Daily Traffic (AADT). The types of information were then summarized by tabulation. The style of presentation was also noted.

Key words: bicycling—bicycle maps

INTRODUCTION

A statewide bicycle map is a map of an entire state which shows available cross-state routes. This map can provide a broad range of information to the bicyclist who plans to travel from one place to another. A bicyclist could get information on road type, conditions, traffic information, and other details while planning a trip.

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 made it mandatory that bicycle facilities be considered in the statewide and metropolitan planning processes. Each state Department of Transportation was required to have a bicycle coordinator to be responsible for ensuring that bicycle facilities are considered in the planning, design, and construction of transportation improvements at the state level. Research has been going on at many state Departments of Transportation (DOT) to evaluate their bicycle facilities and the aids they provide for their bicyclists.

The key interests for most of the states include measure of bicycle suitability criteria or the bicycle level of service and statewide bicycle maps. This paper summarizes the observations made on existing statewide bicycle maps provided by various states.

LITERATURE REVIEW

According to Mozer, the choice of the right bicycle map for a given community is very subjective. Factors that distinguish maps include stated purpose, scale, level of graphic effort (or expertise), type of medium (paper or plastic), size and format, symbol use, field research approach, and distribution system.

There were no standard procedures or guidelines provided by any organization found for determining the design and content of a bike map. This makes sense sometimes because it is logical to include content items such as elevation and grade on mountainous Colorado but perhaps not in relatively flat Kansas.

REVIEW OF EXISTING STATEWIDE BIKE MAPS

A web search was conducted to determine state practices. Web pages of all state DOTs and tourism departments were examined. It was found that 23 states have bicycle maps available for the public, and 17 of these maps are also available online. Each state followed its own style of presenting the data on the bike map. The contents of the maps varied considerably. A note was made of the information each state includes on its bike map. The style of presentation was also noted.

The following Figure 1 lists states with statewide bicycle maps and the contents of each map. All the states have defined their bicycle routes. Twelve of the states include information on parks and areas of interest. Nine of the states also have traffic volume details among their bike routes. Seven states provide grade and/or terrain details. Four states provide online version of the map for free and sell the hard copy of the map, while the rest of the states provide the hard copy version of the map free of charge.

State	Defined Routes	Parks/Areas of Interest	Average Daily Traffic/Traffic Volume	Shoulder Width	Bicycling Laws	Safety Tips	Grades	Bicycle Shops	Services/Restrooms	Written Directions	Climate	Bicycle Contacts	Terrain	Insets of Urban Areas/Segemental Maps	Emergency Contact Information	Types of Routes (Loops, Cross-State, Etc.)	Percent Trucks/Truck Volume	Wind Direction	Online Availability	Hardcopy Availability	Free Map?
Colorado	x		x	x	x	x	x				x	x							x	x	x
Connecticut	x	x							x							x			x	x	x
Georgia	x										x	x								x	
Illinois	x	x			x	x		x	x		x								x	x	x
Iowa	x	x	x		x									x					x	x	x
Kansas	x	x	x	x				x								x			x		x
Maryland	x	x		x		x									x					x	x
Massachusetts	x	x							x											x	
Minnesota	x	x	x	x															x	x	
Mississippi	x																		x		
Montana	x		x	x					x			x	x		x				x		
Nebraska	x		x	x	x	x											x		x	x	
New Mexico	x	x	x	x					x					x		x			x	x	x
New Jersey	x	x				x	x	x		x									x	x	x
New York	x							x		x									x	x	x
North Carolina	x	x							x				x	x					x	x	x
Ohio	x									x						x			x	x	x
Oregon	x		x				x	x			x		x					x	x	x	x
Pennsylvania	x																			x	x
Rhode Island	x				x		x			x		x			x				x	x	x
Tennessee	x				x	x				x				x					x	x	x
Washington	x	x	x	x			x													x	x
Wisconsin	x	x						x								x				x	

Figure 1. State bicycle map content

GUIDELINES FOR CHOOSING BICYCLE MAP CONTENTS

Based on the observations made on existing statewide bicycle maps, the authors recommend the following map content items for consideration when designing a statewide bicycle map.

The map must show routes and various locations on the route clearly. The map should have a distance chart so that the bicyclist will have clear idea of the distance he or she has to travel to reach a destination. This also helps a bicyclist in planning stops for the journey. Iowa, Colorado, and Ohio have such content on their statewide maps. The map may also include information on the available shoulder width and Annual Average Daily Traffic (AADT). Colorado, Iowa, Maryland, and Washington State have AADT details on the bike maps. The above states along with Montana and Nebraska have included pavement shoulder details.

If there is more than one route to a destination, the map should also suggest the most favorable route. (This might depend on climatic condition/terrain and time of year). The state maps of Rhode Island, Illinois, and Connecticut suggest/recommend certain routes among the other possible routes. The map may also give a description of climatic conditions throughout the year at different locations. Terrain details would be useful information to the bicyclist. The state map of Colorado, Oregon, and Georgia provide meteorological information helpful to the bicyclist.

The map may also discuss "Rules of the Road" cycling etiquette and state law as it pertains to cycling. It could also provide safety tips for a hazard free journey and display the basic indications and signals that the bicyclist has to provide. Some of the statewide bicycle maps provide some kind of information on the rules to be followed.

The map should have a clear and easily understandable legend showing the various codes for different routes based on classification, AADT, suitable routes, shoulder details, or type of route (interstate/state highway, etc.). The legend should also explain the symbols used and the scale of the map. The map could also indicate locations of multi-modal intersections/locations, such as Amtrak, local metro service, and ferry service. A map index would be of great use to the bicyclist for locating a place easily. Illinois, Iowa, Ohio, and Rhode Island provide ample information on the inter-modal intersection locations.

Trail routes, recreational centers, and other locations of public interest (such as forests and parks) may also be listed. A brief description of each of the locations would be useful to most bicyclists. An enlarged sub-map or a table of the trial routes and other public recreational locations with associated details, such as distance, contact details, and facilities available, would be of great help to many bicyclists. Almost all of the state-wide bicycle maps examined have provided information on the trials available in their state along with sub-maps of the trial routes.

The map may also have contact information for the local county police, ambulance, inter-modal service, transportation authorities, trial office, highway patrol, or other service providers.

It would be advantageous to the bicyclist if the map has sub-maps of cities on a greater scale to give a further clear picture on the city routes for his/her more specific destination.

DESCRIPTION OF EACH STATE'S BICYCLE MAP

The following paragraphs provide brief summaries of the map content and style for each state in alphabetical order. Links to available bike maps are given in the Table 1 following the summaries.

Colorado Department of Transportation (CDOT) provides a free bicycle map that includes climate information for different seasons in different areas, mileage between major cities, traffic volumes expected on all routes, mountain passes, a clear legend with shoulder, and other details. The 79-page manual also has a guide to bicycle parking and pedestrian information. A credit-card-size pocket guide is included showing rules for multiple-use trails.

Connecticut's bike map shows the state's recommended cycling routes. The map provides both local and cross-state routes. It has additional information on cycling in and around the state. State parks and wildlife areas are noted on the map.

Georgia map presented by Georgia DOT shows statewide bicycle routes and counties with adopted bicycling plans. In addition to showing bicycling routes, the map discusses "Rules of the Road" cycling etiquette and Georgia law as it pertains to cycling. A table of weather patterns in Georgia throughout the year is provided. Safety tips and bicycling contacts are listed as well.

Illinois has a set of nine bicycle maps covering the entire state. Each regional map includes places of interest, local government office contact details, climatic conditions, bicycle rules and safety tips, and a clear legend explaining various routes and symbols used.

Iowa DOT has bicycle map which shows AADT, small maps on trail routes, and a map index of towns and villages. A parks and recreation directory and trail information also help bicyclists coming to the area. Different colored lines distinguish between average daily traffic (ADT) volumes. The map includes mileage charts, Iowa bicycling laws, and insets of urban areas.

Kansas DOT converted their highway map into a bicycle map by highlighting the various routes that are accessible by bicycles. No other details are given except the suggested bicycle routes.

Maryland's state bicycle map lists the various routes on which bicycles are totally prohibited or partially prohibited. It also has description of various places of interest in different counties. The trails in the major trail system are shown on one side of the map. The authors found it hard to discern between the four statewide trails because they look as if they are one trail.

Massachusetts provides information regarding eight trails in the state. The DOT does not have a statewide bike map of its own. A private organization prints the statewide map in four parts.

Minnesota state map is available in four parts covering the northeast, northwest, southeast, and southwest part of the state. The maps and the legend are on separate documents. The legend describes the shoulder detail categories and road classifications. The difference between each category is defined by the use of different colors and lines. State parks, as well as paved off-road state bike trails, are also specified.

Mississippi state bicycle map lists the cross-state routes available. No other details are given except the suggested bicycle routes.

Montana's map and the information provided in a brochure and online are intended to be used in conjunction with the Montana State Highway Map. The bike map has shoulder details. For some other details, the user is referred to the highway map. Emergency information and bike resources aid are included in the bike materials. Colors are used to discern between the type of terrain and shoulder width.

Nebraska issued a bicycle guide in 1999 containing a statewide bicycle map. The legend shows the shoulder details, route classification, traffic volumes, and truck volumes. This map was originally a highway map over which the bicycle routes are highlighted. One side of the map has the state bicycle routes, while the other side has bicycle laws and safety tips.

New Mexico DOT recently released its statewide bicycle map. The map legend shows shoulder details, AADT, route numbers, and mileage chart. The map has small inset maps for cities in the state on a larger scale with more details on the back of the map.

New Jersey DOT issues a bicycle manual which mentions the law, driving instructions, rules, and regulations. It also provides information regarding routes where bicycle traffic is prohibited. A private organization (www.njbikemap.com) provides a statewide bike map online.

New York does not have a complete statewide map as such. It has many individual regional maps and maps for three routes running through the state, namely Routes 5, 9, and 17. Two proposed routes are drawn on the same map.

North Carolina describes the nine routes it has across the state. Each route has its own map and comes with segmental maps to better show the route. Terrain descriptions, road conditions, services, points of interest, and a campground directory are included.

Ohio DOT issues a free bicycle map displaying all municipal communities with greater resolution maps for some larger cities. The map also has a distance chart and a clear legend showing road classifications and symbols used. A cross-state bike map is available for sale.

Oregon bicycling guide includes a bicyclist manual and coast bike route page. Rules of the road, weather, topography, mileage table between cities, grades, summer wind directions, bicycle shops, and traffic volumes are included for bicycling tourists. This map and its accompanying counterparts are packed with information.

Pennsylvania maintains six signed bicycle routes and also has the "rails to trails" program. Each route is shown on a large map, but individual trail maps are available. A bicycle driver's manual and directory are also available. The entire information is issued in the form of a booklet.

Rhode Island has a clear and complete statewide bicycle map with a clear legend, symbols, and road classification. The map is issued by the Rhode Island Department of Transportation. Intermodal connections for the ferry service, rail, and airways are shown with symbols.

Tennessee DOT's bicycle map shows individual routes printed on cards. Though the single sheet statewide map appeared vague to the authors, clear driving directions are given on each route card. The packet shows Tennessee's bicycling laws and safety tips on the back cover

Washington state bicycle map has many details, including AADT and shoulder widths. The map also lists the various routes where bicycling is prohibited.

Wisconsin state bicycle map is an eight-paneled map designed in conjunction with the Department of Transportation (DOT) specifically for bicyclists. It rates county and state highways for bicycle suitability and also shows bicycle trails, mountain bike facilities, and bike stores. Maps are available at bike stores or by telephone order.

Table 1. Links to available bike maps

State	Link
Colorado	http://www.dot.state.co.us/bikeped/maps.htm
Connecticut	http://www.ct.gov/dot/LIB/dot/documents/dbikes/fullmap1.pdf
Georgia	http://www.dot.state.ga.us/dot/plan-prog/planning/projects/bicycle/images/approved_bikeped_Plans_MAP102103.pdf
Illinois	http://www.dot.state.il.us/bikemap/STATE.HTM
Iowa	http://www.msp.dot.state.ia.us/trans_data/mrsid/bikemap.html
Kansas	http://www.ksdot.org/burrail/bike/biking/bikemap2002.pdf
Maryland	http://www.sha.state.md.us/SHAServices/mapsBrochures/maps/OPPE/maps.asp
Massachusetts	http://www.state.ma.us/mhd/paths/bikep.htm
Minnesota	http://www.dot.state.mn.us/sti/map.html
Mississippi	http://www.mdot.state.ms.us/bicycling/bikeroutes.pdf
Montana	http://www.mdt.state.mt.us/planning/ctep/bicycle.html
Nebraska	http://www.dor.state.ne.us/info/docs/bikegide99.pdf
New Mexico	http://www.nmshtd.state.nm.us/upload/images/Bike_pedestrian_Esquestrian/BIKE02map.pdf
New York	http://www.dot.state.ny.us/pubtrans/bikemap.html
North Carolina	http://www.ncdot.org/transit/bicycle/maps/maps_highways.html
Ohio	http://www.dot.state.oh.us/bike/default.htm
Oregon	http://www.odot.state.or.us/techserv/bikewalk/orbikgud.htm
Pennsylvania	http://www.dot.state.pa.us/
Rhode Island	http://www.dot.state.ri.us/WebTran/ristatebikemap.pdf
Tennessee	http://www.tdot.state.tn.us/bikeroutes/routes.htm
Washington	http://www.wsdot.wa.gov/bike/Planning_Maps.htm
Wisconsin	http://www.dot.wisconsin.gov/travel/bike-foot/bikemaps.htm

SUMMARY

The authors observed the existing statewide bicycle maps through a web search on Web pages of all state DOTs and tourism departments. It was observed that 23 states have bicycle maps available for the public, and 17 of these maps are also available online. Each state followed its own way of presenting the data on the bike map. The contents of the maps varied considerably. Based on the observations made on existing statewide bicycle maps, the authors recommend content items for consideration when designing a statewide bicycle map. The paper provides a brief summary of the map content and style for each state in alphabetical order.

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Concrete Pavement Pooled Fund Studies Led by the Iowa Department of Transportation

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ABSTRACT

This presentation will provide an overview of several pooled fund studies being led by the Iowa Department of Transportation and conducted at Iowa State University:

- Material and Construction Optimization for the Prevention of Premature PCC Pavement Distress (FHWA, 17 state DOTs, private industry)
- Deicer Scaling Resistance of Concrete Pavements, Bridge Decks, and Other Structures Containing Slag Cement (FHWA, 7 state DOTs, Slag Cement Association)
- Self-Consolidating Concrete Applications for Slip-Form Paving (FHWA, 4 state DOTs, private industry)
- Development of Performance Properties of Ternary Mixes
- PCC Surface Characteristics: Tire-Pavement Noise Program Part 3—Innovative Solutions/Current Practices

Key words: concrete pavement research—pooled fund studies

Design and Evaluation of Accelerated Bridge Construction

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ABSTRACT

New bridge systems are needed that will allow components to be fabricated off-site and moved into place for quick assembly while maintaining traffic flow. Depending on the specific site conditions, the use of prefabricated bridge systems can minimize traffic disruption, improve work zone safety, minimize disruption to the environment, improve constructibility, increase quality, and lower life-cycle costs. This technology is applicable and needed for both existing and new bridge construction. 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 design and construction process for the first application of truly accelerated bridge projects in the state of Iowa. The FHWA Innovative Bridge Research and Construction program has awarded funding for two bridges that utilize precast, high-performance concrete elements to be constructed in Boone County and Madison County, Iowa.

Researchers from the Bridge Engineering Center at Iowa State University will install instrumentation and monitor the structural behavior of these two bridges following construction. A discussion of the structural instrumentation and monitoring of these innovative bridges will be presented.

Key words: accelerated bridge construction—bridge construction—bridge replacement—innovative bridge design—instrumentation—structural monitoring

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INTRODUCTION

This paper presents an overview of the preliminary design for two innovative bridge replacement projects, located in Boone County and Madison County, Iowa. The projects are being partially funded by the Federal Highway Administration (FHWA) Innovative Bridge Research and Construction (IBRC) program.

These two bridge projects are representative of those undertaken by county engineers across Iowa and throughout the nation. Projects such as these are located on lightly traveled local roads. Due to the light volume of traffic, it is not essential to construct the bridge in an overly accelerated fashion. However, the rapid construction technologies demonstrated on these projects can hopefully be expanded and adapted for use in higher traffic urban areas on the primary road system in Iowa.

IBRC PROGRAM

The FHWA established the IBRC program as part of the Transportation Equity Act (TEA-21) federal highway funding bill in 1998. The program is intended to demonstrate the application of innovative material technology in the construction of bridges and other highway structures, and has two components. The larger component provides funds for repair, rehabilitation, replacement, or new construction of bridges and other highway structures using innovative materials. The smaller component is intended to support research and technology transfer activities related to the program’s goals.

Projects submitted for funding to the IBRC program are encouraged to utilize the capabilities and qualities of the innovative materials being deployed, including the following:

- Bridge components designed for rapid installation or expansion
- Combinations of more than one innovative material in a bridge component
- Innovative bridge designs that result in beneficial features such as shallow superstructures, longer spans, or fewer substructure units
- Innovative applications that enhance bridge integrity and decrease vulnerability to damage from both natural and manmade hazards
- Bridges that incorporate smart materials or embedded instrumentation for future continuous monitoring of operational performance
- Projects that stress innovative technology to monitor, measure, and report on engineering and operational performance of bridges, particularly those with high performance materials

Application for FY 2004

The Iowa DOT, working closely with the FHWA Iowa Division Office, submitted applications for two IBRC projects for accelerated construction in fiscal year 2004.

Madison County

The innovative aspects of the Madison County project are intended to minimize construction time by using high-performance concrete, such that the overall profile (grade and elevation) of the bridge does not change by designing girders with shallower cross-sections and by using precast concrete abutment units. As a result of the reduced profile, the bridge owner will not need to purchase additional right-of-way and will minimize any required earthwork. It was envisioned that the overall construction time could be reduced by 50%, which addresses the IBRC goal of using bridge components designed for rapid construction. The total federal funding requested was $400,000. This total included approximately $235,000 for innovative materials and $155,000 for instrumentation and performance evaluation.

The total FHWA funding awarded was $200,000.

Boone County

The innovative aspects of the Boone County project are the use of precast reinforced concrete abutments, piers, and slab units, which will significantly reduce construction time. The two piers required for the replacement structure will make it possible to install two different types of systems for side-by-side comparison. It is estimated that the use of precast bridge elements will reduce construction time by approximately 60%. The total federal funding requested was $435,000. This total included approximately $270,000 for innovative materials, $15,000 for preliminary engineering, and $150,000 for an innovative material performance evaluation for a post-construction period of approximately two years.

The total FHWA funding awarded was $400,000.

Past Iowa IBRC Projects

It should be noted that the Iowa DOT and a number of Iowa counties, have been involved with IBRC projects since the inception of the program. Past projects have used the following advanced technologies:

- Fiber reinforced polymer (FRP) deck panels and reinforcing
- High-performance concrete deck using proprietary admixtures
- Rehabilitation of steel girder bridge using external FRP prestressing tendons
- Construction of bridge deck using corrosion-resistant MMFX reinforcing steel
- Bridge replacement utilizing glue-laminated timber composite with FRP materials
- High-performance steel girders (hybrid)
- Strengthening of steel girder bridge using FRP plates bonded to bottom flange
- Ultra high strength (up to 30 ksi) prestressed concrete beams
- Temporary detour bridge using FRP superstructure units

BOONE COUNTY BRIDGE

The proposed bridge site is located on 120th Street over Squaw Creek in north-central Boone County, Iowa, approximately two miles east of US Highway 17. The calculated drainage area for this particular site encompasses 88 square miles and provides a design discharge of 3,450 cubic feet per second. Soil conditions at the site consist of very sandy lean clay near the abutments and west pier and a firm to very firm glacial clay foundation material near the east pier. Standard blow counts range from 19 to 27 in the intended very firm glacial clay foundation layer.

The existing bridge is one of only eleven Marsh Rainbow Arch bridges remaining in Iowa. The bridge is a 76-foot–long, single arch span with an 18-foot–wide roadway measured from gutter-to-gutter. The narrow roadway, as well as the deteriorated condition of the existing concrete arch ribs, has necessitated the replacement of this historic structure (see Figure 1).

The Marsh Rainbow Arch Bridge was once fairly common in Iowa and other states in the central part of the United States (Hippen 1997). This patented bridge design, developed by James B. Marsh of Des Moines, was constructed from 1911 to the late 1930s. The bridges were designed for use as relatively small highway bridges with individual spans from 40 to 100 feet. The arch spans provide an early example of composite construction, consisting of structural steel lattice girders encased in cast-in-place concrete (see Figure 2). In addition to the aesthetic appeal of the Rainbow Arch, the structural steel components provided sufficient strength prior to placing concrete to permit construction of the bridge without the need for falsework in the streambed.

Figure 1. Existing bridge, Boone County

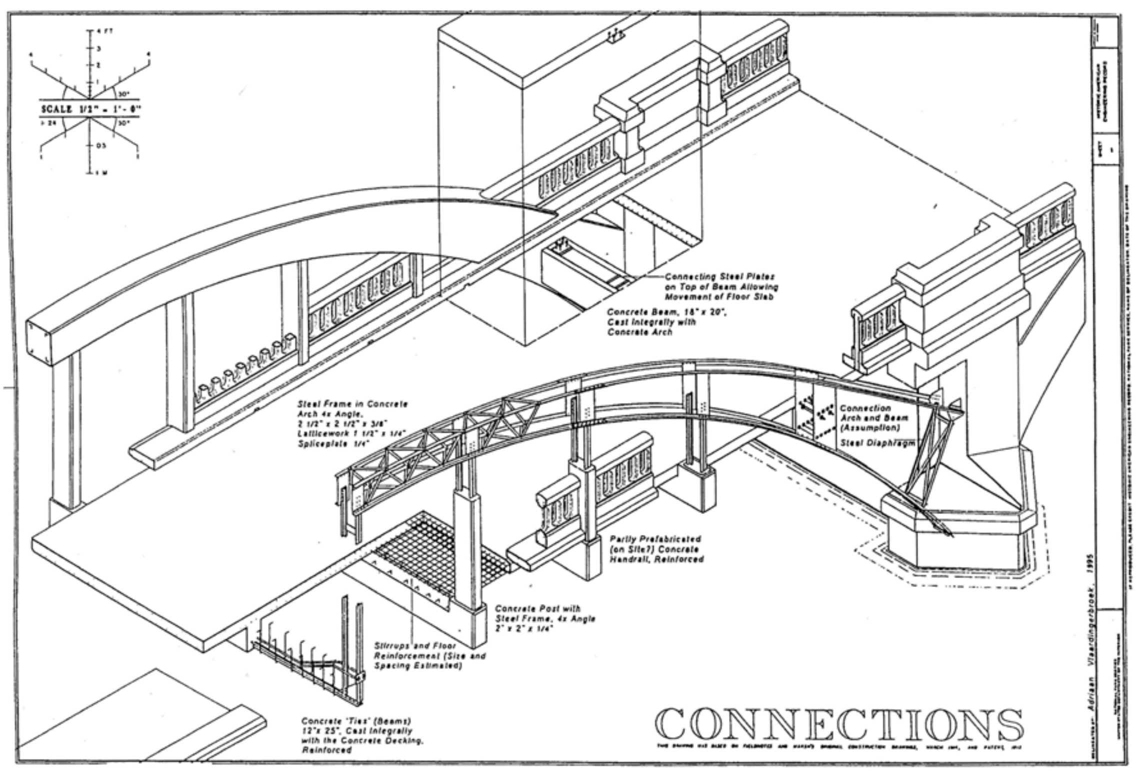


Figure 2. Marsh arch details

Proposed Replacement Bridge

The proposed replacement bridge is a 151-ft 4-in. by 33-ft 2-in., three-span, deck-on-girder structure, which will be constructed with a 30-degree right ahead skew. The span length geometry of the replacement bridge is based on a typical Iowa county bridge standard that would have been constructed had this structure not been chosen as a demonstration of innovative accelerated bridge construction. The three spans are 47-ft 5-in., 56-ft 6-in., and 47-ft 5-in., respectively. The abutments will be designed as integral abutments, and both piers will be designed as fixed piers.

The proposed replacement abutments will utilize a steel H-pile foundation. In order to accelerate construction, a precast concrete pile cap will be designed. The connection of the cap to the piles will be made using a grouted connection to the pile group (see Figure 3). Voids in the precast abutment pile cap will be sized to accommodate the piles and pile driving tolerances.

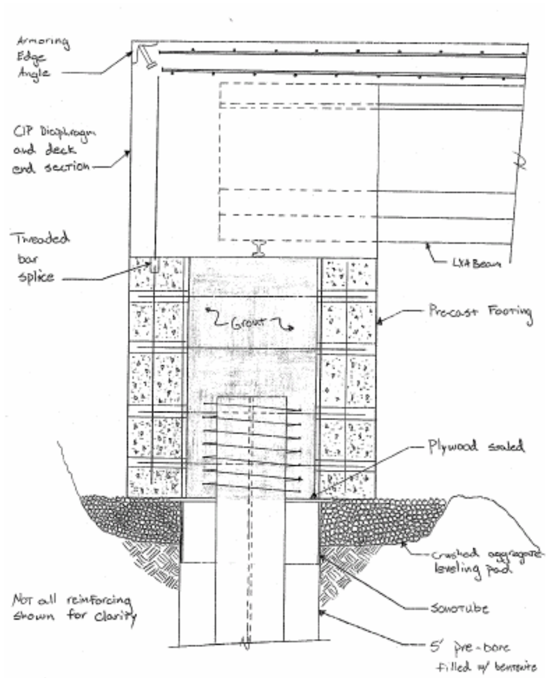


Figure 3. Precast concrete abutment, Boone County

The use of steel pipe piles or concrete-encased steel H-piles is being considered, and it is possible that both types will be used in separate piers, pending final review of the geotechnical information available from the site and the development of a suitable structural connection to the cap beam. The pier itself will consist of a precast pile cap with cast-in-place concrete diaphragms surrounding the girder ends (see Figure 4).

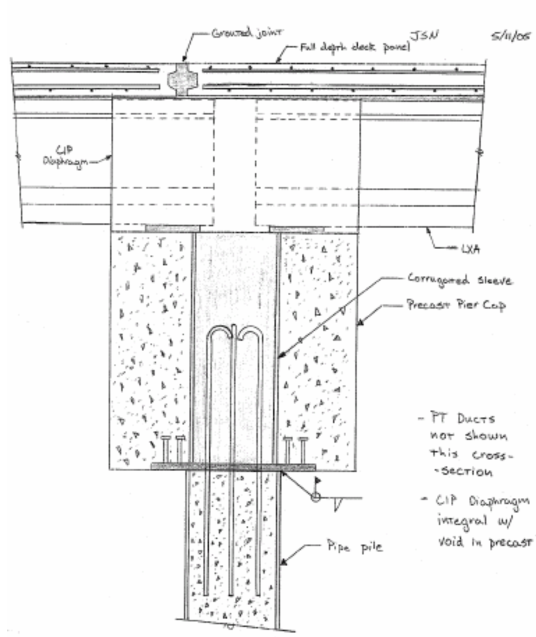


Figure 4. Precast pier cap beam, Boone County

The bridge superstructure will consist of pretensioned, prestressed concrete beams (Iowa DOT LXA series) with full-depth, precast, posttensioned concrete deck panels. The deck panels will be set on the beams using a screw leveling system, allowing panel adjustment to accommodate the profile grade. Grouted keys will be used between each panel to accommodate irregularities in the panel edges and avoid localized stress concentrations when posttensioning. Following the posttensioning, shear keys in the deck panels and haunches will be grouted to connect the deck to the PPC beams.

The use of a precast concrete deck is complicated by the 30-degree skew. In order to avoid the need for custom tapered panels and to make the abutment act as an integral connection to the superstructure, the deck at the ends of the bridge will be cast in place integrally with the abutment diaphragm also following the posttensioning of the deck panels (see Figure 5).

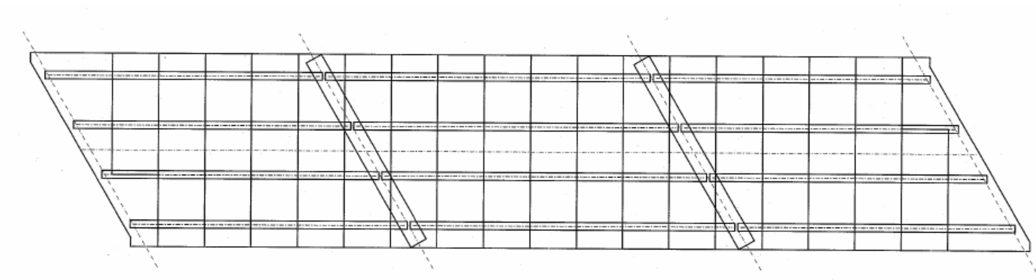


Figure 5. Precast concrete deck panel layout, Boone County

The use of a precast concrete barrier rail is being considered for this bridge. A precast barrier would consist of individual T-shaped segments similar in appearance to a corral rail-style of barrier common on secondary roads in the Midwest. The precast segments would be posttensioned to the precast concrete deck panels using high-strength rods.

MADISON COUNTY BRIDGE

The proposed bridge site is located on 290th Street over an unnamed branch of North Fork Clanton Creek in southern Madison County, Iowa, approximately one mile east of U.S. Highway 169.

The calculated drainage area for this particular site encompasses 212 acres and provides a design discharge of 340 cubic feet per second. Soil conditions at the site consist of approximately 30 to 35 feet of clays with blow counts ranging from 7 to 21 overlaying shale and limestone.

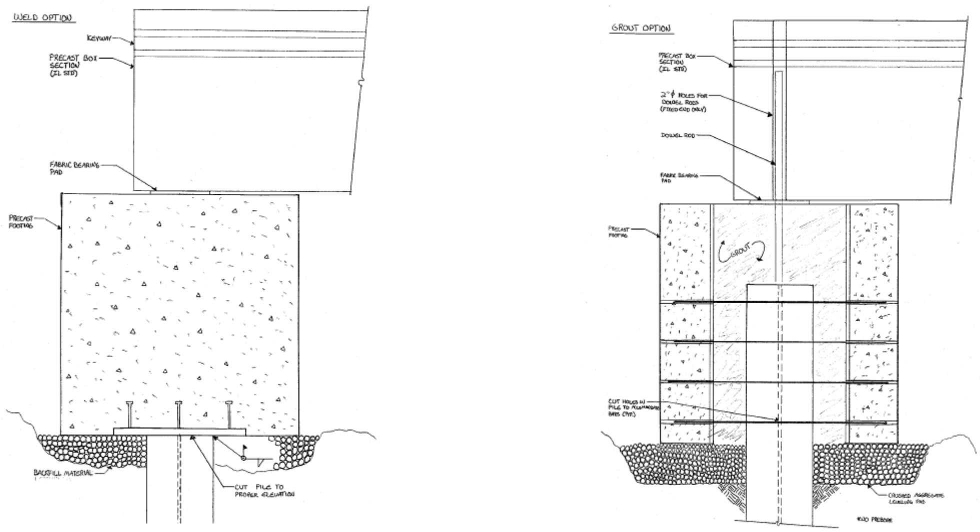
The existing bridge consists of a single 21-foot–long span comprised of timber stringers with transverse plank decking with a gravel wearing surface. The roadway width measures 17-ft 8-in., measured to the face of the timber guardrail. The bridge is supported on timber piling and backwalls (see Figure 6).

Figure 6. Existing bridge, Madison County

Proposed Replacement Bridge

The proposed replacement structure consists of a single span, 46 ft 8 in. long, and will provide a 24-foot–wide roadway section. This increased roadway width and span length will involve a small quantity of excavation as the existing profile grade will be matched. No additional right-of-way will need to be acquired for the replacement of this structure. However, a temporary easement will be needed to rebuild a field entrance near the bridge site.

The substructure for the proposed bridge will consist of a precast concrete abutment section supported on driven steel H-pile end bearings in limestone. Connection of the precast component to the piles will be by means of either a welded connection to an embedded plate in the bottom of the abutment barrel or by means of a grouted connection following installation of the abutment section (see Figure 7).



a) Welded connection to steel pile **b) Grouted connection to steel pile**

Figure 7. Precast concrete abutment, Madison County

The proposed replacement superstructure consists of a set of precast, pretensioned concrete box girder sections. The details for these specific box girders have been obtained from the Illinois DOT and may be modified slightly in the final design to satisfy owner preferences. The box girder sections are 27 in. deep and 48 in. wide (see Figure 8). Adjacent box girders are connected by means of a 1-in. diameter steel rod with shear transfer provided by a grouted shear key (see Figure 9). Earth behind the abutment will be retained by means of a concrete diaphragm cast within the precast box girder sections. A gravel overlay may be placed on the bridge deck to obtain the proper deck cross-section and to protect the precast panels from grading operations. Steel guardrails will also be installed.

Connection of the box girders to the precast abutment will be made through the use of steel dowel bars at the fixed abutment, similar to the details used by the Illinois DOT. Due to the short expansion length of this bridge, a fixed connection at both abutments is also being considered. In addition, the use of a posttensioned connection may be utilized in an attempt to promote continuous, rigid frame behavior similar to a typical integral abutment bridge.

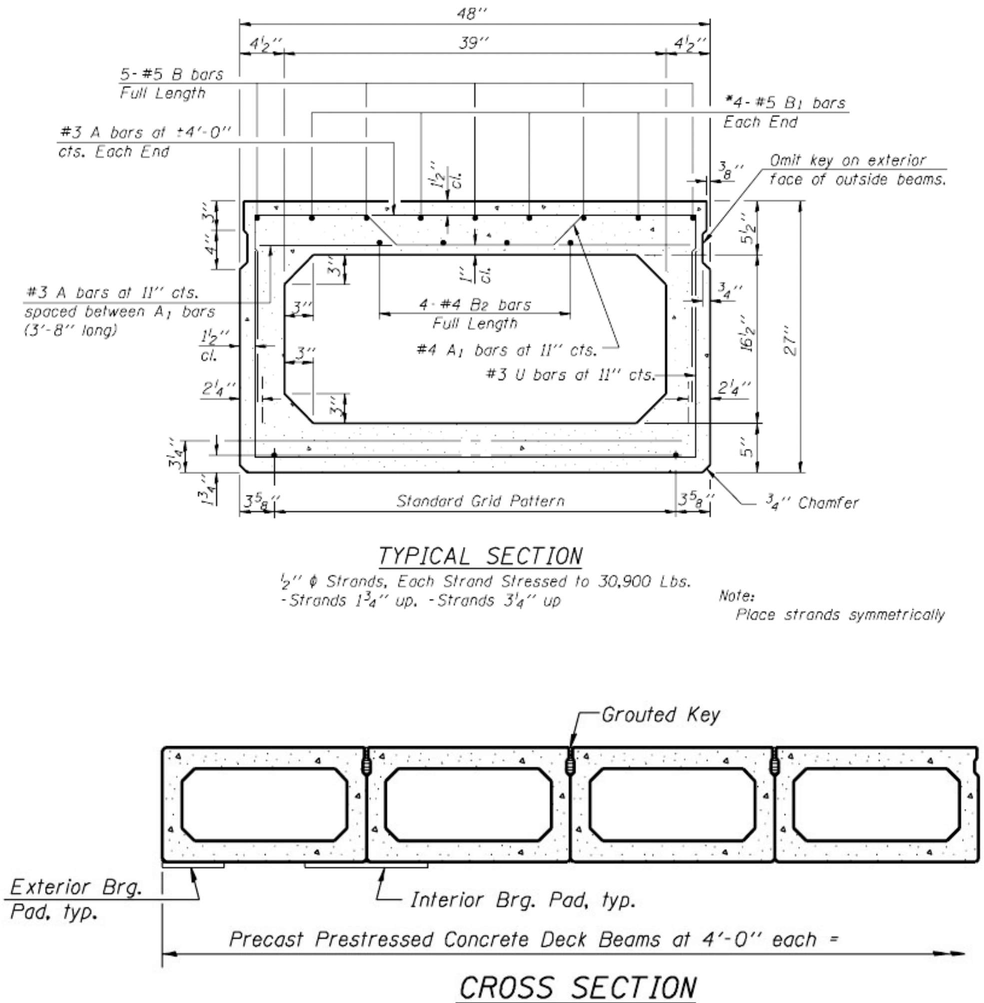


Figure 8. Illinois DOT standard concrete deck beams

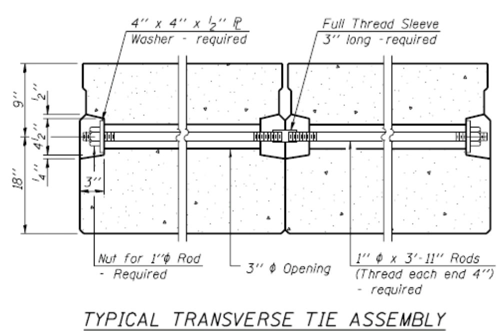


Figure 9. Transverse tie assembly for precast concrete box girders

INSTRUMENTATION AND MONITORING

Instrumentation will be installed and monitored by researchers from the Bridge Engineering Center at Iowa State University to document and aid in the understanding of these two bridges. The final instrumentation plan cannot be determined until the final specific details of the bridges have been determined. However, a number of key components of the two bridges will be monitored to understand both the overall bridge and individual component behaviors throughout an extended period of time, which will cover more than a full annual thermal cycle.

Differential deflection and load transfer between precast concrete superstructure elements is a primary concern of the research team. The deck panels used on the Boone County bridge and the precast concrete box girders used on the Madison County bridge will be instrumented to measure both flexural strain in these components as well as any slip between adjacent deck units.

The connection between the superstructure and substructure components, as well as between the precast substructure elements and the steel piles, will be monitored to assess the degree of structural continuity between these elements. Monitoring will be performed using both strain gauges and deflection or rotation transducers.

A series of controlled load tests, using a tandem axle dump truck of known weight, will be performed to develop a baseline understanding of bridge behavior. Following an extended period of bridge usage by the public, another round of load testing may be performed to assess any change in performance.

SCHEDULE

The Iowa DOT Office of Bridges and Structures is currently performing the final design on the two bridges. The current schedule indicates a January 2006 letting date, with construction to be performed during the 2006 construction season. Structural monitoring instrumentation will be installed and load testing will be performed following completion of construction.

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Surface Transportation Weather Forecasting and Observations: Assessment of Current Capabilities and Future Trends

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ABSTRACT

A significant increase in the awareness of the need for improved surface transportation weather services has occurred over the last five years. During this time, a coordinated national effort has been undertaken to highlight the unmet weather needs of the transportation community and identify research requirements and implementation strategies for improved weather services. The benefits of such technology improvements will include improved safety, efficiency, and capacity of the national surface transportation system. Because of the activities mentioned above, there will be an acceleration of research, development, and implementation of technologies focused on improving surface transportation weather services. In the next several years, the surface transportation system will move away from being primarily a reactive system to being a proactive system with respect to weather. The rapid rise in awareness of the impact of weather on the transportation system and the new relationships that are developing between the weather and transportation communities provide a significant opportunity for improving surface transportation weather services. Active participation by end users in defining surface transportation weather service needs will contribute to and help accelerate the development and implementation of new capabilities.

Key words: forecasting—modeling—observations—weather

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

ENVIRONMENTAL SENSOR SYSTEM MEASUREMENTS

Air Temperature, Dew Point, and Wind

Air temperature, dew point, wind speed and direction measurements are relatively mature, so there is little room for significant improvement. Air temperature measurement accuracy should be within 1 ^{o}F and dew point within 2^{o} to 4^{o} F. Wind speed measurement accuracy should be within 2 miles per hour and wind direction accuracy should be within 5 degrees (measurement range is from 0 to 360 degrees). When comparing these measurements, one should be aware that variations can exist over short distances (tens of feet) and with time (minutes). Differences can be expected over short distances due to local variations in terrain, elevation, shading, ground cover, and proximity to obstructions such as buildings and water bodies. Differences can also vary over short time periods, especially during periods when moisture typically changes rapidly, such as during frontal passages and periods of precipitation.

Visibility

Visibility is typically measured using an optical system that looks for light attenuation between a transmitter and receiver separated by 12 to 30 inches. An algorithm is used to correlate the amount of light attenuated and the visibility.

Several factors can impact visibility measurements. A dirty sensor lens and/or moisture on the lens can be misinterpreted as lowered visibility. A slowly failing transmitter (dimmer light source) can also be misinterpreted as lowered visibility. It is important to keep the lenses clean and follow the manufacturer's maintenance recommendations.

The most significant limitation of a visibility sensor is that it only measures the air between its transmitter and receiver. If visibility conditions vary greatly in the area, the reported visibility may not be representative of the conditions in a broader region. If a visibility sensor is being considered for a site that is prone to fog, care must be taken to site the instrument in the area most prone to fog. In addition, the sensor height should be close to the height of a truck cab (~6 feet) so that the visibility reported by the system corresponds to the condition in the driver's line of sight.

For road applications, it is more important to measure visibility below one mile. Visibility sensors should be accurate to within 50 feet and many manufacturers indicate accuracy to within 35 feet.

Visibility measurements are critical for fog detection and diagnosis. The lack of visibility measurements is currently a major constraint on the development of high-resolution surface visibility products. The addition of visibility measurements at environmental sensor system (ESS) sites is encouraged, as they will provide a much needed data source for future surface visibility products.

Winter Precipitation

Winter precipitation (snow, ice, sleet) is difficult to measure accurately using automated systems. The standard method for observing precipitation is to use a gauge consisting of a collection container and a device or scale to determine the amount of precipitation that falls through the orifice. This technique has been employed for several hundred years and continues to be used today, although there have been distinct improvements in the instrumentation used to make the measurements. Most of the gauges in operation today were developed for climatological analysis purposes.

The majority of precipitation gauges used today are not well-designed for measuring winter precipitation, particularly snow. There are several reasons for this. The first is that snow tends to stick to the sidewalls of the gauge orifice rather than falling into the gauge, as rain does. This results in a significant under-measurement of precipitation during the period when it is snowing, and a false over-measurement of precipitation when this snow melts and later falls into the gauge. In extreme cases, snow can actually cap over the opening of a gauge. The second reason that snow is more difficult to measure than rain is that snowflakes are generally less dense than raindrops and are affected more by the distortions in airflow around the gauge. When air encounters an object, such as a gauge, it tends to flow around the object. Adding wind shields around a gauge can reduce this problem. A considerable effort has gone into determining the most effective wind shield. Studies have shown that improperly shielded gauges may record less than 20% of the true precipitation when wind speeds are on the order of 20 miles per hour.

The accumulation of light snow is more difficult to observe because the precipitation rates for light snow are considerably less than for light rain, yet the reporting increment for the liquid-equivalent amount remains the same, at 0.01 inches.

The National Weather Service (NWS) is currently replacing the tipping bucket gauges with improved weighing gauges. These new gauges will be more sensitive to light precipitation and, because they don't have to tip before they report new accumulations, they will provide better real-time information.

The NWS also uses an optical device called the light-emitting diode weather identifier (LEDWI) at automated surface observing system (ASOS) sites to determine precipitation type. LEDWI is currently able to distinguish between rain and snow at precipitation rates greater than 0.01 inches, although during windy conditions (>25 knots) a vibration develops that causes rain to be reported as snow and snow to be reported as rain. The NWS is currently involved in a system procurement program to replace or upgrade the LEDWI systems.

ASOS stations also have icing sensors that are able to tell when freezing precipitation, freezing fog, or frost is occurring. Currently, the NWS reports freezing rain when the icing sensor indicates icing and LEDWI says rain is occurring. However, freezing drizzle is not reported. ASOS does report freezing fog, but the report is not based on the observation that ice is detected on the icing sensor, but rather freezing fog is reported when the temperature is at or below freezing and the visibility is less than 5/8 mile. The lack of accurate, dependable precipitation detection has been a national issue since the NWS began its automation program; therefore care must be taken when interpreting these data.

During the past 10–15 years, state and local transportation departments have purchased and installed roadside environmental sensing systems, and many include precipitation identification (yes/no) sensors. Some installations include precipitation type measurements using optical devices. Roadside ESS sites do not measure the liquid equivalent precipitation rate, which is critical for winter maintenance operations and could provide data important for assessing flooding and washout risks. Research conducted for the FAA and airlines concluded that liquid equivalent precipitation information is required for effective aircraft anti-icing, as the water amount determines when chemical deicing material will fail due to dilution. Dilution of chemicals used for roadway winter maintenance is also a major factor for determining snow and ice control strategies, but little has been done to integrate real-time liquid equivalent information into the winter maintenance decision process.

There is considerable variation in the precipitation identification sensors used by various companies. At present, little is known about the overall quality of these measurements, but most precipitation sensors do well except during critical periods of mixed precipitation, very light precipitation, and windy conditions.

Video Imaging

Video cameras are being installed along roadways at a rapid pace. Video data (still and streaming) are being utilized to observe traffic, incidents, weather, and road conditions. Research has been conducted and is accelerating on ways to develop image processing techniques designed to derive weather and road condition information automatically from video images. Fixed camera sites provide a better data set for post-processing than adjustable sites (e.g., pan-tilt-zoom). In the future, it is likely that weather and road condition information will be derived from fixed camera images. The Federal Highway Administration (FHWA) began a research project in mid-2005 as part of the Clarus Initiative (Pisano, Alfelor, and Pol 2005) to develop algorithms to derive weather information from fixed video images. The Massachusetts Institute of Technology Lincoln Laboratory is leading this research effort.

Pavement Temperature

Several studies of pavement temperature have been conducted over the last decade with varying results. The uncertainty in results prompted the Aurora Program to conduct a detailed evaluation of the accuracy of pavement temperature measurements (Aurora 2005). The study used temperature sensors attached to the surface of the pavement and compared the output with data from the sensors embedded in the pavement (e.g., pucks). Under ideal laboratory conditions, the average error of in-pavement sensors should be less than 2^{o} F. During periods of rapidly changing conditions, including periods of high solar loading, the embedded pavement sensors are less accurate because the thermal properties of the sensors differ from the pavement and because the sensors are embedded below the pavement surface where heat and cold can be trapped. Under these conditions, temperature differences can easily exceed 10^{o} F. These results are consistent with similar studies and indicate that great care must be taken when interpreting pavement temperature. The good news is that pavement temperatures should be more accurate during poor weather conditions (cloudy and wet), when solar effects are minimal.

Pavement temperature prediction models are generally configured to predict the surface conditions of the pavement (top 1/16th of an inch), while the pavement sensors are embedded in the pavement. Because of the uncertainty in pavement temperature accuracy, one must be very cautious about using the pavement data to rate or score pavement temperature forecasts. Pavement prediction accuracy should not be expected to be more accurate than the measurement accuracy.

It is important to measure the topmost layer of the pavement. This is one reason why thermistors, applied to the pavement surface, are often used to indicate “truth.” The accuracy of these devices, rapid response rates, and inexpensive costs make them very attractive pavement temperature sensors. It is anticipated that the use of inexpensive thermistor technologies will grow and be used to provide pavement temperature information between fully instrumented ESS locations.

Pavement Condition

A number of manufacturers provide pavement condition information. Road surface conditions reported by these in-pavement devices generally include dry, moist, wet, chemical wet, chemical concentration, frosty, snowy, and icy. The most common technique used to diagnose road condition and chemical concentration is to measure electrochemical conductivity. A limiting factor of using this technique is that the type of chemical (NaCl, $MgCl_2$, $CaCl_2$, etc.) is not known, so when the system reports a certain chemical concentration, the user does not know from the device the chemical that is contributing to the report; thus, there is uncertainty in the effectiveness (melting capacity) of the chemical. Devices that measure chemical concentration using electrochemical conductivity techniques are considered passive. This means the device obtains information by reception and does not process the information.

The primary reason for measuring chemical concentration is to determine the freeze point temperature of the roadway. The freeze point temperature is a function of the chemical concentration on the road. Rather than diagnosing the freeze point temperature from chemical concentration measurements, it is better to directly measure the freeze point by lowering the temperature of the device and measuring when the solution freezes. Devices that directly measure the freeze point temperature are considered active. This means that the device obtains information by action (i.e., lowering the device temperature).

Active sensors provide a more accurate freeze point measurement because it is measured directly. However, the sensor is still prone to problems, as the sensor is in the pavement and can only take measurements in a small area that may or may not be representative of the predominant condition of the roadway. Active sensors also produce waste heat as they cool. The waste heat has the potential to impact the accuracy of the pavement temperature measurement if the sensors are collocated in the same device.

There are some new sensors reaching the market that are based on optical and thermal characteristics of the pavement contaminant. These sensors measure thermal conductivity, electrochemical polarization, and surface capacitance to determine whether water, snow, or ice is on the pavement. These technologies are only now emerging in the road weather marketplace; little is known about their overall skill and durability. They are also more expensive than in-pavement sensors, but the price is expected to drop as they become more widely used. Because remote sensors measure a broader area of the roadway, they have great potential for diagnosing the predominant condition and should be considered for high-risk sites (e.g., bridges and other areas prone to icing). These devices hold promise for broader application, but additional testing is required to assess their performance.

WEATHER AND ROAD CONDITION PREDICTION CAPABILITIES

Weather Prediction Modeling

Predicting weather at road scales (a few miles to tens of miles resolution) pushes the limits of weather predictability. While weather forecast models are becoming more sophisticated, the ability to know the current state of the atmosphere in three dimensions around the earth and predict future conditions remains a significant challenge. The primary shortcoming is the lack of global observations (surface and aloft) at the resolutions required to support new high resolution weather models.

Weather models have come a long way over the last 30 years, but until recently they all depended almost entirely on data collected twice a day from balloons (called rawinsondes) released around the globe. Across the nation, the balloon sites are about 250 miles apart. These balloon data are sparse over unpopulated land areas, almost nonexistent over the oceans, and unreliable from undeveloped countries.

Given these limitations, it is obvious why weather models have traditionally had difficulty predicting the weather. The models simply do not know very much about the state of the atmosphere over most of the earth, and they have to guess at the weather conditions between observations. The situation is changing for the better, thanks to faster computers, better observation systems, and more reliable communication technologies, but it will take several years before the new data can be fully incorporated and utilized in weather modeling systems.

Another factor that limits weather forecasting skill is that traditional weather models, including operational models currently being used by the NWS, do not initialize with any clouds or precipitation. The models themselves generate clouds and precipitation, and it takes a few hours of model run time for the clouds and precipitation to develop. This means that even at the start of the model run, there can be great differences between observed clouds and precipitation and the predicted conditions. The reasons for which models have traditionally started dry are complex and beyond the scope of this document, but the situation is improving. Techniques are being developed and tested in research models that allow models to begin with clouds and precipitation data based on observations. This is discussed in more detail later in this paper.

Weather models also need to make assumptions about the characteristics of the land, including vegetation type, greenness fraction, land use, snow cover, and soil moisture. This was traditionally determined using seasonally averaged data, and there was no daily feedback in the process. This means that if snow cover was not normal for a particular day, but it did snow the day before, the weather model would be initialized without the knowledge that snow was on the ground. The lack of updated land surface information reduces the forecast skill. The good news is that this problem is being mitigated, as land surface models are now being coupled with weather models. It is anticipated that most weather models will have this capability within the next two years.

Maintenance decision support system (MDSS) research (Mahoney and Myers 2003; Pisano, Stern, and Mahoney 2005) has shown that predicting weather and road conditions requires weather data at an hourly resolution to properly characterize rapidly changing conditions associated with sunrise, sunset, frontal passages, and precipitation episodes. Road temperature models are particularly sensitive to the solar cycle, as road temperatures rise and fall quickly at dawn and dusk, respectively. The temporal resolution of weather model data provided by the NWS is only three hours. Therefore, anyone using the standard NWS models will only be able to provide forecast information at this temporal resolution. They may provide hourly output by interpolating, but the true resolution will remain three hours. It is likely that the NWS will eventually disseminate selected weather parameters at hourly resolution, but the timeframe for this is unclear.

Commercial weather providers that operate their own weather prediction models tend to perform better, as they have the flexibility to modify model characteristics to optimize the systems for specific applications, including generating hourly output data.

New Data Sources

Data assimilation is the process by which disparate data are gathered, processed, and readied for inclusion into weather models. The incoming data must be valid and incorporated into the models in a way that preserves the laws of physics. Much research is being conducted to develop methods and techniques that allow weather models to utilize high-resolution weather observation data sets, such as radar and satellite data. These data, as well as data from global navigation satellites, ground observation systems, and buoys, have great potential to improve weather modeling, but these data have only recently been incorporated into research models; therefore, it will be a few years before they are incorporated into the operational NWS models and the benefits are fully realized.

Forecast Skill

Short term forecasts (0–48 hours) have improved significantly over the last decade as operational and research weather models have improved, but beyond 48 hours, the forecast skill drops. The most difficult parameters to forecast are visibility and precipitation timing and amount, which are critical elements for surface transportation. When it comes to winter precipitation prediction, the forecast skill drops significantly after only 24 hours. For summer precipitation, namely thunderstorm prediction, weather prediction models have difficulty after only a few hours. The models predict conditions conducive to thunderstorms reasonably well, but they are still unable to predict exactly when precipitation will occur and how much precipitation will fall. This is mainly due to the fact that thunderstorms alter their immediate environment on very small scales by creating cold downdrafts and gust fronts, which change the stability structure around the storm. Because weather models do not know exactly where the updrafts and downdrafts are in each storm, they have difficulty evolving the structure forward in time.

Visibility is also difficult to predict because several factors influence visibility, including sun angle, relative humidity, water droplet size and concentration, and precipitation type and rate. Local effects, such as proximity to water bodies, hills and valleys, air pollution sources, and land use, also affect visibility. Each of these factors is difficult to predict individually, let alone combined into a visibility prediction.

Wind and temperature are probably the best predicted weather elements, particularly if statistical corrections are made to the data. However, during periods of rapid changes, the timing of wind shifts and temperature changes can be incorrect.

Forecasting the amount and distribution of water vapor in the atmosphere remains a significant challenge, as there are few observations of water vapor above the surface. The distribution of water vapor both horizontally and vertically has a major effect on the formation, evolution, and dissipation of clouds and precipitation. New data sets from weather satellites, global navigation satellites, aircraft, and surface-based observations will improve the situation, but it will likely be a decade or two before a sufficient quantity of high-resolution data sets around the earth will routinely be available to support high-resolution weather forecasting models.

High-Resolution Models: High-resolution models (also called meso-scale models) better predict the characteristics of a storm (e.g., whether the storm will be a single cell, multi-cell, line of cells, contain precipitation bands, etc.), but they still have difficulty on the timing and amount of precipitation. One must remember that high-resolution models are initialized at the beginning of the forecast run using data from low-resolution national-scale or global-scale models. If the larger-scale models are wrong, the high-resolution models will be wrong. If the larger-scale models are correct, the high-resolution models will generally provide better information about the structure of the weather systems.

Faster computers and the availability of new and more frequent observations allows models to be run more frequently, which provides opportunities for more frequent forecast updates and improved skill. The ability to provide more frequent updates will certainly aid decision makers.

Ensemble Modeling

It is well-known in the weather community that different weather models predict the weather with different skills. For example, some models are better with temperature, while others are better with precipitation. Some may be better with summer precipitation (thunderstorms), while others are better with winter storms. Faster and less expensive computers have provided the opportunity for meteorologists to run multiple models at the same time and analyze the results. The resulting outputs from each model are blended to optimize the overall forecast and to assess the predictability of a particular weather situation. If multiple models are used and the data are intelligently blended, there is a higher probability that the overall forecast will be improved over any of the individual models. This technique was demonstrated for the surface transportation community as part of the MDSS project.

Methods and techniques are being developed and debated in the scientific community for extracting forecast confidence and/or probability information from model ensembles. Risk management decision makers that rely on weather information have expressed a strong interest in obtaining forecast confidence information. In what format the information is provided to users and how it is interpreted is another area of active research.

Road Temperature Prediction

Numerous road temperature models have been developed and are utilized by the road weather community. Some of the models have been developed openly at universities and national labs, while others have been developed by the private sector and are proprietary. The majority of the models used operationally are heat balance models that predict the temperature profile from the pavement surface down several feet into the subsurface layer. The major differences are in how the models handle precipitation, snow and ice accumulation, and subsurface moisture. There is no simple way of judging the skill between the models, as there has not been any comprehensive scientific review of the available models, primarily due to the proprietary nature of the models used in the industry. A review of the literature suggests that, in general, the models' performance is similar when given similar weather and road characteristic data.

The best ways to improve road temperature predictions are to 1) provide the road temperature model with more accurate weather data, 2) ensure that the model includes accurate road characteristic data, and 3) initialize the model with observed pavement surface and subsurface temperature data. Some vendors use generic road characteristics for their road temperature model while others incorporate actual as-built data. The use of as-built data is preferred.

Road temperature models that utilize direct and indirect solar radiation from the weather models perform better than models that use cloud coverage data from weather models. Road temperature prediction is highly sensitive to predictions of cloud amount and depth and the time of day the clouds appear. Because weather models are gridded, they can only generate cloud characteristics at each grid point. A course model grid (>10-mile grid spacing) will smooth cloud characteristics. For example, the model cannot generate a single fair weather cloud or thunderstorm on scales less than 10 miles. A finer model grid (< 10 miles) will have a better chance of defining cloudy-clear boundaries, but smoothing will still take place. Because the prediction of individual clouds is difficult, road temperature predictions will be prone to errors during partly cloudy conditions both in the day and night.

Road temperature prediction accuracy is best during cloudy (widespread overcast) conditions and/or when precipitation is occurring. The predictions are worse during clear days when solar influences are greatest and clear nights when radiational cooling is strong. Because the in-pavement sensors are prone to errors during similar conditions, it is hard to state with confidence the absolute errors that can be expected with pavement temperature predictions. Another complicating factor is that the statistics used to report forecast errors differ between studies and vendors, making it hard to identify typical or expected errors.

During cloudy conditions, the average difference between the measured pavement temperature and the prediction should be within 3^{o} F. During clear days, the average temperature difference should be within 10^{o} F, with the highest difference near noon. There appears to be no consistent bias in pavement prediction, as some models tend to be slightly colder and some slightly warmer than the observations. The differences are likely to be related to the different weather models used to drive the pavement models, the accuracy of the pavement characteristic data used by the pavement temperature model, and the way the data are handled within the model. Of course, different pavement sensors will have different biases as well, which makes it difficult to determine the source of the error.

Road temperature prediction systems can be designed to minimize the difference between the observations and the predictions, but because of the measurement uncertainty of the in-pavement sensors, this technique may not result in the most accurate pavement surface prediction, only a good match between predictions and measurements. Strict pavement temperature quality control procedures must be implemented before any statistical corrections to pavement temperature models are made, as poor-quality data could degrade the output.

Blowing and Drifting Snow Prediction

As experienced winter maintenance personnel know, blowing and drifting snow have a major impact on winter maintenance operations. Weather models do not explicitly predict blowing and drifting snow, but do contain data that can be used to diagnose blowing and drifting snow conditions. Whenever it is snowing and there is wind, there will be blowing snow. For the sake of discussion, we will concentrate on conditions that cause snow to blow and drift after the precipitation has ended. Whether snow will blow around depends primarily on wind speed, the age of the snow, whether it has rained since it last snowed, whether the air temperature has risen above freezing since the last snow, the type of snow that originally fell (dry or wet), and the characteristics of the land surface that is holding the snow.

General alerts about blowing snow conditions can be generated using recent and predicted weather data, but it is very difficult to predict the amount of snow that will be moved without a very high resolution (<30-foot grid resolution) snow drift model. Local land characteristics (hills, valleys, road cuts, vegetation type and height, etc.) will have a major influence on the amount of snow moved, and these data are difficult to find and update in real-time prediction systems. Snow drift models are available, but they require a lot of pre-winter season configuration to ensure they have properly captured the local environment. Snow drift modeling should be considered for areas that are prone to drifting, as the models can identify the location and direction of drifting and estimate the amount of snow that will move.

Road and Bridge Frost Prediction

Whether or not road or bridge frost will form depends primarily on the pavement temperature and dew point temperature. In addition, any residual chemical will reduce the likelihood of road frost. As mentioned previously, one of the most difficult weather elements to predict is water vapor, which is reflected in surface dew point measurements. The complexities of predicting pavement temperature were also described. As with most forecasts, near-term forecasts will generally be more accurate than longer term forecasts; therefore, there is hope that as weather and pavement predictions improve, so will road and bridge frost forecasts. In the meantime, care must be taken when interpreting frost forecasts, as an error of only a fraction of a degree in either the pavement temperature or dew point temperature can make a large difference in the prediction of frost.

Because of the uncertainty inherent in these predictions, a better approach would be to obtain information about frost potential in probabilistic or confidence terms, rather than trying to explicitly attempt to determine the timing and amount of frost buildup.

SUMMARY

There has been a significant increase in the awareness of the need for improved road weather services over the last five years. A coordinated national effort has occurred to highlight the unmet weather needs of the transportation community and identify research requirements and implementation strategies for improved weather services. The atmospheric science community is now actively engaged with the transportation community in a partnership that will accelerate technological improvements. Several key organizations are working together on road weather issues including the American Meteorological Society, FHWA, Intelligent Transportation Society of America, National Oceanic and Atmospheric Administration, National Science Foundation, national laboratories, Aurora Program, National Aeronautics and Space Administration, Transportation Research Board, universities, and commercial weather providers. The benefits of the technology improvements will include improved safety, efficiency, and capacity of the national roadway system.

Because of the activities mentioned above, there will be an acceleration of research, development, and implementation of technologies that will improve road weather services. In the next few years, the surface transportation system will move away from being primarily a reactive system to being a proactive system with respect to weather. The rapid rise in awareness of the impact of weather on the transportation system and the new relationships that are developing between the weather and transportation communities, provide a significant opportunity for improving road weather services. Active participation by end users in defining road weather service needs will contribute to and help accelerate the development and implementation of new capabilities.

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Hybrid Interchanges: Developing the Arterial of the Future

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ABSTRACT

Urban arterial roadways are one of the most important facilities provided for the public. To be beneficial, they should be designed to move high volumes of traffic safely at reasonable speeds and provide needed access to businesses and residences. This is the challenge roadway designers have before them. The ultimate goal of arterial design has been to develop a facility that will do all those things, do them well, and do them at a reasonable cost. The ideas presented in this paper suggest that this goal may be nearing fulfillment.

The weak point of arterial design (or the linchpin, depending on the perspective) has been the arterial/arterial intersection. Extremely high volumes, both through and turning, come together at the same point at the same time, and all of these substantial traffic demands need to be served in the best manner we know how. Because of these conflicting needs, intersections have served as bottlenecks to the flow on arterial roadways, providing only 30–50% of the capacity available on the arterial itself. To improve arterial intersections, a widened intersection with turn lanes has typically been provided, controlled by a traffic signal. The alternative to this has been the consideration of a grade-separated interchange. For a congested arterial/arterial intersection, the at-grade solution has typically been too little, while the grade-separated solution has typically been too much. What follows is a concept that the authors hope practitioners see as a method for improving operational efficiency and utilizing untapped capacity on arterials.

Key words: arterials—arterial intersections—hybrid interchanges

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BACKGROUND

Intersection and interchange designers are commonly faced with the issue of comparing alternatives to improve congested intersections. The alternatives commonly involve the comparison of widening the intersection vs. creating a grade-separated interchange. Both alternatives are likely to involve a major widening of the project's footprint. That commonly means acquiring additional right-of-way, which adds to the project's cost and impacts.

The hybrid interchange is a concept that utilizes aspects of both at-grade intersections and grade-separated interchanges. The following is a verbal description of a hybrid interchange:

- A hybrid interchange uses grade separation to modify a single at-grade intersection and create two separate intersections, one above the other.
- Each of these two intersections involves the pairing of two of the four approaches to create two separate intersections.
- Each of these two separate intersections proximate the intersection of two one-way streets.
- Drivers at hybrid interchanges are not required to do anything unusual or unexpected.

This particular intersection geometry creates several opportunities, including the following:

- Two-phase signal operation
- Pairing approaches to take advantage of peak and off-peak directions
- Modifying signal splits to take advantage of the traffic flows at the intersection

Although not a solution for every problem, the hybrid interchange shows significant promise regarding substantial improvements in operational efficiency vs. moderate construction costs (less than the cost of typical grade-separated interchanges). Environmental impacts need to be evaluated on a case by case basis, but the preliminary estimates of impacts caused by the hybrid interchange are very encouraging. It is also recommended that a cost-benefit analysis be conducted to determine the relative value of any alternative studied, including the hybrid interchange.

EXAMPLE PROBLEM INTERSECTION IN RENO, NV

The Regional Transportation Commission (RTC) in Reno, Nevada, has an arterial ring-road network named McCarran Boulevard, which surrounds and serves the greater Reno community. The first of 13 congested intersections selected for improvement and testing of various alternatives is the intersection of McCarran Boulevard and Pyramid Way. It was felt appropriate to test a few alternatives to improve this intersection against the hybrid interchange concept to see whether this alternative deserves any additional consideration.

The design year is 2012, and the following are the design hour turning traffic volumes:

	Eastbound	Westbound	Northbound	Southbound
AM Left turns	483	147	200	198
AM Thru	761	654	1064	1998
AM Right turns	124	217	56	1289
PM Left turns	707	91	200	157
PM Thru	905	827	2015	1259
PM Right turns	70	334	153	785

The current design for all legs is essentially a five-lane divided roadway, with dedicated left-turn lanes in all directions, and dedicated right-turn lanes in a few directions. The intersection currently runs at level of service (LOS) F (2004 conditions).

Alternative Solutions

The proposed alternatives for this intersection are as follows:

A. Widened intersection
   1. Thru lanes/direction = 3
   2. Left turn lanes/direction = 2
   3. Right turn lanes/direction = 1 (2 RTL's, SB)
   4. Roadway footprint = 130 ft (approx.)

B. Overpass
   1. Single point urban interchange (SPUI)
   2. East/West thru traffic (McCarran Blvd) = 3 lanes/direction, w/ramps
   3. North/South traffic (Pyramid) same as Alternative A (2 right turn lanes, SB)
   4. Roadway footprint = 160 ft (approx)

C. Hybrid interchange
   1. Thru lanes/direction = 2
   2. Left turn lanes/direction = 1
   3. Right turn lanes/direction = 1
   4. Roadway footprint = 90 ft (approx)

Level of Service Analysis for Alternatives

The LOS analyses for the alternative are as follows:

A. Widened intersection
   1. AM Condition
      a. AM LOS = E (Almost F)
   2. PM Condition
      a. PM LOS = E (Almost F)

B. Overpass
   1. Flyover LOS (East/West)
      a. AM LOS = A
      b. PM LOS = A
   2. SPUI Intersection LOS
      a. AM LOS = D
      b. PM LOS = D

C. Hybrid interchange
   1. Upper intersection LOS (Eastbound/Southbound)
      a. AM LOS = D
      b. PM LOS = C
   2. Lower intersection LOS (Westbound/Northbound)
      a. AM LOS = C
      b. PM LOS = D

COST, RIGHT-OF-WAY, NEIGHBORHOOD, ENVIRONMENTAL, AND OTHER IMPACTS

The cost, right-of-way, and environmental impacts are going to vary substantially from location to location, as well as impacts to neighborhoods or businesses. Any preferred alternative needs to be reviewed carefully in an isolated manner, taking into consideration all potential costs and impacts.

Also, the system costs and impacts need to be identified and compared, and will be affected based on the intersection/interchange that is selected to solve a given problem. The implications of choosing an intersection/interchange improvement that requires a six-lane divided in lieu of a four-lane divided facility should be examined carefully.

For instance, the following possible choices are likely, and system decisions need to follow:

A. Choice = Widened intersection (six-lane, divided)
   Implication = Widened arterial (six-lane, divided)

B. Choice = Overpass with SPUI
   (1) Implication = Widened arterial (six-lane, divided)
   (2) Implication = Non-widened arterial (four-lane, divided)

C. Choice = Hybrid interchange
   Implication = Non-widened arterial (four-lane, divided)

The implications are obvious, and making the decision for the preferred alternative at a selected intersection affects other system-wide decisions (which may be more far reaching). The city may save money because an at-grade widened intersection was selected over the cost of a grade-separated interchange or a hybrid interchange, but the city may need to face the implications (cost, right-of-way, neighborhood and business impacts, and environmental impacts) of the broader scale decisions to widen the arterial from four lanes to a six-lane divided facility.

Other implications may be equally obvious. If a city faces a similar situation to that confronting the RTC in Reno, NV, the alternatives should be considered on both an isolated and system-wide basis. The impacts at the intersection/interchange being studied and the impacts on the arterial away from the affected intersection/interchange have been discussed. In addition, the RTC is looking at this location to set the stage for improvements to 13 similar intersections along McCarran Boulevard. It would be appropriate to test not only various treatments to the affected intersection/interchange, but to the arterial and the 13 other similar intersections as well. Only when the system implications are examined does it become possible to understand the impacts and implications of selecting any one treatment over another.

Anticipated Benefits

A ballpark estimate of benefits for the intersection described above, when converted to a hybrid interchange, is as follows:

Annual time and fuel savings	= $6.3M/year
Annual air quality impacts	= 450,000 lbs reduction in CO/year
	= 50,000 lbs reduction in HC/year
	= 36,500 lbs increase in NOX/year (idling decrease)

According to these figures, it would require one to two years of benefits to pay back the cost of construction.

System Implications on the Arterial and Freeway Network

As mentioned above, making decisions for individual intersections can have system implications, whether they involve arterial corridors, arterial systems, freeways, or transportation within urban areas. If building hybrid interchanges eliminated the need for expanding arterials, and if the added capacity on the arterials resulted in a reduced need to expand freeways, the financial impacts of widening those facilities is significant. The costs of converting intersections to hybrid interchanges should be weighed against the costs of not widening corridor facilities that are in place (because the arterial per-lane efficiencies are anticipated to increase 50–100%). After testing the impacts of hybrids (using traffic simulation), the current arterial roadways may not need to be widened. Since much of projected travel volume diverts from congested arterials to freeway corridors, the impacts on freeways also need to be tested (again, by traffic simulation). So, it may be possible that substantial construction might not be required on either arterials or freeways if hybrid interchanges were implemented in a systematic way. The following are hypothetical (theoretical) construction cost savings, which can illustrate these implications:

A.	Arterial corridors (not widening)	=	$10-20M/mile
B.	Arterial systems (not widening)	=	$20-40M/sq. mile
C.	Freeway systems (not widening)	=	$50-100M/mile
D.	Urban areas (arterials and freeways)	=	$30-60M/sq. mile

Another way of looking at this is for every 10 lanes of arterials for which capacity is increased by converting to hybrid control, the equivalent of 3 additional lanes of freeway capacity are created.

HYBRID INTERCHANGES: A NEW GEOMETRIC FAMILY?

In performing this analysis, it became evident that hybrid interchanges are likely not something new. In looking around the United States, there are several examples of hybrid interchanges (in multiple geometric forms). These were created to respond to the conditions at a specific location and were designed to perform a specific function. The keys to being a hybrid are to use vertical separation (grade-separation) and still use at-grade signals (or other treatments as may be appropriate) to help resolve various transportation problems. It is possible that hybrids may become a preferred option to resolve various problems, rather than a make-do option, because nothing else can be constructed to resolve a problem within given cost and right-of-way restrictions.

CONCLUSION

When brainstorming alternatives for improvements to arterials operating at very congested levels, it may be valuable to “think outside the box” and develop alternatives that are not depicted in standard manuals. Many of these ideas sound outlandish initially, but can eventually be accepted once they are constructed and proven. We sometimes forget that everything that is in current manuals and standards was originally developed as a new idea. All new ideas are met with skepticism and doubt, which is normal, but an idea’s time has come, practitioners will recognize it for what it is, and can weigh the advantages/disadvantages for themselves. Two relatively new concepts were seen in that light recently, but are rapidly gaining acceptance: the SPUI and the roundabout. Perhaps it is time for another concept: the hybrid interchange.

Safety and Design Alternatives for Two-Way Stop-Controlled Expressway Intersections

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ABSTRACT

Expressways are a quickly growing segment of the U.S. highway system, and their growth is planned to continue into the future. Understanding the safety performance of two-way, stop-controlled rural expressway intersections will help corridor planners and designers include special consideration in their plans and designs for accommodating future intersection safety improvements. Such an understanding will allow the operators of existing expressway corridors to plan for accommodating special safety treatments as traffic volumes grow and as adjacent land uses change. Planning for these changes is important, as the analysis shows that intersection crash rates, crash severity rates, and fatal crash rates increase with minor roadway volume.

Keywords: divided multi-lane highways—expressways—expressway intersection safety—safety performance functions

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

The conversion of two-lane rural highways into multi-lane, median-divided highways with partial or no access control (rural expressways) is one of the fastest growing parts of the country's highway network (Maze et al. 2005). Between 1996 and 2002, the miles of expressways grew nationally by 27%. The popularity of expressways is demonstrated through a recent survey the authors conducted of 35 state transportation agencies (STAs) with the largest expressway systems (Maze, Burchett, and Hawkins 2004). Twenty-seven STAs responded, and 26 said they intended to continue expanding the expressway network in their state over the next 10 years. The reasons for building expressways are clear: expressways often operate at speeds equivalent to those of rural interstates, but cost much less. In comparison to rural interstate highways, expressways do not require the purchase of all access rights, generally require less right-of-way, and require fewer or no grade separations through expensive overhead bridges and interchanges.

It was also found that many of these same STAs are experiencing problematic crash rates at ordinary two-way stop-controlled (TWSC) expressway intersections. Almost all STAs surveyed are experimenting with treatments and design modifications to ordinary TWSC intersections, ranging from special public information and education campaigns to roundabouts and low-cost intersection grade separation strategies.

The purpose of the research described in this paper is to provide a better understanding of the safety performance of TWSC expressway intersections. By understanding the safety performance of TWSC intersections, highway professionals can better identify where safety problems are likely to occur. Under the best circumstances, if the conditions that result in the most problematic intersections are known during the corridor planning stage of an expressway's development, highway planners and designers can avoid creating these conditions. For existing facilities, conditions that make existing intersections problematic can be identified so that traffic safety engineers can proactively identify where safety treatments should be applied and prioritize the implementation of those improvements.

To understand the safety performance of TWSC intersections, three analyses are presented in this paper. The first is a descriptive analysis using bar charts to illustrate how crash rate, crash severity, and fatal crash rate are affected by increasing intersection traffic volumes. The second analysis presents two safety performance functions (SPF) for TWSC intersections. SPFs estimate crash density (crashes per intersection per year) as a function of intersection characteristics. The third analysis uses an SPF to estimate the expected safety performance of all of the intersections, given each intersection's traffic characteristics. The expected safety performance is then compared to the actual performance. The intersections with the poorest and best actual performances in comparison to expected performance are then examined to determine common characteristics across these intersections and the characteristics that cause deviation from the expected.

In the research report that is the basis for this paper, the STA survey helped identify 17 strategies that STAs are applying, either experimentally or routinely, to reduce crashes at expressway intersections (Maze, Burchett, and Hawkins 2004). Although it is beyond the scope of this paper to report on all 17, the continuum of strategies can generally be grouped into the following 4 categories:

- Intersection recognition strategies. These strategies alert the driver and draw attention to an approaching intersection using improved signage, approach rumble strips, warning signs, beacons, and intersection lighting.
- Speed change lane strategies. These strategies include off-set right- and left-turn lanes and left-turn median acceleration lanes.

- Low-capital–cost geometric strategies. These strategies replace movements that have a higher crash risk with those that have a lower crash risk. These generally involve indirect left-turn strategies (jug handles, loops, and median u-turns). A strategy the authors believe is very promising is shown in Figure 1. This is a directional median with a median u-turn. In this case, the higher crash risk minor road-cross and left-turn movements are replaced by a lower crash risk median u-turn. Although this strategy is commonly applied in urban areas in some states, it is not one generally applied in rural areas, and rural applications are even discouraged by AASHTO's "Policy on Geometric Design of Highways and Streets" (the Green Book) (AASHTO 2001). One STA surveyed has been implementing this strategy at rural expressway intersections and reported excellent safety performance.
- Conflict-reducing high-capital–cost strategies. These strategies include signalization, relocating half of the intersection to create two-tee intersections, grade separating the intersection, constructing an interchange, and grade separating an entire corridor. Figure 2 shows an aerial photo of a grade-separated intersection. Although they are an unusual design, the two Iowa grade-separated intersections reduced the crash risk by one-third of the expected rate at a TWSC intersection with the same entering volumes.

The research report provides more detail on the continuum of strategies and identifies expected benefits and attributes of locations where the strategies are most appropriate.

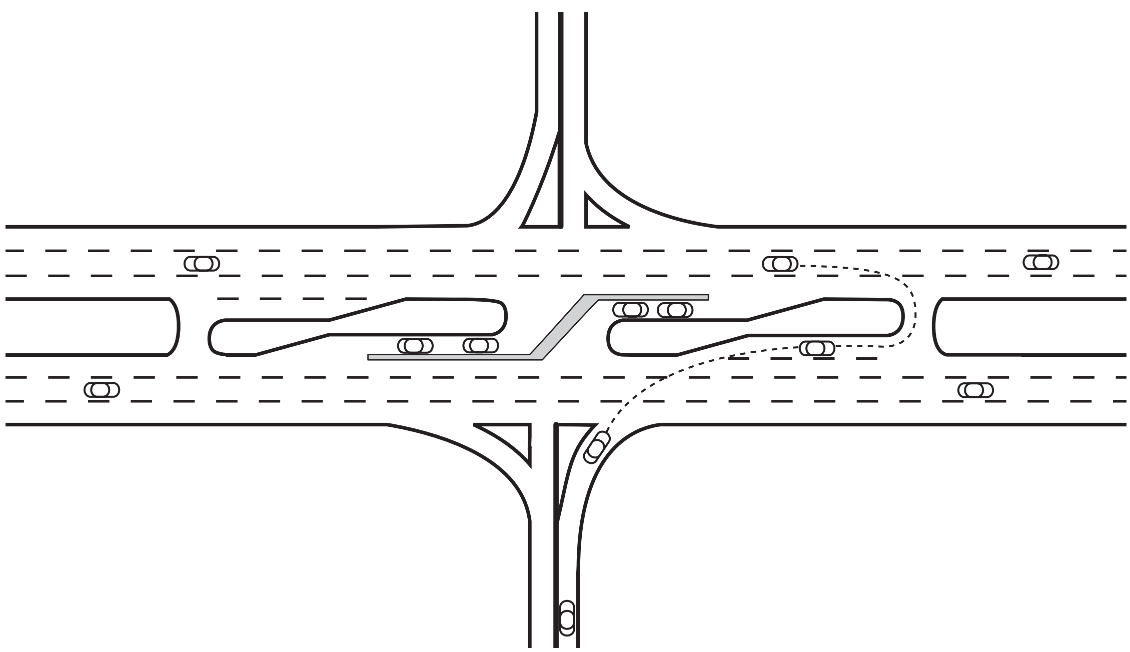

Figure 1. Directional median opening and median u-turn

Figure 2. Aerial photo of grade-separated intersection, US 59 and US 34

DESCRIPTIVE ANALYSIS

The expressway intersection database that was developed to study the safety performance of TWSC rural expressway intersections includes five years of crash data (1996–2000) for 644 intersections on rural expressways in Iowa. The majority of these intersections have very low volumes and many experienced extremely low crash densities (crashes per intersection per year). Of the 644 intersections in the dataset, the minor roadways at 155 intersections are gravel-surfaced. The mean crash rate is 0.15 crashes per million entering vehicles (MEV) and the median crash rate is 0.068 crashes per MEV, indicating that the crash rate distribution is skewed towards intersections with very low crash rates and very low traffic volumes.

CRASH RATE ANALYSIS

In prior work conducted by the authors, it was observed that crash rates on expressways increased with increasing volumes (an upward-sloping safety performance function) (Maze et al. 2005). For example, crash rates on Iowa expressways with a daily traffic volume of less than 7,000 vehicles per day (VPD) averaged a crash rate of 0.79 crashes per million vehicle miles, while expressways with more than 10,000 VPD averaged 1.0 crashes per million vehicle miles. It was also observed that as volumes grew, the percentage of crashes at intersections grew. On expressways with fewer than 7,000 VPD, 21% of the crashes were at intersections, and when the volume increased to more than 10,000 VPD, crashes at intersections almost doubled to 39%. When a similar comparison was made with Minnesota data, where expressways experienced much higher traffic volumes than those in Iowa, the trend of higher crash rates and more intersection-involved crashes was even stronger. The new data from 644 intersections are used to determine the factors driving the trend toward more intersection-involved crashes.

Figure 3 graphs the crash rate, the crash severity rate (a system where the crash severity index for the intersection is divided by MEV per year), and the fatal crash rates for all intersections, summarized by increasing minor roadway volume. A simple severity index is used that applies a weight of five for a fatal crash, four for a major injury crash, three for a minor injury crash, two for a possible injury/unknown crash, and one for a property damage-only (PDO) crash.

It was expected that the average crash rate and the crash severity index would increase as the minor roadway volume increased. In other words, as crossing traffic volumes increase, the crash rate increases and crashes become more severe. The fatality rate also increases as minor roadway volume increases; the fatality rate is calculated using 100 million entering vehicles. Because each of these rates increases across increasing minor roadway volumes, the safety performance of the intersection declines as minor roadway traffic volumes increase. When a similar plot was created and stratified by increasing major roadway volume, the crash rates and severity rates did not tend to increase with increasing major roadway volumes.

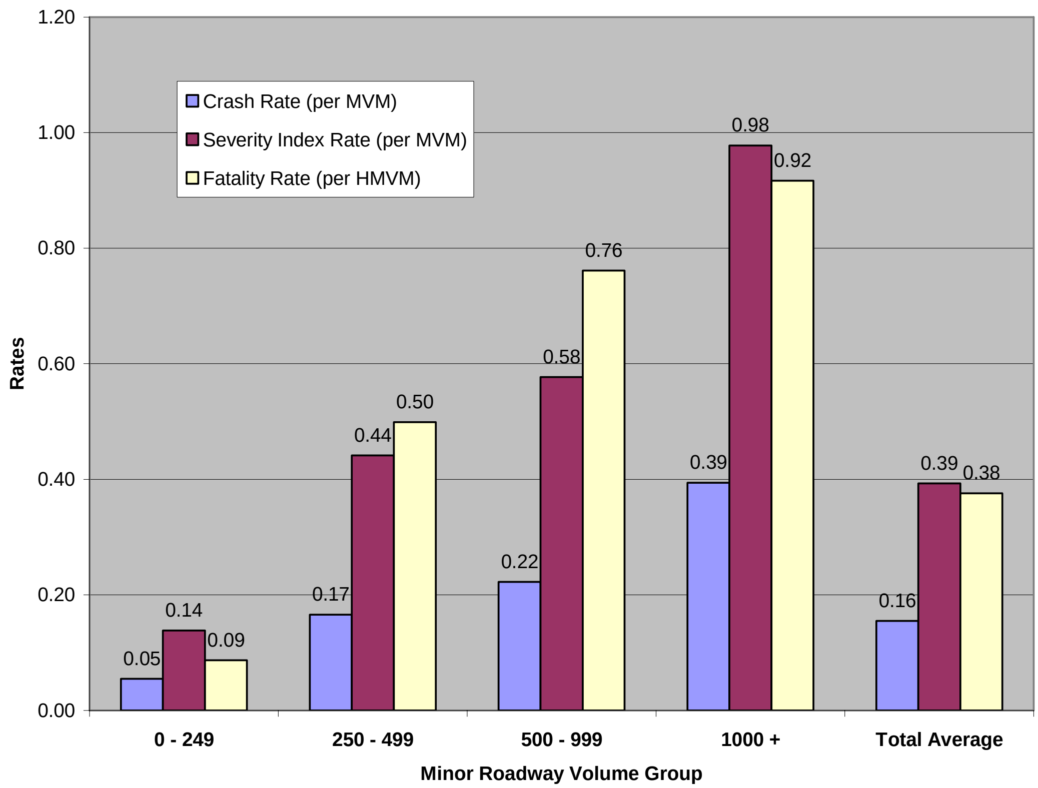

Figure 3. Crash rate, crash severity rate, and fatal crash rate vs. increasing minor roadway volume

Crash Type

To help understand the relationship between crash type and volume on the minor and major roadways, Figure 4 was created. Figure 4 stratifies crash rates by minor roadway volume and excludes all PDO collisions. It was expected that as volumes increase, the proportion of right-angle crashes would increase. Right-angle crashes are generally a result of a driver on the minor roadway approach failing to select an appropriate gap in traffic. Figure 4 shows that the distribution of crash types changes with increasing volume. The proportion of right-angle crashes, therefore, also increases as minor roadway volumes increase. A similar bar chart was developed for increasing major roadway volume, and no increase in right-angle crashes with increasing major roadway traffic volumes was observed. Because right-angle crashes are likely to be more severe, increasing minor roadway volumes result in increasing crash severity, as seen in Figure 4.

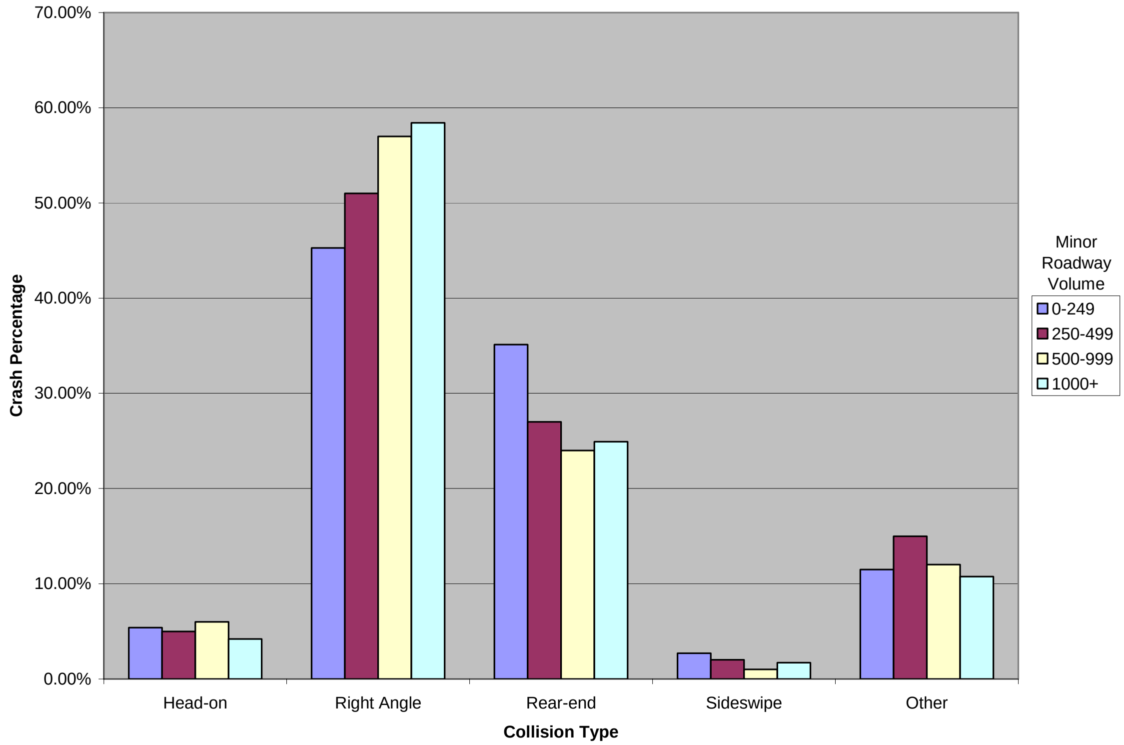

Figure 4. Crash type by minor roadway volume without PDO

Safety Performance Function

This section describes the analysis of the intersection database using maximum likelihood to estimate parameters for a negative binomial SPF. Model parameters were estimated using the software package *LIMDEP Version 7.0*. Given that the dependent variable of the model is count data (crash density = crashes per intersection per year), both Poisson and negative binomial models were considered for the analysis. Generally, crash data suffer from over-dispersion, which is a problem for the Poisson model but not the negative binomial model. Therefore, the negative binomial model was chosen.

Using the 644-intersection database, SPFs were estimated. The SPF generally involves the use of traffic volumes as independent variables and the crash density as the dependent variable.

Next, several regressions were performed using a negative binomial model. The purpose for working with the model is to obtain a general understanding of the relationships between the volumes and crashes. The authors used a Rho-squared value to demonstrate the goodness of fit of the model. Like R-squared, the Rho-squared value varies from 0.0 to 1.0 and measures the model's ability to account for variance in the dependent variable. The closer this value is to 1.0, the better the model represents the data set (similar to an R-squared value). Rho-squared is commonly used when measuring the goodness of fit of a model that has a discrete dependent variable (such as count date) (Ben-Akiva and Lerman 1985). In Equation 1, the numbers in parentheses below each parameter represent the statistical significance of the parameter estimates (p-value). The p-value measures the chance that the parameter estimate is not statistically significantly different than zero. Therefore, the smaller the p-value, the more certain it is that the relationship (the parameter estimate) between the independent variable and the dependent variable is statistically significant.

$$\text{Crash density} = e^{\underset{(0.881)}{0.02278} + \underset{(0.0001)}{(0.00005 * \text{Major ADT})} + \underset{(0.00001)}{(0.00042 * \text{Minor ADT})}} \qquad (1)$$

Rho-squared value = 0.381

As expected, crash density increases with both minor and major roadway volumes. There is a strong statistical relationship between the independent variables and the dependent variable. The relatively low Rho-squared value indicates that there are important unaccounted variables, but the Rho-squared value is very good for this type of analysis. A model that also estimated the product of minor and major roadway volumes was included to test the importance of the interaction between minor and major roadways, but the interaction variable did not improve the model. The interaction term was dropped from further analysis.

In Equation 1, note that the coefficient for the minor roadway volume is nearly 10 times as large as that for the major roadway volume, indicating the stronger impact that minor roadway volume has on increasing crash density. Illustrating the relative importance of minor and major roadway volume, the crash density increases by 0.5% if the volume on the major approach increases by 100 VPD, and the crash density increases by 4% when the volume on the minor roadway increases by 100 VPD. This is interpreted to mean that crash density increases with increasing major road volume and crash rate increases with minor roadway volume.

Crash Severity Index Model

In evaluating the relationship between the intersection variables and crash severity, the intersections that experienced crashes were separated from those that did not. Three hundred twenty-seven intersections experienced at least one crash during the five-year period (1996–2000). Almost half the intersections in the dataset had no crashes. Crash severity was calculated over a five-year period for the remaining 327 intersections. The traffic volumes form the independent variable, while the crash severity index averaged over five years forms the dependent variable. Equation 2 shows the results.

$$\text{Crash severity index} = e^{\underset{(0.001)}{2.10612} + \underset{(0.00001)}{(0.0000688 * \text{Major ADT})} + \underset{(0.00001)}{(.00004 * \text{Minor ADT})}} \qquad (2)$$

Rho-squared value = 0.53

Equation 2 offered the best statistical properties of any statistical model estimated. In other models, other variables were included, such as median width and a dummy variable for the presence of right- and left-turn lanes at the intersection (the dummy variable is set equal to zero when no left-turn lane is present and one when a left-turn lane is present). When these variables were added, the regression resulted in lower Rho-squared values or parameter estimates that were not statistically significant.

Intersections with High and Low Crash Severity Rates

To identify intersections where the safety performance was worse or better than expected, the model of the intersection crash severity index (Equation 2) was used to estimate the expected five-year crash severity index for all intersections that experienced a crash. Inputting the actual major and minor roadway volumes into the model provides an estimate of the safety performance expected. Because some variables are not accounted for in the model, some intersections will perform worse than expected (intersections with a higher crash severity index) and some will perform better than expected. Next, 10 intersections for which the expected severity index exceeded the actual by the greatest amount (intersections with a low crash severity) and the 10 intersections for which the actual severity index exceeded the expected by the greatest amount (intersections with a high crash severity) were identified. The 10 intersections with the highest severity indexes and the 10 with the lowest severity indexes represented extremes in our data set. The next step was to look for characteristics to explain why the intersection's performance deviated from what was expected.

Table 1 shows the average of the traffic volumes on the minor and major roadways for the 10 intersections with the highest and lowest severity rates and for the overall average (all 327 intersections). The authors were surprised to find that several of the intersections with low severity rates were on expressway segments with some of the highest volumes in Iowa. The same intersections with low severity rates have relatively low minor roadway volumes, which is consistent with expectations. Conversely, the intersections with high severity rates have relatively high minor roadway volumes, which is also consistent with expectations.

Table 1. Average daily traffic on approaches of high- and low-severity TWSC

	ADT on major roadway (expressway)	ADT on minor roadway
Low severity rate intersections	20,360	424
High severity rate intersections	11,490	2,300
Average for all intersections with at least one crash	10,840	1,362

Horizontal and vertical curves and intersection skewness (where intersecting roads meet at an angle other than 90 degrees) create situations where sight angles make it difficult to judge a gap across the median and appear to result in poorer safety performance. For example, 8 of the 10 most poorly performing intersections involve one or a combination of the following: 1) intersection skewness values of 15 degrees or more, 2) location on a horizontal curve of 3 degrees of curvature per one hundred feet or more, and/or 3) location on a vertical curve with a grade of 4% or more. One of the two remaining intersections is mildly skewed (less than 15 degrees) and is located immediately after a horizontal curve. All intersections with high severity rates are on expressways that are primary rural commuter routes, creating peaked intersection volumes, resulting in periods of congestion and delay and causing aggressive driving.

Figure 5 compares the types of crashes at the intersections with high crash severity rates and low crash severity rates. The difference in the distribution of crash type is stark. Sixty-six percent of the crashes at intersections with high severity crash rates are right-angle crashes, while only 13% are right-angle crashes at intersections with low crash severity rates. The high proportion of right-angle crashes indicates the difficulty that motorists have in judging safe gaps in traffic.

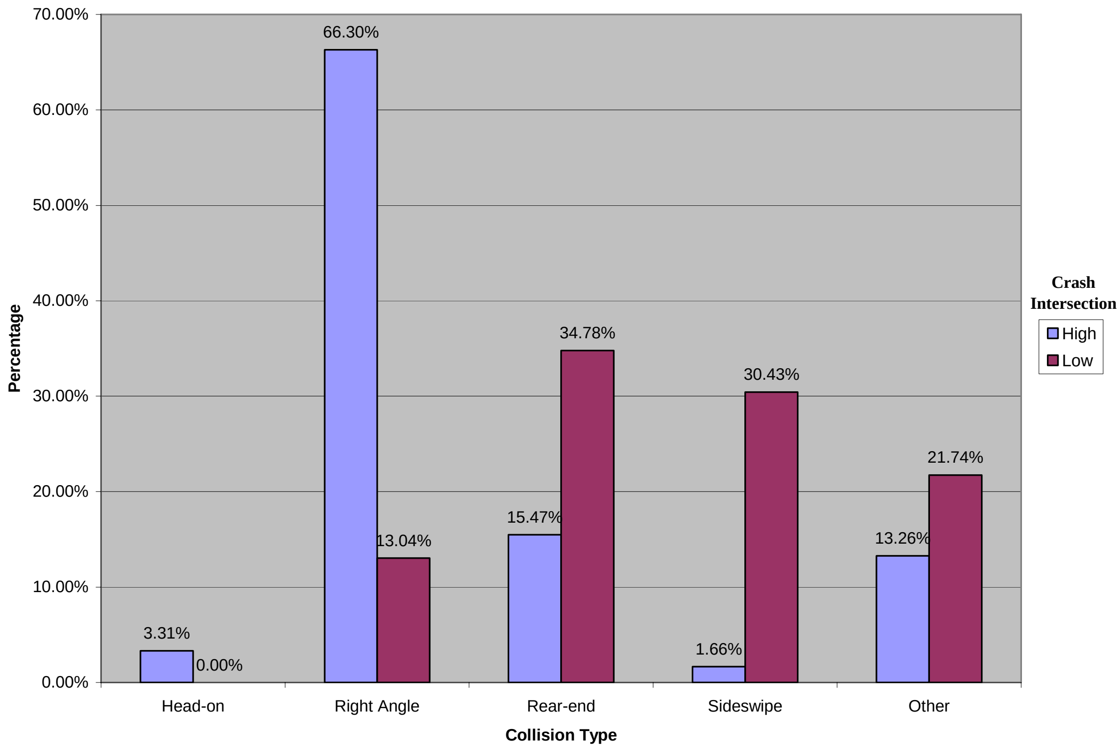

Figure 5. Crash type distributions for high and low crash severity intersections

CONCLUSION

The data analysis shows that TWSC expressway intersection crash rates, crash severity rates, and fatal crash rates increase with increasing minor roadway traffic volumes. Therefore, for intersections at which the minor roadway has high volumes (say, more than 2,000 VPD) or is expected to have high volumes, design engineers should examine the feasibility of special safety treatments instead of designing ordinary TWSC intersections.

Although future research must be done to further quantify the impacts of horizontal and vertical curves and intersection skewness, the preliminary findings presented in this paper suggest that these geometric features create problems for drivers trying to judge acceptable gaps in traffic. If curvature and skewness are related to safety performance as strongly as they appear to be, designers should work during corridor planning and preliminary engineering to avoid placing intersections where these conditions exist.

ACKNOWLEDGMENTS

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Delivering Amber Alerts to Remote Nebraska Highway Facilities Using Public Broadcasts

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ABSTRACT

Nebraska has been a leader in developing methods to deliver traveler information, evidenced by the state's growing advanced intelligent transportation system, which includes kiosks, the nation's first statewide 511 number, dynamic message signs, and a highway condition and reporting system. By the end of 2006, Nebraska will have deployed nearly 40 overhead electronic message boards across the expanse of the state's 500-mile I-80 corridor. The message boards are one important tool in Nebraska's Amber Plan.

Note: This research was still in progress at the time of publication; contact the author above for more information.

Key words: advanced intelligent transportation systems—Amber Plan—overhead message boards

Simple Design Alternatives to Improve Drainage and Reduce Erosion at Bridge Abutments

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ABSTRACT

Bridge approach settlement and the formation of the bump at the end of the bridge is a common problem that draws upon considerable resources for maintenance. Recently, a field and laboratory investigation was carried out to investigate bridge approach performance problems identified by Iowa DOT personnel. Field inspections of seventy-four existing and under construction bridges in Iowa revealed that inadequate drainage, wetting, induced soil collapse, and erosion are among the primary factors contributing to the approach slab performance problem. By characterizing backfill materials used behind bridge abutments, it was found that the specified granular backfill gradation is within the range of most erodible soils and is susceptible to increased compaction resistance due to the moisture bulking phenomenon. In addition, it was determined that a large portion of granular backfill particles are smaller than the perforation size in commonly used subdrain tile. To eliminate these problems, alternative backfill materials and drainage systems were evaluated in the laboratory using the newly developed scaled Bridge Approach Drainage Model (BADM). Using the BADM, measurements of void size, approach slab settlement, and drainage capacity were determined for thirteen different drainage/backfill designs. The results indicate that drainage performance can be greatly improved with the use of porous backfill, granulated tire chips, or geocomposite drainage systems. This paper presents a summary of the field investigation and simple design recommendations that can be implemented immediately to minimize problems associated with poor drainage, erosion, and void development at bridge approaches.

Key words: backfill—bridge abutments—bridge approach—drainage—erosion

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INTRODUCTION

Bridge approach settlement is a major problem that has gained national attention in recent years. In 1997, an NCHRP Synthesis Report *Settlement of Bridge Approaches (the bump at the end of the bridge)* (Briaud et al. 1997) showed that 25% of bridges nationwide suffered from bridge approach settlement problems with an annual maintenance cost of $100 million. According to a survey conducted by Laguros et al. (1990) of 61 different transportation agencies, bridge approach settlement is considered a “significant” problem in 70% of the agencies. Hoppe (1999) also reported that 44% of the state DOTs classify bridge approach settlement as a “major” problem. Seasonal temperature change, loss of fill material by erosion, poor construction practices, settlement of foundation soil, and high traffic levels are some of the reported major contributors to bridge approach settlement (Briaud et al. 1997).

In 2002, Iowa DOT personnel developed a research plan to investigate causes of bridge approach settlement specific to Iowa bridge designs and soil conditions. Results of this research study are reported in White et al. (2005). The research report includes (1) a detailed literature review with documentation of design, construction, and maintenance practices used by several state DOTs; (2) field inspection observations and documentation of existing bridge approach problems and construction problems; (3) characterization of the bridge approach pavement problems using elevation profiles and International Roughness Index (IRI) measurements; (4) characterization of backfill materials used behind bridge abutments with emphasis on compaction and erosion properties; and (5) analytical structural investigations on potential failure of the paving notch region and approach slab.

During field inspections, it was determined that two major causes of bridge approach settlement in Iowa are (1) poor surface and subsurface drainage and (2) erosion of the embankment and backfill materials, which form the focus of this paper. In addition to approach slab pavement problems, poor drainage and erosion lead to exposure of steel H-piles and void formation under concrete slope protection below the bridge. Poor drainage and severe soil erosion were observed at about 40% of the bridges inspected (White et al. 2005). At almost all bridges, however, some evidence of erosion was observed. These findings led to development of the Bridge Approach Drainage Model (BADM) system, which is a one-fourth scaled model of a bridge approach section that allows for the laboratory evaluation of various drainage tile, geosythetics, and backfill materials subject to controlled water infiltration conditions. Using the BADM system, measurements of void size, approach slab settlement, and drainage capacity of various design details were made and compared for optimization. The results of the BADM testing are described herein with newly proposed design details that can be implemented immediately to minimize problems associated with poor drainage, erosion, and void development at bridge approaches. Before presenting the results of the BADM tests, a brief review of backfill material specifications, compaction requirements, and drainage details documented in literature is provided. Complete details of this study can also be found at http://www.ctre.iastate.edu/research/detail.cfm?projectID=545.

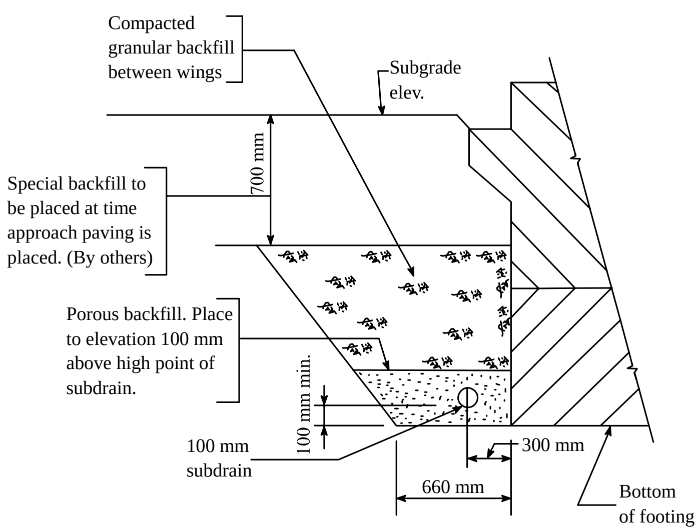
Backfill Materials

Ideal properties of backfill material include high compactability, no time dependent properties (e.g., consolidation), resistance to erosion, and elastic behavior. Backfill materials typically used behind bridge abutments are selected granular materials with some fines. FHWA (2000) recommends the use of backfill materials with less than 15% passing the No. 200 sieve. Wahls (1990) recommended the use of materials with a plasticity index (PI) less than 15%, percent of fines less than 5%, and compaction ranging from 95% of ASSHTO T-99 to 100% of ASSHTO T-180. Furthermore, Wahls (1990) reported that well-graded materials with less than 5% passing the No. 200 sieve provide for compaction with small vibratory compactors. CalTrans specifies a PI less than or equal to 15% and a relative compaction of 95% or more. Hoppe (1999) reported that 59% of the DOTs that responded to a survey limit the percent of soil passing the No. 200 sieve between 4% and 20%, with the fill placed and compacted in 150 to 200 mm lifts. However, about 50% of DOTs reportedly had difficulty obtaining the specified degree of compaction in the proximity of the bridge abutment because of compaction equipment space limitations. Iowa DOT currently specifies 20% to 100% passing the No. 8 sieve and 0% to 10% passing the No. 200 sieve for granular backfill. For compaction control, the majority of state DOTs require AASHTO T-99 as the reference laboratory compaction method (similar to Iowa DOT Laboratory Test Method 103). No moisture limits are specified for granular backfill according to Iowa DOT specifications.

Drainage Systems

Water that infiltrates down between the abutment and the bridge approach pavement or flows around the bridge can erode the backfill if not drained properly. According to Briaud et al. (1997), both surface and subsurface drainage need to be considered. An important design element is that surface runoff should be directed away from the bridge joints, slope protection, and abutment area.

A review of several drainage designs implemented by other state DOTs was carried out for comparison to practices in Iowa. The review shows that there are three main variations of subsurface drainage systems: (1) granular backfill with some porous backfill around a perforated drainage pipe, which is the current Iowa DOT design detail (see Figure 1); (2) adding a geotextile fabric around the porous fill and drain tile; and (3) using various vertical geocomposite drainage systems along the abutment face. Wrapping the porous fill with geotextiles helps reduce erosion and fines infiltration, while vertical geocomposite drains provide a pathway for water to reach the drain tile. Some states combine two or more of these details to increase the drainage efficiency.



Note: Special backfill may be substituted for granular backfill

Figure 1. Typical Iowa DOT subdrain design (Iowa DOT Bridge Standards 2005)

FIELD RECONNAISSANCE

Bridge investigations were carried out in all six Iowa districts to document various problems reported by Iowa DOT personnel at bridge approaches. In total, sixty-six existing and eight under construction bridges were investigated. Evidence of inadequate drainage and soil erosion were observed more frequently than any other bridge approach problem. Therefore, it is believed that improving drainage and reducing erosion will greatly improve bridge approach performance. Additional problems observed included poor paving notch construction, poor backfill compaction and moisture control, and differential settlement between the bridge and embankment (fill and foundation) (White et al. 2005).

Drainage System

Inadequate drainage and severe erosion problems were observed at approximately 40% of the inspected bridges. Through visual assessment, it was determined that drainage and water infiltration problems occur due to ineffective subdrains and end drains, poor granular backfill characteristics (i.e., low permeability and poor compaction), and unsealed expansion joints.

Subsurface Drainage Tiles

At all bridge sites investigated, visual observations were made as to the conditions of the subdrain tile. The inspections showed that many subdrains were completely dry, partially collapsed, or plugged with soil and debris (see Figure 2). In addition to visual observations, the subdrains of six bridge sites were inspected using a special Iowa DOT forensic camera ("snake" camera) that is inserted through the subdrain outlet using a special cable and camera head. The inspections using this procedure confirmed earlier visual inspections of collapsed and plugged subdrains.

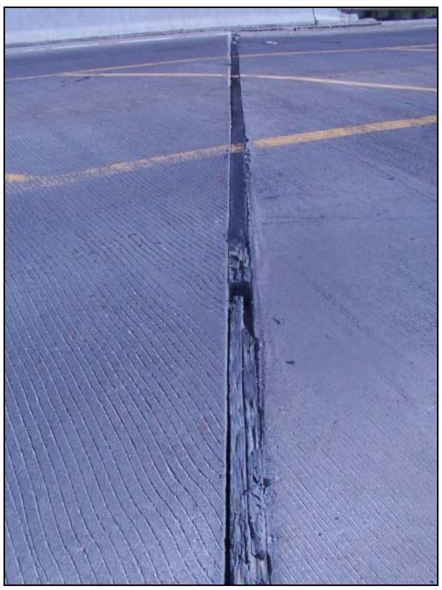
Monitoring new bridges under construction also revealed that out of eight bridges, only two bridges had the proper porous backfill material around the perforated subdrain (see Figure 1). Other bridges used granular backfill directly around the subdrain, which is believed to contribute to plugging of the drain tile. Determining the grain-size distribution for six granular samples collected at six bridge sites and comparing them to the average opening sizes of the perforated subdrains revealed that, on average, about 70% of the granular backfill gradation is finer than the average openings of the perforated subdrain (about 2 mm). On the other hand, porous backfill with no particles smaller than the No. 8 sieve has virtually no material finer than the openings of the perforated subdrain.

Figure 2. Example of a blocked subdrain outlet (Bridge No. 9266.2R218, District 5)

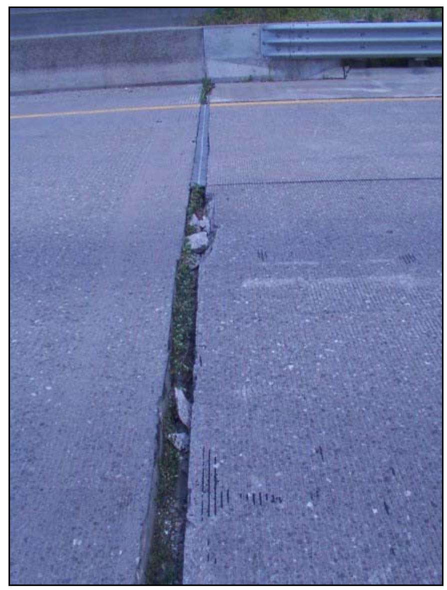
Expansion Joints and Surface Water

Unsealed expansion joints inhibit proper surface drainage by allowing water infiltration through the joint and down into the bridge approach backfill (see Figure 3). The majority of expansion joints observed were not sealed sufficiently to prevent water infiltration. Iowa DOT uses flexible foam filler, which is a common joint filler type, but recently, recycled tire chips are increasingly being used over flexible foam. Both types of joint filler however do not completely seal the expansion joint. Currently, Iowa DOT practices do not intend for the expansion joints to be sealed. This is primarily due to a lack of identifying a suitable joint sealing system capable of withstanding the harsh environmental conditions in Iowa.

In addition to unsealed expansion joints, ineffectiveness of redirecting surface runoff and infiltrated water away from the bridge can also contribute to erosion and ponding water in the pavement shoulders and erosion of the slope protection from the embankment under the bridge.



(a) Deteriorated joint filler
(Bridge No. 2034.20035, District 5)



(b) Missing joint filler
(Bridge No. 9265.7L218, District 5)

Figure 3. Typical unsealed expansion joints

Soil Erosion

Erosion of soil under the approach slab, at the shoulders, at the embankment under the bridge, and along the abutment sides are all forms of erosion observed during field inspections (see Figure 4). If not mitigated, soil erosion can lead to other problems, such as exposure of steel H-piles, undermining of abutment, faulting of pavement, or failure of the slope protection cover under the bridge.

Besides the inability to redirect surface runoff, soil erosion under approach slabs is also attributed to the use of erodible backfill materials. Briaud et al. (1997) provided a range of most erodible soils and reported that soils with silt and fine sand are more erodible than other types. The erodible range of soils was compared to the Iowa DOT gradation requirement for granular backfill and the grain-size distribution obtained for four different granular backfill samples collected at new bridge sites. It was found that the Iowa DOT gradation requirement and the gradations from the field samples have a common region with the range of most erodible soils. This is primarily attributed to the wide range allowed to pass the No. 8

sieve. Currently, Iowa DOT specifies 20% to 100% passing the No. 8 sieve. Limiting the percentage passing the No. 8 sieve to 60% would shift the grain-size distribution away from the range of most erodible soils.

(a) Bridge No. 9703.40020, District 3 **(b) Bridge No. 7871.5L/R029, District 4**

Figure 4. Erosion (a) at an embankment under the bridge and (b) under a slope protection

Wetting-induced soil collapse is another important factor that increases the likelihood of soil erosion. Upon saturation, granular soil can collapse creating a void and, thus, a water path, which facilitates additional soil erosion and progressive approach slab settlement (see Figure 5). Laboratory testing was conducted to measure the Collapse Index (CI), which is the change in the sample height upon saturation relative to its original height, for the granular and porous backfill materials used behind bridge abutments in Iowa. The highest CI measured for granular material was 6% when placed at the bulking moisture content range (3% to 7%), while porous material did not collapse at any moisture content (0% to 12%). Most of the field moisture contents measured at new bridge sites were within the bulking moisture content range making the backfill susceptible to collapse upon saturation. For a fill height of 3.0 m, a typical granular material would be expected to settle about 18 cm.

(a) Bridge No. 7779.0065, District 1 **(b) Bridge No. 9703.40020, District 3**

Figure 5. Void formation under the approach slab

BRIDGE APPROACH DRAINAGE MODEL (BADM)

Due to the high frequency of drainage problems observed in the field, a one-fourth scaled bridge approach model was constructed in the laboratory to evaluate the following:

- The current Iowa DOT drainage and backfill specifications
- The current drainage and backfill field practice
- Various backfill and drainage alternatives based on previous related research and practices of other states

The BADM system consists of an approach slab, abutment, backfill, and a drainage pipe. The model is scaled to about one-fourth of the original dimensions, except for the drainage pipe and soil which are full-scale. The model is 74 cm high, 58 cm wide and 81 cm long. Plexiglas is used to retain the backfill material inside. A perforated HDPE pipe with a 10 cm diameter similar to the subdrain used in the field is installed behind the abutment.

Water is forced to flow through a 2.5 cm expansion joint at the approach slab/abutment interface, through the drainage system, and out of the subdrain. The water is then collected in a collector and pumped back into the model via a submerged water pump (see Figure 6). To disperse the water before flowing into the expansion joint, a perforated Plexiglas tank is placed on top of the expansion joint. The inlet flow is altered until a maximum steady state flow condition is achieved. Once steady state flow is reached, the flow rate is fixed until the end of the test.

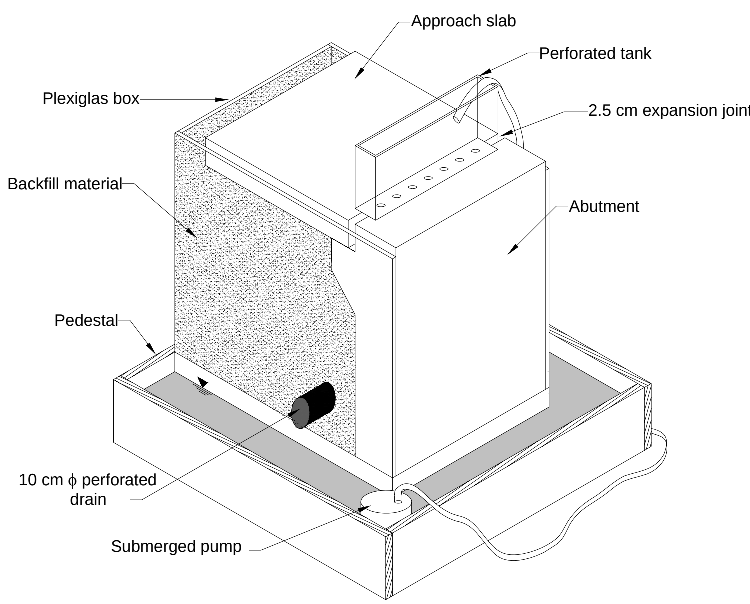


Figure 6. Schematic of Bridge Approach Drainage Model (BADM) system

To compare different drainage details, each test was allowed to run for four hours in a steady state condition. Settlement at the end of the approach slab, void development, and maximum steady state flow rate were recorded for each experiment. Settlement was calculated by measuring the difference between the approach slab elevation before and after the test. In addition, the time needed for water to flow out of the drain was noted.

Thirteen different models using granular and porous backfill materials with geocomposite drains, tire chips, and geotextile reinforcement were tested. A summary of the results is presented in Table 1. The most poorly performing model (producing minimum flow, maximum void, and maximum differential settlement) was the model simulating current practices observed in the field (No. 3 in Table 1), whereby granular backfill materials were poorly compacted at the bulking moisture content with no porous backfill around the drainage pipe.

The current Iowa DOT design also performed poorly when the granular backfill material was placed within the bulking moisture content range. A settlement of 5.1 cm, a void of 11.4 cm, and a maximum steady state flow of 32 cm^3/sec were measured (No. 1 in Table 1). The maximum steady state flow measured was considered low compared to values measured from other tests. By placing the granular backfill material at higher moisture contents (w = 12.6%) (No. 2 in Table 1), settlement and void formation were eliminated, but the maximum steady state flow remained low.

Table 1. Summary of BADM test results

Description	Settlement (cm)	Void (cm)	Maximum flow rate (cm^3/sec)	Time for water to drain (min)	Erodible Backfill (yes/no)	Collapse Susceptible Backfill (yes/no)
1. Iowa DOT design, granular backfill with average moisture content = 3.0% (bulking)	5.1	11.4	32	10	yes	yes
2. Iowa DOT design, granular backfill with average moisture content = 12.6% (non-bulking)	None	None	31	12	yes	no
3. Field practice-1, granular backfill with average moisture content = 3.0% (bulking) (without porous fill around subdrain)	5.7	10.2	33.5	10	yes	yes
4. Field practice-2, granular backfill with average moisture content = 5.5% (bulking) (with porous fill around subdrain)	5.7	5.1	67	11	yes	yes
5. Geotextile (CONTECH C-60NW non-woven fabric) around porous backfill, granular backfill with average moisture content = 4.8% (bulking)	5.1	6.4	82	10	yes	yes
6. Geotextile (CONTECH C-60NW non-woven fabric) around porous backfill and reinforcing geotextile (CONTECH C-80NW non-woven fabric) for granular backfill with average moisture content = 5.2% (bulking)	2.5	4.4	63	7	yes	yes

Table 1. (*continued*)

Description	Settlement (cm)	Void (cm)	Maximum flow rate (cm^3/sec)	Time for water to drain (min)	Erodible Backfill (yes/no)	Collapse Susceptible Backfill (yes/no)
7. Geocomposite vertical drain (Tenax Ultra-Vera™) and reinforcing geotextile (CONTECH C-80NW non-woven fabric) for granular backfill with average moisture content = 4.2% (bulking)	5.4	12.7	222	4	yes	yes
8. Geocomposite vertical drain (STRIPDRAIN 75) and reinforcing geotextile (CONTECH C-80NW non-woven fabric) for granular backfill with average moisture content = 3.7% (bulking)	6.4	3.8	383	1	yes	yes
9. Geocomposite vertical drain (STRIPDRAIN 75) and reinforcing geotextile (CONTECH C-80NW non-woven fabric) for granular backfill with average moisture content = 12.0% (non-bulking)	None	None	383	1	yes	no
10. Tire chips (18 cm thick) behind the abutment, geofoam vertical separator (2.5 cm thick) and granular backfill with average moisture content = 3.9% (bulking)	4.8	5.1	552	1	Tire chips – no Granular Backfill - yes	Tire chips – no Granular Backfill - yes
11. Tire chips (18 cm thick) behind the abutment, geofoam vertical separator (2.5 cm thick) and reinforcing geotextile (CONTECH C-80NW non-woven fabric) for granular backfill with average moisture content = 4.0% (bulking)	3.2	None	554	1	Tire chips – no Granular Backfill - yes	Tire chips – no Granular Backfill - yes
12. Tire chips (18 cm thick) behind the abutment, geofoam vertical separator (2.5 cm thick) and reinforcing geotextile (CONTECH C-80NW non-woven fabric) for granular backfill with average moisture content = 12.0% (non-bulking)	None	None	552	1	Tire chips – no Granular Backfill - yes	no
13. Porous backfill with average moisture content = 4.6% (non-bulking)	None	None	92	4	no	no

Three drainage details performed much better than the other tests. These details are (1) using a geocomposite drain with backfill reinforcement and moisture content above bulking (No. 9 in Table 1); (2) using tire chips behind the bridge abutment (No. 12 in Table 1); and (3) using porous backfill material (No. 13 in Table 1).

For the geocomposite drain with backfill reinforcement (No. 9 in Table 1), the drainage detail consisted of a vertical geocomposite drain which was 1.9 cm thick HDPE polymer (see Figure 7). The geocomposite drain was laminated with a non-woven, needle-punched geotextile. Granular backfill material was placed at a 12% moisture content and compacted every 5 cm lifts to an average relative density of about 65%. Geotextiles were used as backfill reinforcement. The first reinforcement layer was placed on top of 7.6 cm of granular backfill lift at the bottom of the model. Additional geotextile layers were placed every 13 cm. At the abutment face, the geotextiles were folded and embedded under the overlying reinforcement. The length of the embedded geotextile was approximately 13 cm. After four hours of running the test, no void or settlement developed, and the maximum steady state flow measured was 383 cm^3/sec, which is approximately 12 times higher than the value measured using the specified Iowa DOT drainage detail (Nos. 1-3 in Table 1).

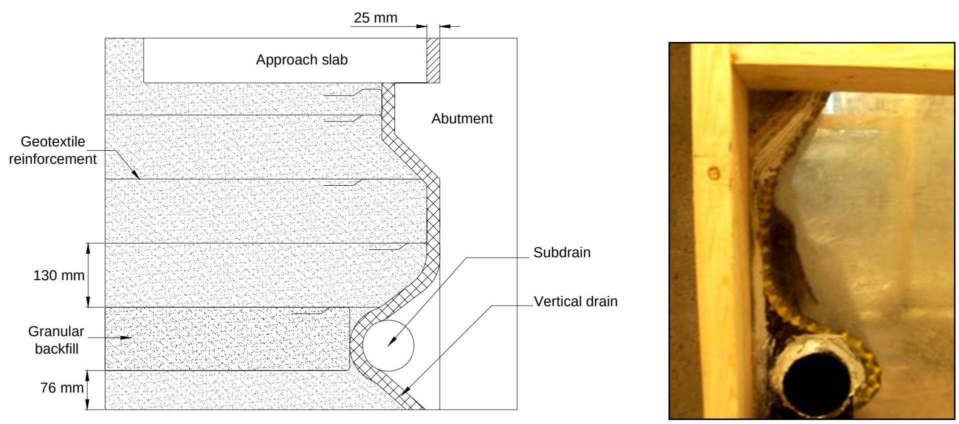


Figure 7. Using geosynthetic vertical drain at the face of the abutment with soil reinforcement

Similar to drainage detail No. 9, No. 12 (see Table 1), which uses tire chips behind the bridge abutment, showed good performance. The purpose of test No. 12 was to evaluate the use of tire chips as a drainage material, as well as its effectiveness in alleviating settlement and void development. Tire chips were placed behind the abutment without compaction over an 18 cm wide zone. For a separation barrier, a 2.5 cm thick foam board was placed between the tire chips and the granular backfill (see Figure 8). Granular backfill material was placed at a moisture content of 12%, and compacted in 5 cm lifts. An average relative density of about 70% was achieved. After four hours of running the test, the use of tire chips combined with saturated granular backfill eliminated settlement and void formation. The maximum steady state flow measured was 552 cm^3/sec, which is 43% higher than the drainage detail No. 9 and 17 times higher than drainage details Nos. 1 through 3. Although 30% of the tire chips were smaller than the drainage pipe openings, none of the tire chips were washed out. Even though this drainage detail No. 12 resulted in the highest flow capacity, this detail may require more complex construction.

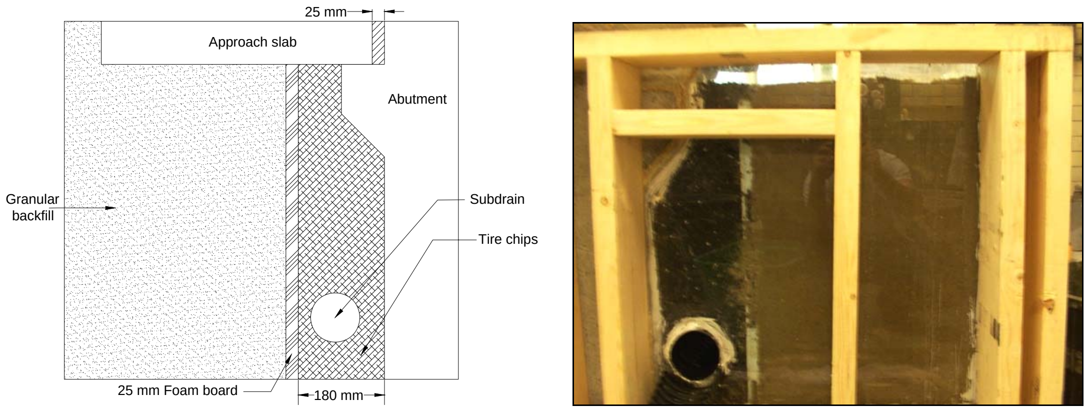


Figure 8. Using tire chips behind the bridge abutment

Another successful drainage detail was drainage model No. 13 (see Table 1), where porous backfill was used as a substitute for the granular material behind the abutment (see Figure 9). The drainage detail was evaluated based on the excellent performance of the porous material in the collapse test (no measurable collapse). The porous fill was placed at a moisture content of 4.6% and compacted every five cm to an average relative density of about 71%. The porous backfill prevented void formation and approach slab settlement. Furthermore, the maximum steady state flow measured was 92 cm^3/sec, which is approximately three times higher than the maximum steady state flow measured using the current Iowa DOT specification drainage detail (Nos. 1 through 3). However, the maximum steady state flow was lower than the measured using a geocomposite vertical drain and tire chips. Despite the relatively low flow, drainage detail No. 13 can be applied at bridge sites due to its good performance and simple construction sequence.

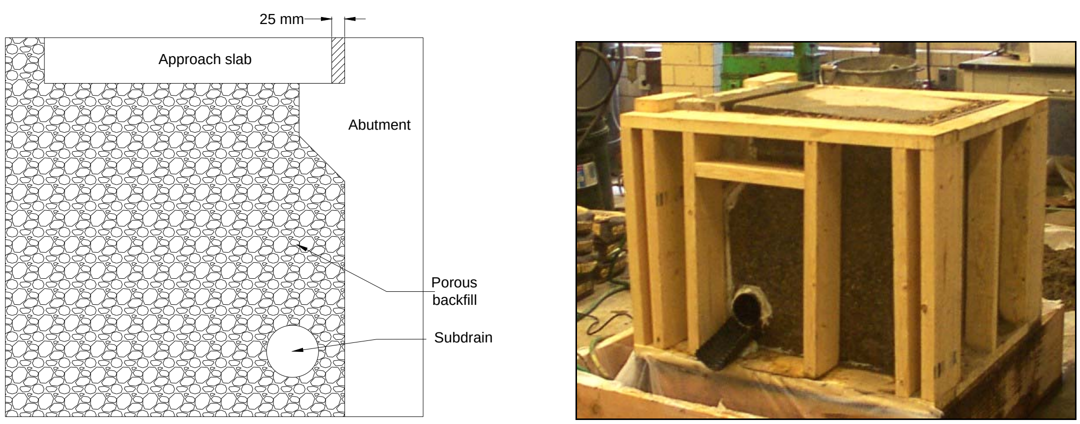


Figure 9. Using porous fill material behind the abutment

SUMMARY AND CONCLUSIONS

Bridge approach settlement and the formation of the bump is a common problem in Iowa that draws upon considerable resources for maintenance and creates a negative perception in the minds of transportation users. This study was undertaken to investigate the causes of bridge approach settlement in Iowa and develop simple concepts to improve drainage and reduce erosion.

An extensive field investigation was carried out in all six Iowa districts to document various bridge approach problems. Poor drainage and severe soil erosion were observed at approximately 40% of the inspected bridges and is primarily due to ineffective subdrain and end drain systems and water infiltration through insufficiently sealed expansion joints. Currently, Iowa DOT does not require the expansion joints to be fully sealed, which is primarily due to a lack of a suitable joint sealing system for Iowa's harsh environmental conditions.

Soil erosion under approach slabs is attributed to the use of erodible granular backfill and void formation caused by soil collapse upon saturation. Limiting the percentage passing the No. 8 sieve to 60% reduces the granular soil erodibility. Soil collapse is highest when the granular backfill is placed within the bulking moisture content range (3% to 7%), while porous backfill does not collapse at any moisture content. Void development creates a path for infiltrated water to further erode the backfill. If not remedied, erosion may lead to exposure of H-piles supporting the abutment, failure of the concrete slope cover in the embankment under the bridge, and faulting of the approach slab pavement.

To develop simple alternatives to improve drainage and alleviate erosion, a one-fourth scaled model was constructed in the laboratory. The model evaluated different backfill materials and drainage techniques based on practices of other states and concepts developed by the authors for achieving optimum performance. Settlement, void formation, and maximum steady state flow were recorded and used for evaluation of each BADM test. Of thirteen different experiments, three drainage systems performed best. These details are (1) using geocomposite drain with granular backfill reinforcement and moisture content above bulking; (2) using tire chips behind the bridge abutment; and (3) using porous backfill material. Using geocomposite drain with backfill reinforcement eliminated settlement and void formation and increased the maximum steady state flow by 12 times when compared to the value measured from the Iowa DOT drainage detail. Using tire chips behind the bridge abutment eliminated settlement and void development and resulted in the highest maximum steady state flow. However, this drainage detail may require complex construction. Using porous backfill as a substitute for granular backfill eliminated settlement and void development. Furthermore, the maximum steady state flow was three times higher than the value recorded for the Iowa DOT drainage detail. Using porous backfill can be easily and successfully applied in the field due to its erosion resistance and simple construction sequence.

ACKNOWLEDGMENTS

The Iowa Department of Transportation and the Iowa Highway Research Board sponsored this study. The authors would like to acknowledge the steering committee, Iowa DOT personnel, and contractors who helped us throughout the project and made the field investigations possible. Matt McCants with Contech Construction Products Inc. provided the geocomposite drainage materials used in the laboratory model testing.

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Implementing Emergency Alternate Routes: Wisconsin Experience

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ABSTRACT

Each year, incidents such as crashes or spilled loads cause major backups on interstates in urban and rural areas. This results in traffic randomly detouring from the freeway and choking local roads and streets. In an attempt to improve traffic conditions during these incidents, the Wisconsin DOT developed and implemented an emergency alternate route system in District 1 along Interstates 39, 90, and 94, and in District 4 along Interstate 39/US Highway 51 and State Trunk Highway 29.

The primary purpose in creating emergency alternate routes is threefold: (1) identify, in advance, alternate routes to use along each corridor; (2) establish procedures for when and how to implement alternate routes; and (3) enhance interagency communication during events.

By implementing coordinated, designated alternate routes during incidents and other major traffic delays, the time needed to clear the roadway and return it to free-flow conditions is reduced. Safety is increased as secondary crashes are minimized, agency response time is enhanced with improved communication, and incident duration is reduced.

Key words: alternate routes—incident management—operations guide

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

Each year, incidents such as crashes or spilled loads cause major backups on interstates in urban and rural areas. This results in traffic randomly detouring from the freeway and choking local roads and streets. In an attempt to improve traffic conditions during these incidents, the Wisconsin DOT (WisDOT) developed and implemented an emergency alternate route system in District 1 along Interstates 39, 90, and 94, and in District 4 along Interstate 39/US Highway 51 and State Trunk Highway 29.

The primary focus of creating emergency alternate routes is threefold:

1. Identify, in advance, alternate routes to use along each corridor
2. Establish procedures for when and how to implement alternate routes
3. Enhance interagency communication during events

By implementing coordinated, designated alternate routes during incidents and other major traffic delays, the time needed to clear the roadway and return it to free-flow conditions is reduced. Safety is increased as secondary crashes are minimized, agency response time is enhanced with improved communication, and incident duration is reduced.

METHODOLOGY

A significant aspect of this effort was collaborating with local municipalities and agencies to discuss and agree on how to divert freeway and expressway volumes onto county and local roads. The agencies included two state transportation departments (WisDOT and the Illinois DOT), 13 counties, public works departments, elected officials, emergency service responders, state patrol officers, and homeland security officials. Actively engaging the appropriate stakeholders was critical to obtaining buy-in and building positive momentum for the project.

Major elements of the planning process included the following:

- Inventorying candidate alternate routes, which included collecting data on crash history, geometric constraints, capacity, bridge height and roadway weight restrictions, local circulation, connectivity, and significant traffic generators
- Developing implementation guidelines to establish clearly the criteria that must be met to employ the alternate routes
- Creating an operations guide that graphically illustrates the alternate route for each freeway segment and lists actions that must be taken to implement the route; these actions include placing portable message signs and assigning personnel to direct traffic at key locations
- Preparing an interactive operations e-guide that enables dispatchers and users in the field to access information from a laptop or mobile data terminal
- Securing approvals from WisDOT and local agencies to formalize their cooperation and alternate route implementation procedures
- Providing guide updates and continual training for local officials and emergency responders

Preliminary Alternate Route Selection

As part of the proposal phase of this project, the team from SRF Consulting Group, Inc., identified roads that could potentially be used as alternate routes within each district. The list included a combination of state trunk highways, county trunk highways, and local streets. These roads were selected as possible alternate routes because of their general location and alignment, shown in Figures 1 and 2.

As illustrated in these figures, many of the routes are located a significant distance from the study corridors. At this point, the roadway characteristics that were needed to handle freeway-type traffic volumes had not been analyzed, and a field survey of roadway characteristics was needed to proceed.

To facilitate completing the field survey, the original list of potential alternate routes was reduced to primary alternate routes by WisDOT staff and the SRF team. Roadways were eliminated based on a variety of factors regarding their feasibility to serve as an alternate route. By using the initial criteria to eliminate several of the original routes, the study progressed more efficiently and was more cost effective.

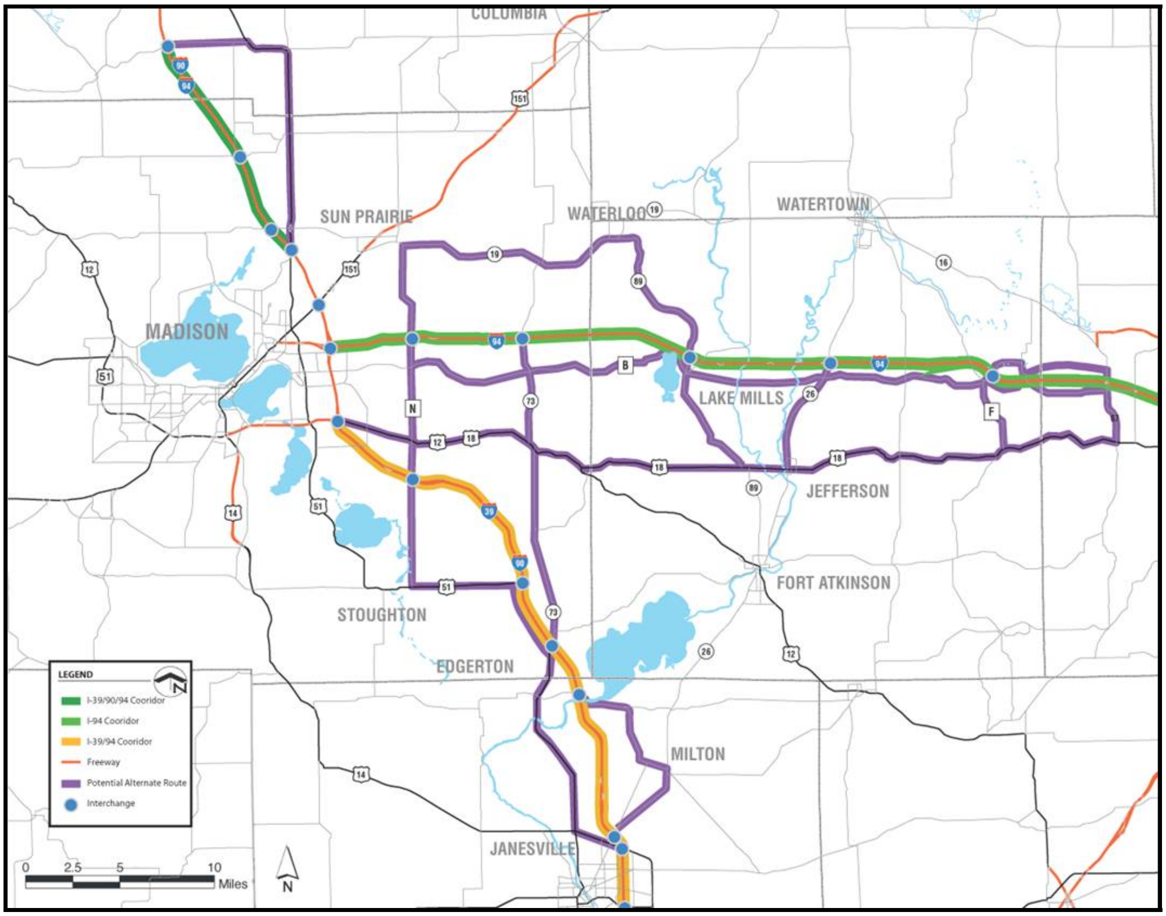

Figure 1. District 1 preliminary alternate route candidates

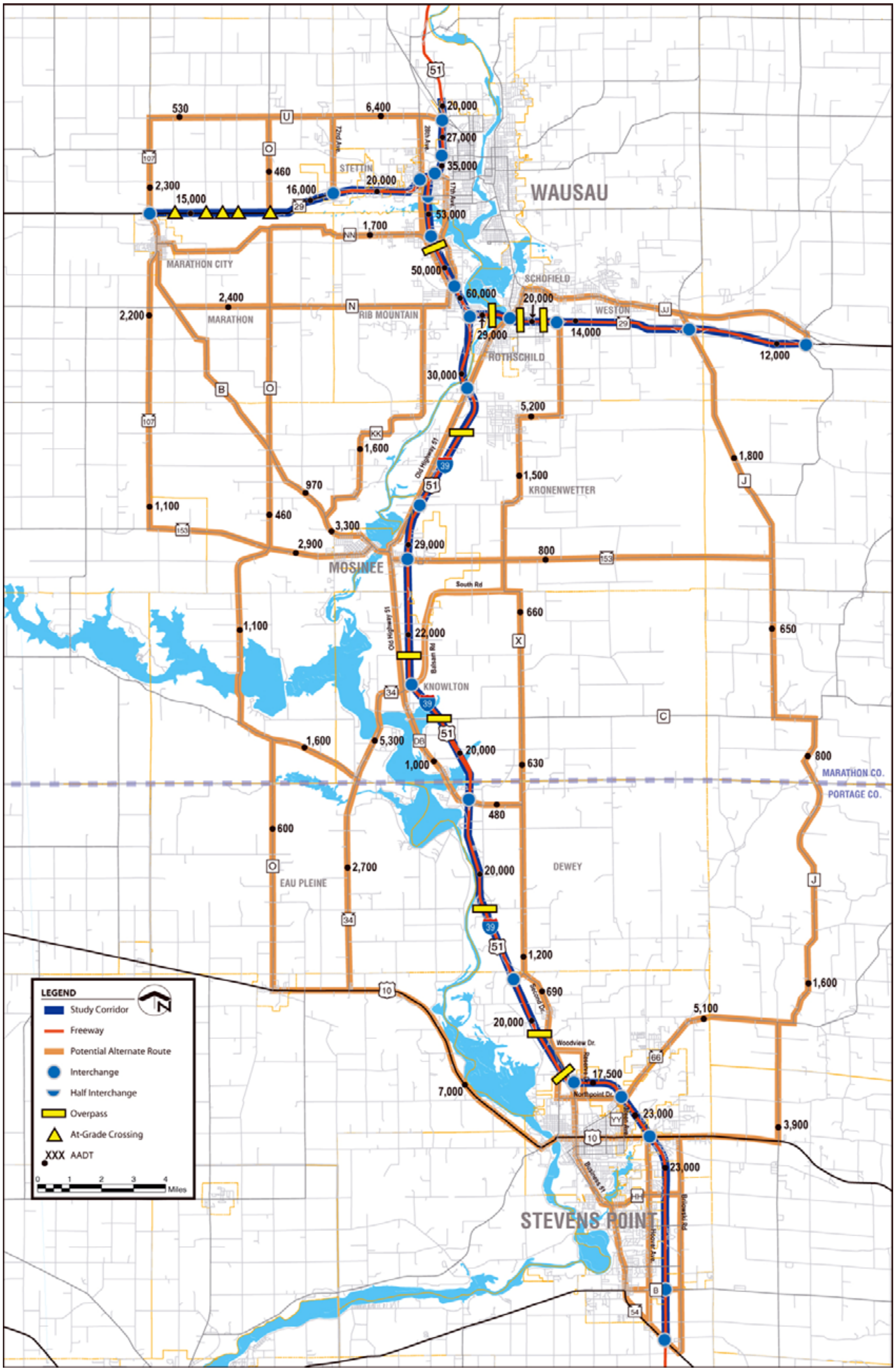

Figure 2. District 4 preliminary alternate route candidates

Data Collection Process

Roadway data was collected along the primary alternate routes. Photographs were taken of each route to provide a visual perspective of the roadway, and the following roadway characteristics were recorded:

- Location/termini
- Length
- Speed limit
- Number of lanes
- Traffic controls
- Design type: section type, lane width, shoulder width, shoulder material, etc.
- Geometrics
- Pavement condition
- Capacity constraints
- Access type: classified according to quarter-mile or half-mile spacing
- Summary comments: overall assessment of road as an alternate route

Alternate Route Selection Criteria

Next, the SRF team discussed the criteria that should be used to select the alternate routes. Compatibility with other WisDOT districts and consistency with existing statewide policies and procedures used to select alternate routes was considered essential. In addition, by establishing criteria or rules to use in selecting and eliminating routes, a technical approach and method to quantify which roads should be used as alternative incident management routes was provided.

The following criteria were proposed for selecting alternate routes:

- Use state highways, whenever possible.
- Avoid alternate routes with weight restrictions.
- Minimize the use of alternate routes that have at-grade railroad crossings, especially if a high number of trains use the railroad line.
- Avoid alternate routes with height restrictions because of underpasses or low bridge clearance.
- Use alternate routes that continue to carry traffic in the same general direction as the interstate.
- Avoid alternate routes that carry traffic in the opposite direction of intended travel for more than one mile.
- Avoid alternate routes traversing communities with multiple signals (more than four to six).
- Minimize use of alternate routes through residential areas.
- Minimize use of alternate routes that require traffic to make 90-degree turns.
- Consider alternate route options at all interchanges.
- Minimize the length of the alternate route segments.

Two additional factors were considered during this phase:

- Select roadways that WisDOT has established as existing long truck routes. However, because there are few long truck routes in Wisconsin District 4 (I-39/US 51, STH 29, US 10, STH 34, and a portion of STH 66 between Wausau and Stevens Point), this factor was not incorporated for that study area.
- Use state trunk highways. However, due to the lack of state roads that parallel either I-39/US 51 or STH 29, this factor could not be used consistently throughout both districts.

After WisDOT staff reviewed the above criteria, they were subsequently approved by the local agencies involved in the project. The final emergency alternate routes selected are shown in Figures 3, 4, and 5.

Figure 3. District 1 I-39 alternate routes

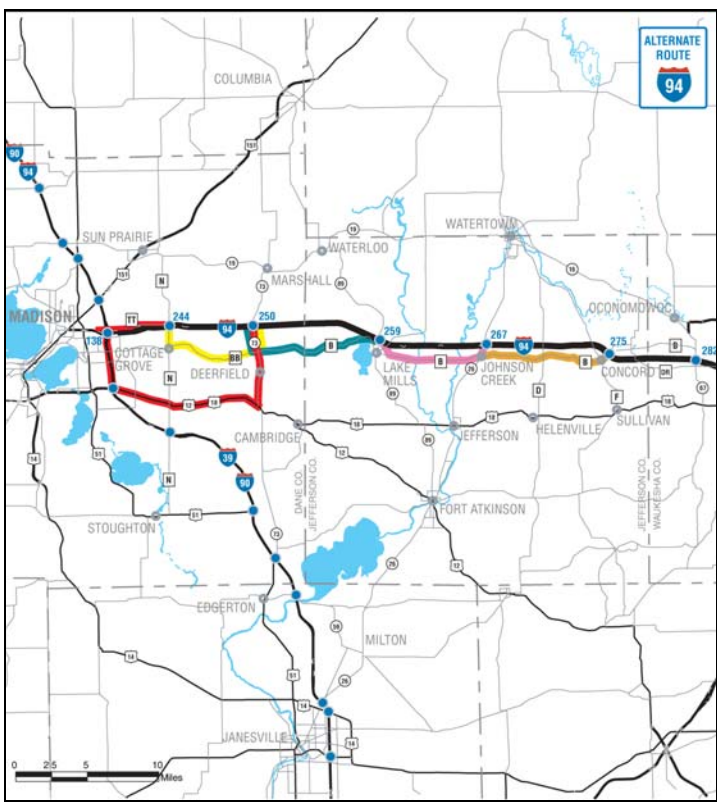

Figure 4. District 1 I-94 alternate routes

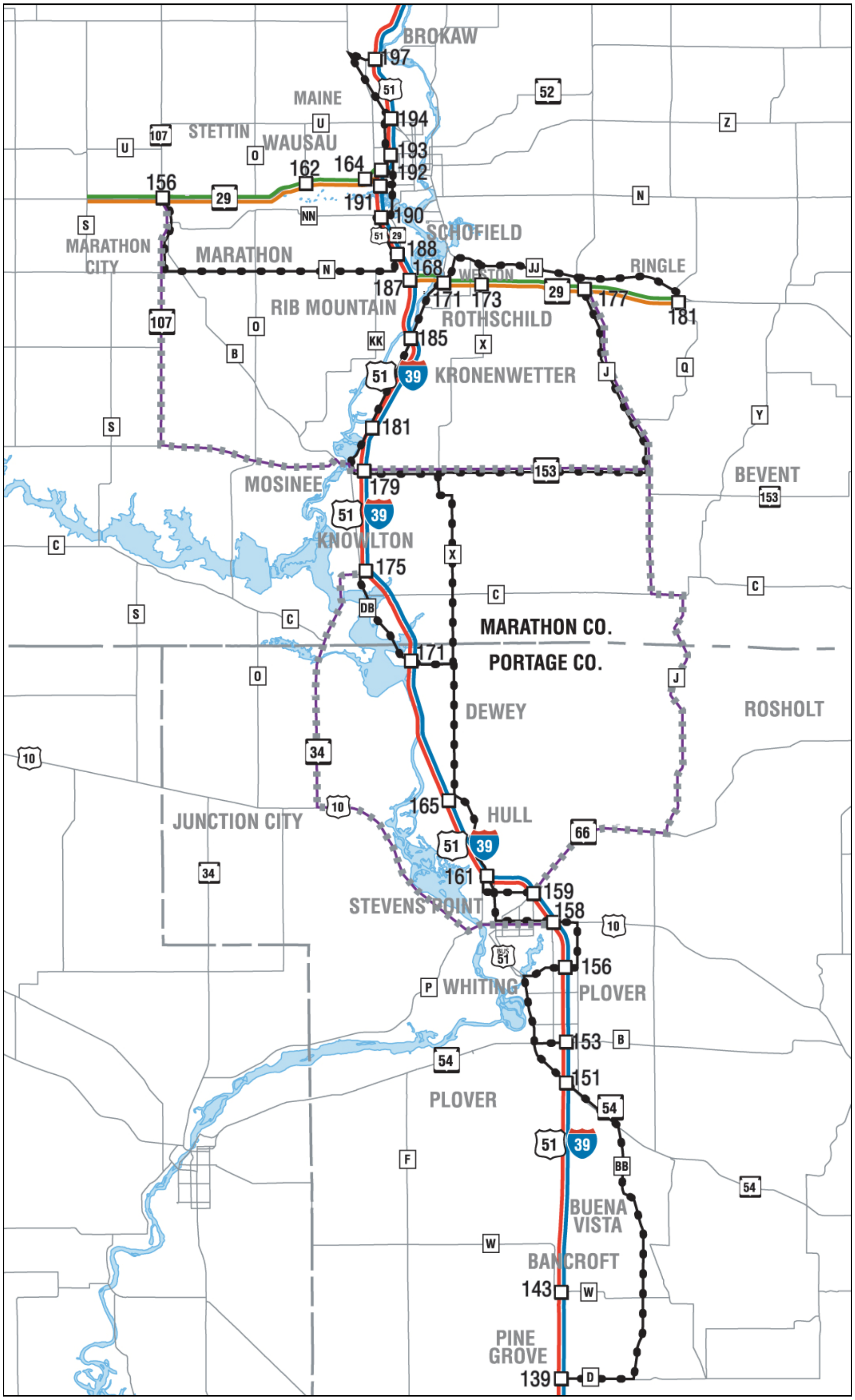

Figure 5. District 4 I-39 alternate routes

COMMUNICATION ISSUES

As a part of the effort to identify alternate routes and create appropriate procedures for their use, an inventory of the emergency response dispatch systems in the study area was also completed. A series of telephone interviews and emails were primarily used to collect data and were supplemented by discussions at a project team meeting. This information was then used to evaluate what communication systems and processes were used in the study areas.

Through this effort, it became apparent that each emergency incident has its own unique characteristics and circumstances, which influence the response procedures. Each responding agency has numerous responsibilities; however, many responsibilities are not known until additional information is gathered when first responders arrive at the scene. Securing answers to a variety of incident issues often takes time, and several immediate questions need to be answered:

- Are hazardous materials present?
- Are there any additional safety risks to the people involved in the incident?
- Are there any safety risks to the people near the incident scene or to the people traveling through the surrounding area?

First responders must answer these questions and many more when they arrive at an incident. During the study, it was determined that a comprehensive method of assessing potential problems and implementing procedures at the scene of an incident would hasten communication and improve coordination between all responding agencies.

Technology plays a large role in the ability of agencies to communicate with each other. While many agencies can communicate between the dispatch center and people at the scene, there is limited communication between the dispatching agencies. Efforts to improve this communication would result in improved service. In addition to high-technology improvements that were not cost-effective to implement, several low-technology ideas were available to enhance communication between agencies that do not use the same communication systems.

Activating, delivering, and deploying necessary equipment to the scenes of incidents can be especially problematic in rural settings, such as the I-39 corridor. Some of the issues that had to be addressed for rural settings included the distances between cities, the number of jurisdictions along the corridor, and the scattered location of and diverse ownership of various management response tools. It can also take several hours to collect and deploy the equipment used to reroute traffic.

In addition to identifying the necessary resources, it was determined that efforts should be taken to communicate details to all agencies that may be affected. Quickly identifying safety needs and potential environmental issues is critical to minimizing impacts.

To address these issues, the operations guide incorporated detailed notifications and actions for all of the involved agencies to follow during incidents.

RESEARCH RESULTS

To date, emergency alternate routes have been implemented in several locations (urban and rural) at different times of the day (peak and off-peak) for a wide range of incidents. Because each incident has unique characteristics that make it difficult to plan for, this system provides a proactive mechanism for emergency responders. Furthermore, the benefits of using alternate routes include the following:

- Reducing secondary crashes
- Keeping people and freight moving
- Improving response time
- Reducing incident duration
- Allocating staff and equipment effectively

With the implementation of the emergency alternate routes, WisDOT is making great strides in improving motorist safety and maintaining system operation and performance.

Traffic Operation and Safety Analyses of Minimum Speed Limits on Florida Rural Interstate Highways

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ABSTRACT

Traffic operation and safety characteristics were analyzed in relation to the posting of the minimum speed limit of 40 mph on Florida rural interstate freeways. The operational analysis results showed that 57% of the recorded vehicles exceeded the maximum speed limit. In addition, while only 0.14% of recorded vehicles had speeds below the 40 mph posted minimum speed limit, safety analysis results revealed that 9% of crash-involved vehicles were estimated to have occurred at speeds below 40 mph. The over-involvement of slow moving vehicles in the crash data suggests that even a small proportion of vehicles traveling under 40 mph can have negative impacts on safety. Thus, regulation of vehicle speeds at the lower end of speed distribution is important. Further, the second-order crash risk model developed to estimate the crash risk of a vehicle on the freeway as the function of the deviation from the mean traffic speed indicated that the minimum risk occurred when the pre-crash driving speed is 8 mph above the mean speed, which is equvalent to the 85th percentile speed observed in the field.

Key words: freeway operation—minimum speed—speed variation

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BACKGROUND

The relevance of the 40-mph minimum speed limit posted on the Florida interstate highway system (FIHS) is increasingly being questioned in light of the National Highway System Designation Act of 1995, which repealed the federally sanctioned maximum speed limit of 55 mph. Following this act, Florida had the maximum speed limit raised to 70 mph, the maximum speed limit allowed by Florida state statutes, leaving the 40 mph minimum speed limit unchanged. It seems logical to question the safety and operational effects of the wide existing gap (30 mph) between the two speed limits. This wide gap can affect the operation of these freeways by increasing speed differentials between vehicles, which can lead to an increase in passing maneuvers (and its attendant consequences of improper lane changing), tailgating (driving too close to the slow vehicle in front), frustrations for faster drivers, and the formation of platoons of traffic.

The posting of 40-mph minimum speed limits was motivated by the desire to make the traffic flow as uniform as possible by bringing slower drivers closer to the average speed of traffic on interstate freeways, which are designed for high-speed mobility. This paper reviews the operational and safety aspects relevant to the posting of these limits on the FIHS by analyzing individual vehicle speeds and traffic crashes that occurred on these sections. The theme throughout the analysis is the contribution of slow moving vehicles to the speed distribution and occurrence of traffic crashes.

This paper is organized as follows. The following section reviews previous studies related to the effect of speed variability and the posting of minimum speed limits. The next sections describe the research methodology used to accomplish this study, followed by a data analysis section, and finally a discussion of the research findings.

Literature Review

The posting of minimum speed limits on freeways influences drivers to operate their vehicles at speeds higher than the minimum speed values that authorities consider safe for normal operation, and thus reduce speed variability between fast and slow vehicles. Certainly, this would reduce traffic conflicts and crashes likely to be caused by interactions among the slow moving vehicles and other vehicles on the freeway.

Several studies have documented the effects of speed variability, slow driving, and the risk of crashing. The consistent findings in these studies are that the risk of crashing is smallest in the vicinity of the median speed of the traffic stream and that slow drivers are as hazardous as fast ones in the operation of the freeways. After examining 10,000 drivers on 35 sections of rural highways in 11 states, Solomon (1964) was the first to show an association between speed deviation from the mean speed and frequency of crashes. Solomon found that there was a parabolic relationship between the crash risk of a vehicle and its deviation from the mean traffic speed, where traveling slower or faster than the mean speed had higher crash risks. In this study, the minimum risk of crashing was found to occur when the difference between the vehicle speed and average traffic speed reaches 10 mph above the average speed. Solomon’s results were supported by another multi-state study done by Cirrilo (1964). This study analyzed crashes that occurred on both rural and urban sections of interstate highways in 20 states. Cirrilo found that the crash risk was highest for vehicles traveling 32 mph below the average speed and fell to a minimum for vehicles traveling 12 mph above the average speed, rising again with a further increase of speed. Harkey et al. (1990) recently reproduced these findings, where the minimum risk of crash involvement was found to be at about the 90th percentile speed (90% of the drivers are traveling below this speed), for analyzed crashes that occurred on urban roadways in North Carolina and Colorado.

A study conducted by West and Dunn (1971) found that the presence of a significant number of slow moving vehicles in the traffic stream increases speed variability and the number of crashes. The authors found that vehicles traveling at speeds of about two standard deviations below the average traffic speed were more involved in crashes than vehicles traveling at the average speed. To minimize the crash risks associated with slow driving, the authors suggested using the 15th percentile speed (15% of drivers are traveling below this speed) to set the minimum speed limits. A study by Hauer (1971) supported West and Dunn's findings after finding that the imposition of minimum speed limits on highways was twice or thrice as effective as an equivalent maximum speed limit in reducing the frequency of overtaking and thus traffic conflicts and crashes. The negative effects of slow moving vehicles in the traffic streams was also pointed out by a Transportation Research Board study, which found that the likelihood of traffic crashes on the highway increases as the speed variance increases, because the latter causes significant lane changing maneuvers, which is a potential source of conflicts on the freeways (TRB 1984). In the fatality models developed in this TRB study, speed variance was found to have a statistically significant effect on the fatality rates on interstate freeways. Other studies have supported the positive association between speed variation in the traffic stream and crashes, including Garber and Gadiraju (1998) and Garber and Ehrhart (2000).

METHODOLOGY

The posting of higher speed limits on interstate freeways and the existence of high operating speeds necessitated the evaluation of the minimum speed limit posted on these highways. The effect of the wide existing gap between maximum and minimum speed limits can be assessed by a cross-sectional study examining driver characteristics and the resulting operating speeds for similar sites with and without posted minimum speed limits. However, all interstates highways in Florida have minimum speed limits posted; therefore, creating such a study was not possible at the time of this research. Thus, this study was limited to reviewing the operational and safety characteristics of the Florida rural interstate freeways in relation to the posting of minimum speed limits and the prevailing vehicle operating speeds. Of interest in the review was the operating speeds at the lower end of the speed distribution and the speed variances resulting therefrom. The safety review was focused on analyzing crashes in terms of speed and crash type, occurrences that might be associated with the speed differences among the involved vehicles. The research efforts were also directed at conducting a regression analysis to determine the relationship between speed and crash frequency.

DATA COLLECTION

Identification of Study Sites

At the beginning of the research, the entire freeway system was reviewed by driving through it and observing geometric and traffic operating conditions. During this field review, some few samples of the vehicle speeds were taken using a radar gun. In addition, all telemetered traffic monitoring stations (TTMS) found on Florida intersate freeways were evaluated to determine the suitability of their location in relation to this research. TTMS sites collect individual vehicle records on the roadway on a 24-hour basis through the year. These sites are maintained by the Florida DOT (FDOT). Site selection criteria were then devised based on the FDOT speed zoning manual (FDOT 1997). Site selection was targeted to choosing sites that are representative samples of the FHIS and that will produce the highest free-flow speed possible, i.e., the sites devoid of curves, sustained grades, and any other geometric constraints. Seven sites where then selected, two on six-lane freeways and five on four-lane freeways.

Collection of Speed Data

After the study sites were identified, individual vehicle data were collected for 24 hours from the TTMS sites on a typical weekday under good weather and dry pavement conditions. A computer program was designed to extract volume and speed data from the files downloaded from the TTMS site. This program was capable of checking the integrity of the data by checking for any recording errors and later removing all erronous data and other anomalities. However, when the erronous data were too extensive (greater than 5%), all data were discarded and another day's data was downloaded from the site. Data from over 350,000 vehicles were verified to be good and used to analyze traffic operations in the freeway sections.

Traffic Crash Data

Four years (1998–2001) of crash data were acquired for a two-mile section in each study site. The source of this data was the safety office of the FDOT, which maintains all police-reported crashes that occurred on the state-maintained highway system. It is worth noting that police officers in Florida are required by law to report any crash that has resulted in fatality, injury, or property damage over $500. A total of 169 crashes were reported on these freeway sections during this time and were used for further analysis. It should be noted that any crash that occurred under inclement weather or jammed traffic conditions was not counted in this study.

ANALYSIS OF TRAFFIC OPERATIONS

Analysis of traffic operations was done by evaluating several statistics involving traffic volume, headways, and speeds. In the traffic volume analysis, the researchers were interested in hourly variations and the resulting level of service. Level of service is the stratification of the quality of operation of the roadway from A through F, with A representing the most favorable driving conditions and F the worst, measured at the peak-hour period of the day. In headway analysis, focus was given to vehicle platooning and vehicle speeds in these sections. Statistics useful for speed distribution analysis were the mean speeds, standard deviation of traffic speeds, and 85th and 15th percentile speeds, which traffic engineers normally consider as measures of fast and slow moving vehicles, respectively. In addition, pace speed and coefficient of variation of traffic speeds were analyzed. The coefficient of variation of speeds, which measure the relative dispersion of vehicle speeds, is calculated as the ratio of the standard deviation to the mean speed. The 10-mph pace is the 10-mph speed range with the highest number of vehicles in the speed distribution. Table 1 presents a summary of traffic operation characteristics.

Table 1. Summary of traffic operating characteristics

Statistics	Four-lane sections	Six-lane sections
Level of service	A	B
Percent traveling above 70 mph	56	57
Percent traveling below 40 mph	0.18	0.10
15th percentile speed (mph)	67	65
Average speed (mph)	73	73
85th percentile speed (mph)	80	80
Pace speed (mph)	69-79	69-79
Percent traveling in pace	65	66
Standard deviation of speed(mph)	6.2	5.4
Coefficient of variation of speed	8.5	6.8

The results presented in Table 1 shows that the level of service in these freeway sections is B or better, which indicates good operating conditions most of the time. Analysis of vehicle headways compared the mean speeds of platooned vehicles and non-platooned (or free flowing) vehicles. The results show that the differences between the speeds of platooned and non-platooned vehicles were insignificant for both four-lane and six-lane freeway sections.

The speed analysis results depicted in Table 1 show that, on average, the mean speed of all vehicles on these sections was 73 mph, which is 3 mph above the posted maximum speed limit. Specifically, the percentages of vehicles exceeding the maximum speed limit were 56% and 57% on four-lane and six-lane sections, respectively. Table 1 further shows that only 0.18% and 0.10% of the vehicles had speeds below the posted minimum speed limit (40 mph) on the four-lane and six-lane sections, respectively. The average speeds on the four-lane sections ranged from 66 mph to 74 mph, and 67 mph to 85 mph on the shoulder (outermost) and median lanes (innermost), respectively. On the six-lane sections, the average speeds of the vehicles on the shoulder, middle, and median lanes ranged from 67 mph to 70 mph, 72 mph to 75 mph, and 75 mph to 81 mph, respectively. The results further show that the 15th percentile speed by lane ranged between 61 mph to 77 mph and was averaged to be 65 mph across all sites.

Examination of the standard deviation of vehicle speeds showed low standard deviation of speeds, ranging between 4 mph and 10 mph. The averages of these standard deviations of speeds were 6.2 mph and 5.4 mph on four-lane and six-lane sections, respectively. The results further show low values for the coefficient of variation, ranging between 5% and 14%. These results suggest that the dispersion of the vehicle speeds from the mean speed is relatively small; thus, traffic flow in these sections is sufficiently uniform. Further examination of speed characteristics indicated that traffic in these highway sections flows at higher paced speeds. About two-thirds of recorded vehicles were traveling in this speed range in both the four-lane and six-lane sections.

ANALYSIS OF TRAFFIC CRASHES

Distribution of Crashes by Type

Analysis of crash severity indicated that of the 169 crashes that occurred on these sections in the four-year period, eight (4.7%) were fatal crashes, 99 (58.6%) were injury crashes, and 62 (36.7%) were property damage-only crashes. Analysis of the crashes further indicated that there were four major types of crashes that were frequently reported in the crash report forms: collision with a roadside object and run-off-road, rear-end crashes, angle/sideswipe crashes, and overturned vehicles. Stratification of crash by type indicated that about 41% of the total crashes involved a vehicle hitting a roadside object or a vehicle running off the road. The rear-end and sideswipe crashes were the second and third most occurring crashes, and accounted for 18% and 15%, respectively. All other crash types accounted for 12% of the crashes and are categorized as other.

Speed of Crash Involvement

The speed of crash involvement is the vehicle speed estimated by the investigating officer when the crash occurred, also referred to as pre-crash speed. The estimated vehicle speeds of the 244 vehicles involved in crashes were extracted from the crash report forms for the 169 crashes that were analyzed in this study. The distributions of the estimated vehicle speeds before crashes and the actual speed of the vehicles recorded from the TTMS are presented in Figure 1. Examination of the two distributions shows that the estimated pre-crash speeds are skewed to the left of the actual vehicle speeds collected at the site. Figure 1 further shows that the majority of the vehicles involved in crashes were traveling with estimated speeds between 60 and 70 mph. This range contains 66% of the vehicles involved in the crashes. However, the curve representing actual vehicle speeds shows that the majority of the vehicles, as defined by the pace speed, were traveling between 69 mph and 79 mph.

At the lower end of speed distribution in Figure 1, the results show an over-involvement of slow moving vehicles in crashes. For example, while the field data show that only 0.14% of the observed vehicles were traveling with speeds below 40 mph (the minimum speed limit) averaged across all sites, crash data analysis revealed that about 9% of vehicles involved in crashes had estimated speeds below 40 mph. Furthermore, the percent of vehicles involved in crashes with estimated speeds below 55 mph were significantly higher than the overall percentage of vehicles observed traveling below 55 mph in the field. This analysis suggests that slow driving is dangerous and poses a large risk of crashing, though the proportion of slow moving vehicles in the traffic stream on these freeway sections is low.

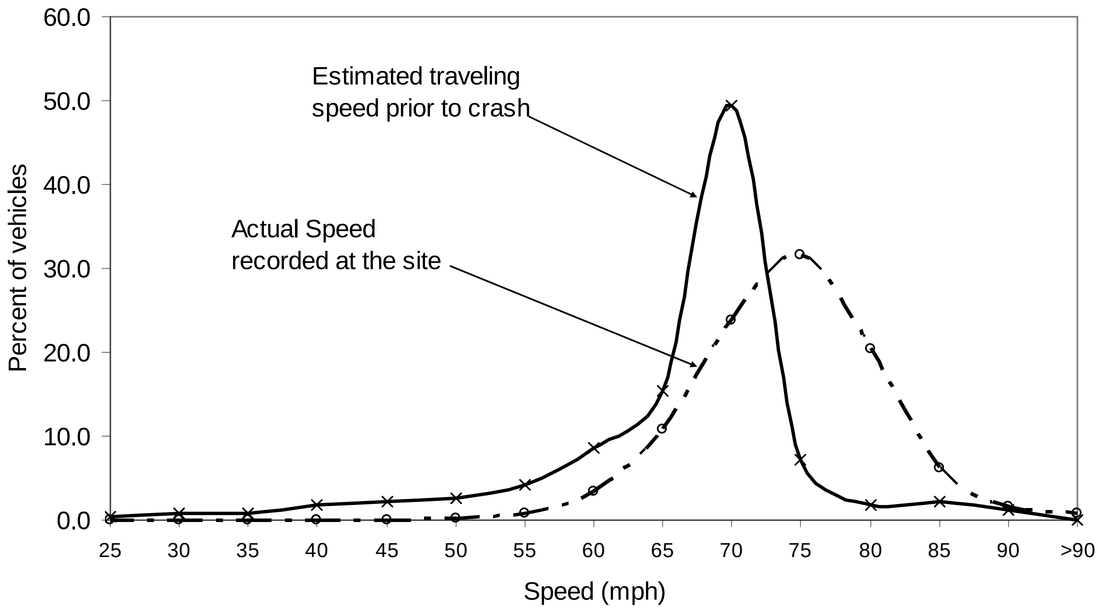

Figure 1. Distributions of estimated pre-crash speeds and actual speeds

Crash Involvement Rates

Crash involvement rates in these sections were calculated as the ratio of the number of vehicles involved in a crash within a speed range to the total number of vehicle miles of travel within that speed range. This statistic implies the effect of vehicle miles of travel (a product of volume and distance traveled) on the occurrence of crashes. Vehicle miles of travel is also known as a traffic exposure variable. The speeds of the vehicles were expressed as deviations from the average speed of, and then correlated to, the crash involvement rates. It is worth noting that the analysis of actual speed data collected across these sites revealed that the average speed of traffic on these highways was 73 mph. Crash involvement rates are presented in Table 2. Examination of the crash involvement rates and the speed deviations necessitated the use of a logarithmic transformation of the vehicle involvement rates, as shown in the third column of Table 2.

Table 2. Crash involvement rates versus deviation of speed from the mean speed

Speed deviation (mph)	Crash involvement rate (CIR) (crashes/million vehicle miles)	logCIR
-30	30000	10.31
-25	6494	8.78
-20	2885	7.97
-15	1215	7.10
-10	769	6.64
-5	497	6.21
0	914	6.82
5	134	4.90
10	123	4.81
15	413	6.02
20	580	6.36

A second-order polynomial regression model was later used to model the crash involvement rate as a function of the deviation of the vehicle speeds from the mean speed to produce a crash risk model. The dependent variable measuring the risk that a vehicle will be involved in a crash takes the following form:

$$\log \boldsymbol{CIR} = \beta_{\boldsymbol{o}} + \beta_1 \boldsymbol{SD} + \beta_2 \boldsymbol{SD}^2 + \varepsilon \qquad (1)$$

Where β_i are the regression coefficients determined by the model, *SD* is the deviation from average speed, and ε is the error term for uncorrelated variables with a mean of zero and a constant variance. The regression results are presented in Table 2. The fitted curve is also presented in Figure 2.

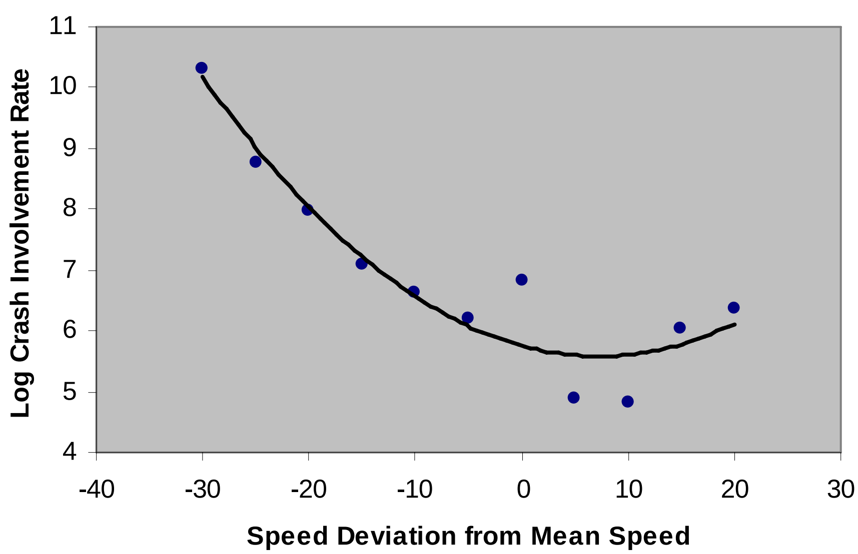

Figure 2. Crash involvement rate versus deviation from the mean speed of traffic stream

The quality of the fitted model was evaluated by its coefficient of determination (R^2) and F-statistic. From the model results presented in Table 3, an R^2 of 90% suggests that the second order polynomial model adequately fits the data and the relation has a significant curvature. In addition, the small p value (0.0001) of the overall model obtained from the model F-statistic indicates that the model is plausible and statistically significant. The partial t-tests for the individual coefficients in the model also show that all coefficients are statistically significant. Therefore, the risk of crash involvements can be estimated using the following parabolic equation:

$$\textit{Crash Involvement Rate} = \exp(0.0033SD^2 - 0.048SD + 5.754) \qquad (2)$$

After equating the first derivative of Equation 2 to zero, the minimum crash risk was found to be about 8 mph above the mean speed, or 81 mph, as the actual vehicle speeds showed an average speed of 73 mph. This finding mirrors the results that were obtained by Solomon (1964) and Cirrilo (1968), who showed that the driving speed at the minimum risk of crashing on rural interstate freeways is about 10 mph and 12 mph above the average speed, respectively. Furthermore, the data show that the risk is higher when traveling at the average speed than when traveling at 8 mph above and below the average speed.

Further analysis of the regression model developed shows that the minimum risk of crashing is equivalent to the 85th percentile speed of the vehicle speeds observed in the field. These findings mirror the recommendation made by an FHWA study (Parker 1985) that the minimum and maximum speed limits should be set at 10 mph below and above the average speed, respectively. Parker's study suggested that a speed 10 mph below the average represented the 10th percentile, while a speed 10 mph above the average represented the 90th percentile.

Table 3. Regression Results of the Polynomial Model

	Coefficient, β	**Standard error**	**t**	**p value**
SD	-0.0478	0.0131	-3.65	0.007
SD^2	0.0033	0.0008	4.33	0.003
Constant	5.7542	0.2464	23.35	0.000

F = 38.15, Prob > F = 0.0001, R^2 = 0.9051

CONCLUSIONS

This paper has presented the results of operational and safety analyses of the Florida interstate freeway system in relation to the posting of minimum speed limits. The research approach was to compare operating speed characteristics with crashes that were reported in the sampled sections. The major theme throughout the analyses was to quantify the level of importance that various speed characteristics had for safety of operations on the selected rural freeway sections.

The operational analysis results show that 57% of the recorded vehicles exceeded the maximum speed limit of 70 mph, while only 0.14% of the vehicles had speeds below the minimum speed limit of 40 mph. The 85th and 15th percentile speeds were 80 mph and 65 mph, respectively. It should be noted that these statistics are used as a guide in setting maximum and minimum speed limits, respectively, in conjunction with other safety aspects of the particular roadway. Comparison of actual vehicle speeds recorded at the site and estimated pre-crash vehicle speeds revealed overrepresentation of slow moving vehicles in the crash data (a slow vehicle was defined as one traveling below the minimum speed limit). While only 0.14% of the vehicles had field-recorded speeds below 40 mph, 9% of the crash-involved vehicles were estimated to have speeds below 40 mph. The overrepresentation of slow moving vehicles in the crashes shows that slow moving vehicle can cause problem for the safety of operation of highways, even when their proportion in the traffic stream is very small. This finding may perhaps underscore the need to raise the minimum speed limits.

The safety modeling results indicate that the presence of a significant amount of vehicles traveling slowly with respect to the driving population increases the risk that the vehicles will be involved in a crash. The relationship between crash risk and the deviation of the speed from the mean traffic speed explained by a crash risk model was found to be parabolic, with a minimum risk occuring when the vehicles were travellng at 81 mph (or 8 mph above the mean speed). The 81-mph speed was equivalent to the 85th percentile speed of vehicles observed in the field. These results replicate findings from previous research.

Should the minimum speed be raised to 65 mph, equivalent to the 15th percentile of operating speeds observed in the field? Although the results of the polynomial modeling indicate a safe driving speed, this research could not accurately answer this most fundamental question. Therefore, further research is needed to answer this question by probing the effect of the minimum speed limit sign on driver behavior, as simulation analysis would not be able to depict driver behavior appropriately on roadways with and without posted minimum speed limit signs. In the meantime, it can be presumed that higher minimum speed limits might increase incidents of driver error, particularly for vulnerable drivers, e.g., older drivers, recreational vehicle drivers, and drivers of vehicles towing trailers, who are probably comfortable with speeds below 65 mph but above 40 mph. Additional work is also needed to increase confidence in the results reported herein by attempting to determine the actual pre-crash speed, because the use of police-reported pre-crash speeds as the representative operating speed prior to crashing could skew the results.

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National Household Travel Survey Add-On Use in the Des Moines Metropolitan Area

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ABSTRACT

The Des Moines Area Metropolitan Planning Organization (MPO) serves a planning area population of 395,174 in fifteen member cities and portions of three member counties. The Des Moines Area MPO chose to purchase 1,200 National Household Travel Survey (NHTS) add-on surveys for a four-county area. Before participation in the 2001 NHTS add-on program, the last time the Des Moines area had collected extensive travel data was in the 1960s.

An essential use of the 2001 NHTS add-on data in the Des Moines metropolitan area is in the Des Moines area MPO's Travel Demand Model. Querying the 7,506 vehicle trips from the NHTS data to discern non–home-based, home-based other, and home-based work trip percentages is valuable to assigning trip types to the Travel Demand Model. Other general travel information determined from the 2001 NHTS add-on data included the length of time people travel to work, vehicle occupancies by trip type, vehicle miles driven annually, and most frequently traveled days.

The Iowa DOT and the Des Moines Area MPO joined equally to purchase the 2001 NHTS add-on data. This partnership means the ability to apply, share, and transfer the Des Moines metropolitan area NHTS add-on data and statistics to the rest of Iowa. The Des Moines Area MPO found the NHTS add-on survey to be a useful tool in calibrating and validating the Travel Demand Model and offering an accurate description of travel in the Des Moines metropolitan area.

Key words: metropolitan planning organization—national household travel survey—travel demand model

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INTRODUCTION

The Des Moines Area Metropolitan Planning Organization (MPO) serves a population of 395,174 in fifteen member cities and portions of three member counties. In 2000, the Des Moines Metropolitan Statistical Area (MSA) consisted of three central Iowa counties: Dallas, Polk, and Warren. When the opportunity arose to participate as an add-on to the 2001 National Household Travel Survey (NHTS), the Des Moines Area MPO chose to purchase 1,200 NHTS add-on surveys for a four-county area, the three counties in the Des Moines MSA and one additional county expected to join the Des Moines MSA with the release of the 2000 U.S. Census data. The expectation that the fourth county, Madison, would join the Des Moines MSA proved to be correct, verified by the release of the 2000 U.S. Census data. However, a fifth county, Guthrie, also joined the Des Moines MSA following the 2000 U.S. Census, but was not accounted for in the 2001 NHTS add-on.

Polk County, Iowa's most populous city and home of its state capitol, Des Moines, accounted for 597 of the completed NHTS add-on surveys. In Dallas County, the fastest growing county in Iowa, 262 household surveys were completed. Households in Warren County completed 241 surveys, and 131 surveys were completed in Madison County. A total of 1,231 NHTS add-on household surveys were completed for the Des Moines Area MPO from June 2001 through June 2002.

Before participation in the 2001 NHTS add-on program, the last time the Des Moines area had collected extensive travel data was in the 1960s. Much has changed in the Des Moines metropolitan area since the 1960s, and the 2001 NHTS add-on data combined with the release of the 2000 U.S. Census data enabled the Des Moines Area MPO to update the true travel picture of the region.

The Iowa DOT and the Des Moines Area MPO joined equally to purchase the 2001 NHTS add-on data. This partnership means the ability to apply, share, and transfer the Des Moines metropolitan area NHTS add-on data and statistics to the rest of the state of Iowa.

NHTS ADD-ON DATA USE

Travel Demand Modeling

An essential use of the 2001 NHTS add-on data in the Des Moines metropolitan area is the survey's input in the Des Moines Area MPO's Travel Demand Model. Querying the 7,506 vehicle trips from the NHTS data to discern non–home-based, home-based other, and home-based work trip percentages is valuable to assigning trip types to the Travel Demand Model. An expected result from the trip type queries was that non–home-based trips per household in the Des Moines metropolitan area would rise, resulting from increased trip chaining by the traveling public, from the data currently used in the Travel Demand Model. This phenomenon, however, was not found to be the case. Non–home-based trip rates did not change significantly in the 2001 NHTS add-on data over the previous Travel Demand Model inputs, indicating that the Des Moines Area MPO in its travel demand modeling efforts had been accounting for non–home-based trips and trip chaining activities adequately, thus validating the current input methodologies.

Home-based other and home-based work trip percentages, derived from the 2001 NHTS add-on data, both increased over the previous Travel Demand Model inputs. The Des Moines Area MPO staff used the 2001 NHTS add-on data to update cross-classification trip rates, using household size and the number of automobiles owned. The data showed smaller households sizes and more vehicles per household than had previously been observed. All new trip type percentages and cross-classification rates from the 2001

NHTS add-on survey data were incorporated into the calibration of the Des Moines Area MPO's Travel Demand Model for use in the creation of the *Year 2030 Long-Range Transportation Plan*.

An additional comparison useful for validating the Des Moines Area MPO's Travel Demand Model was transit usage by the traveling public. The Des Moines Area MPO does not conduct mode choice modeling. NHTS add-on data was able to assist the Des Moines Area MPO in determining whether mode choice modeling would be a valuable use of time and resources. Transit usage accounted for less than one percent of the total trips, according to the 2001 NHTS add-on data. Based on the 2001 NHTS add-on survey data, it was decided that at the present time transit usage was not a large enough percentage of total trips to warrant the creation of a mode choice model.

Travel Time Survey

The Des Moines Area MPO annually conducts a travel time survey as part of its Transportation Management Area Congestion Management System. Four to five routes exhibiting peak hour congestion at level of service D or greater are selected each year for the travel time survey. The Des Moines Area MPO staff drives each route on three consecutive days, Tuesday through Thursday, at peak travel times, morning and afternoon/evening. The days Tuesday, Wednesday, and Thursday were chosen because they have historically been the most typical travel days of the week. The travel time survey is conducted either in the spring or in the fall, when school is in session.

The purpose of the travel time survey is to determine the amount of time, an indicator of congestion, it takes to drive from point A to point B along a route. Historical data can be analyzed to determine whether construction projects completed along the route, meant to reduce congestion, reduced the travel time for commuters and others traveling the corridor.

The 2001 NHTS add-on data for the four-county central Iowa area, however, shows a different travel pattern than the historical data. The most heavily traveled three days of the week in terms of vehicle trips are Tuesday, with 17.4% of vehicle trips, Monday, with 15.8% of vehicle trips, and Friday, with 15.0% of vehicle trips. The remaining four days of the week rated as follows: Saturday, with 14.6%; Wednesday, with 13.2%; Sunday, with 12.2%; and, Thursday, with 11.8%.

A comparison of vehicle trips to person trips from the 2001 NHTS add-on data indicates that the rankings change slightly. Saturday rated highest among person trip days, with 16.1%, Tuesday was second, with 15.8%, and Sunday was third, with 15.4%. The remaining four days of the week rated as follows: Monday, with 15.3% person trips; Friday, with 14.1% person trips; Wednesday, with 12.3% person trips; and, Thursday, with 11.1 percent person trips. As shown by the 2001 NHTS add-on data, the days people most typically travel are the weekend days and Tuesdays. While the rankings by day shift from vehicle trips to person trips, Wednesday and Thursday are not among the three most heavily, or typically, traveled days of the week in either category.

Traditionally, the Des Moines Area MPO's travel time survey is conducted during peak travel times, which are the morning and evening commutes. The travel time survey is conducted from 6:30 a.m. to 8:30 a.m. and from 4:00 p.m. to 6:00 p.m., repeating routes every half hour. The 2001 NHTS add-on data, however, has shown the peak travel times differing from those used during the travel time survey.

In terms of person trips, the afternoon/evening commute is the most heavily traveled time of day. The hour between 3:00 p.m. and 4:00 p.m. is the most traveled in terms of person trips, with 9.1% traveling during this hour. Ranking second in the percentage of person trips is the hour between 4:00 p.m. and 5:00

p.m. with 9.0%, and 5:00 p.m. to 6:00 p.m. ranks third with 8.9%. Lunchtime hours, from 11:00 a.m. to 12:00 p.m. and 12:00 p.m. to 1:00 p.m., rank fourth (8.1%) and fifth (7.3%), respectively. Surprisingly, according to the 2001 NHTS add-on data, the morning commute hours of 7:00 a.m. to 8:00 a.m. and 8:00 a.m. to 9:00 a.m. rank sixth (7.3%) and eleventh (5.0%), respectively, in terms of person trips.

The spring and fall seasons are chosen to conduct the travel time survey to avoid times in the winter and summer when people may have time off for holidays and vacations and when children may have time off from school. The weather is also more temperate during the spring and fall seasons allowing the routes to be surveyed under normal driving conditions. From the 2001 NHTS add-on data, however, it is shown that several of the peak travel months are in the summer and winter. In ranking order, the most heavily traveled months, in terms of vehicle trips, are as follows:

1. August, 11.7%
2. October, 10.2%
3. December, 10.0%
4. November, 9.8%
5. January, 9.6%
6. September, 9.6%
7. July, 9.5%
8. March, 8.0%
9. April, 7.9%
10. February, 5.5%
11. June, 4.1%
12. May, 4.1%

Likewise, for person trips, several of the peak travel months occur in the summer and winter seasons. In ranking order, the most heavily traveled months, in terms of person trips, are as follows:

1. August, 12.3%
2. December, 10.9%
3. October, 10.5%
4. September, 10.2%
5. July, 9.9%
6. January, 9.3%
7. November, 9.3%
8. March, 7.4%
9. April, 7.3%
10. February, 5.3%
11. June, 3.8%
12. May, 3.7%

Based on this 2001 NHTS add-on information, with typical travel days shifted from Wednesday and Thursday to other days of the week, peak travel times not centered as much around morning work commute times, and heavy travel months not necessarily occurring in the spring and fall seasons, the Des Moines Area MPO will re-evaluate the use of and timing of the travel time survey in the future.

General Travel Information

Household, Person, and Vehicle Data

In the four-county survey area, 1,231 households were surveyed for the 2001 NHTS add-on effort, equating to 184,740 households after weighting. Likewise, 2,986 persons were surveyed, for a weighted total of 470,041 persons residing in the four counties. The 2001 NHTS add-on data reported 2.5 persons per household. Of those 2.5 persons per household, 1.9 are drivers aged 15 years or older. From the total population, 93.9% of persons aged 15 years and older are reported as drivers.

Each household averages 2.1 vehicles, meaning there are more vehicles than drivers in the Des Moines metropolitan area. This finding corresponds with the national 2001 NHTS, which reported 1.8 drivers per household and 1.9 vehicles per household (BTS 2003).

Affluence and household size affects the number of vehicles per household. For example, the average number of vehicles owned by a household of two is 2.2, while the average number of vehicles owned by a household of six is 3.3. Table 1 shows the four-county average number of vehicles by household size.

Table 1. Average number of vehicles by household size

Household size	Average number of vehicles
1	1.3
2	2.2
3	2.7
4	2.5
5	2.5
6	3.3
7+	2.7

Household income also is a determining factor for the number of vehicles the household can access. For example, households earning $20,001 to $25,000 average 1.5 vehicles, households earning $60,001 to $65,000 average 2.4 vehicles, and households earning more than $100,000 annually average 2.6 vehicles. Table 2 shows the average number of vehicles by household income level. The average number of vehicles per household peaked at the $65,001 to $70,000 income level, with 2.7 vehicles.

Household income affects the age of the vehicles owned as well. Table 3 shows the average model year of vehicles by household income level. Not surprisingly, as household income levels rose, so did the average model year of the vehicles used by the household. What was slightly surprising is that the average model years were not showing years later than 1995. The data for this survey effort was collected from June 2001 through June 2002, meaning the average vehicle was approximately six years old at a minimum. This data finding is consistent with the national 2001 NHTS, which revealed that vehicles in households with an income of $100,000 or more averaged a model year of 1996, and households with an income of less than $25,000 had an average model year of 1991 (BTS 2003).

Table 2. Average number of vehicles by household income level

Household income	Average number of vehicles
Less than $5,000	1.8
$5,001 to $10,000	1.1
$10,001 to $15,000	1.4
$15,001 to $20,000	1.5
$20,001 to $25,000	1.5
$25,001 to $30,000	1.8
$30,001 to $35,000	1.8
$35,001 to $40,000	2.1
$40,001 to $45,000	2.1
$45,001 to $50,000	2.1
$50,001 to $55,000	2.2
$55,001 to $60,000	2.4
$60,001 to $65,000	2.4
$65,001 to $70,000	2.7
$70,001 to $75,000	2.6
$75,001 to $80,000	2.4
$80,001 to $100,000	2.6
More than $100,000	2.6

Table 3. Average vehicle model year by household income level

Household income	Average model year of vehicles
Less Than $5,000	1992
$5,001 to $10,000	1989
$10,001 to $15,000	1992
$15,001 to $20,000	1989
$20,001 to $25,000	1990
$25,001 to $30,000	1992
$30,001 to $35,000	1992
$35,001 to $40,000	1992
$40,001 to $45,000	1992
$45,001 to $50,000	1992
$50,001 to $55,000	1993
$55,001 to $60,000	1993
$60,001 to $65,000	1993
$65,001 to $70,000	1993
$70,001 to $75,000	1993
$75,001 to $80,000	1993
$80,001 to $100,000	1994
More Than $100,000	1995

The number of person trips per day averaged 3.9. Females made an average 4.2 trips per day and males averaged 3.8 trips per day. Age, rather than gender, had a greater effect on number of daily trips. Persons aged 45 to 54 averaged 4.9 trips per day, making the greatest number of trips per day of any age cohort, while those aged 65 and older averaged only 3.7 trips per day, making the least number of trips per day of any age cohort. Table 4 shows the average number of daily trips per person by age cohort. Those age groups averaging more than four trips per day would likely have work, social activities, and children's activities, increasing the number of daily trips over the other age cohorts.

Table 4. Average number of daily trips per person by age cohort

Age cohort	Average number of daily trips per person
15 to 24	3.8
25 to 34	4.1
35 to 44	4.5
45 to 54	4.9
55 to 64	3.9
65 and older	3.7

Average Trip Lengths

Other general travel information determined from the 2001 NHTS add-on data included the mileage people travel to work or social/recreational activities, which is longer than the mileage people travel to run errands or obtain services. This would indicate that people are willing to travel to reach their workplace or a social activity, but would rather do business closer to home or work for shopping and services. The average work trip length in the Des Moines metropolitan area is 10.7 miles, and the average social/recreational trip length is 13.2 miles. Shopping/errand trips are average 5.6 miles and medical/dental services trips are approximately 7.3 miles in length. By comparison, the average length of all trips is 8.3 miles. Clustering service-type businesses around large corporate locations and housing developments could prove appealing to persons when selecting a home or workplace. Housing and workplace proximity are not necessarily deciding factors, however, when choosing those respective locations. Further research could be done to determine whether mileage between work and home is as important as other factors when choosing a residence and whether this happens by choice or by chance.

Vehicle Occupancies

Vehicle occupancies are higher for social/recreational activities (2.3 persons) and errand trips (1.8 persons) than for work trips (1.4 persons). This phenomenon is not surprising. Families or friends tend to go to social/recreational activities or run errands together, while people are more likely to drive alone to work.

Annual Vehicle Mileage

Of those vehicle owners knowledgeable of miles vehicles are driven annually, the majority of vehicles (61.4%) were driven fewer than 10,000 miles annually. More specifically, vehicles driven 5,000 miles or less totaled 35.5% and vehicles driven 5,001 through 10,000 miles totaled 25.8%. Other mileage percentages of note include: 10,001 to 15,000 annual miles driven, 22.2%; 15,001 to 20,000 annual miles driven, 8.3%; and, more than 20,000 annual miles driven, 8.1%. According to the U.S. Environmental Protection Agency, the national average annual mileage is 12,500 miles for a passenger car and 14,000 miles for a light truck (EPA 2000). Therefore, with a majority of vehicles driven under 10,000 miles annually, the Des Moines metropolitan area is below the national average. Accounting for this fact may be the age of the population in the Des Moines metropolitan area and/or the compactness of the region.

As explained earlier, average work trips and shopping/errand trips average only 10.7 miles and 5.6 miles, respectively. The services and employment opportunities the general population requires are all available in a compact area, with the Des Moines Area MPO planning area measuring only 499.7 square miles. Thus, there is not a need by the majority of the vehicles to be driven greater than 10,000 miles annually.

The state of Iowa ranks fourth in terms of the percent of total population aged 65 and over (Himes 2003). According to the 2001 NHTS add-on data, the average age in the four-county central Iowa survey area was 34.4 years. These older populations tend to drive less than younger populations. For example, the percentage of travel day vehicle miles traveled by the 21–25 age cohort was 3.6%, while the 71–75 age cohort traveled 2.0% of the travel day vehicle miles.

Vehicle Type

The majority (56.2%) of household vehicles owned in the four-county survey area were automobiles/cars/station wagons, with pickup trucks a distant second place (16.0%). Combined, sport utility vehicles and vans were 23.9% of the total household vehicles owned.

Work Trips

Work trips made by public transit/commuter/school buses amounted to 1.1% of total work trips, while work trips made by car/sport utility vehicle/van/pickup truck amounted to 93.2% of total work trips. By comparison, walking accounted for 4.2% of work trips in the four-county area.

CONCLUSION

All of the statistics detailed in the 2001 NHTS add-on survey data add up to create a profile for the Des Moines metropolitan area’s travel pattern. Obtaining the 1,200 surveys through the 2001 NHTS add-on program painted a picture of travel on the transportation system, which caused the Des Moines Area MPO to be more aware of the travel patterns existing in the planning area.

The Des Moines Area MPO found the NHTS add-on survey to be a useful data set in calibrating and validating the Travel Demand Model and offering an accurate description of travel in the Des Moines metropolitan area. The data determined by the 2001 NHTS add-on was vital in updating the Des Moines Area MPO’s Travel Demand Model inputs. Trip type rates and household sizes and number of vehicles owned all were able to be discerned from the 2001 NHTS add-on data, allowing the travel demand modeling process to reflect a more accurate picture of travel in the Des Moines metropolitan area.

Using the outputs from the Travel Demand Model and the peak travel time data from the 2001 NHTS add-on will enable the Des Moines Area MPO staff to re-evaluate the use of and timing of the travel time survey. Coupling those two data sources demonstrates where and when the congestion is occurring in the Des Moines metropolitan area, and new routes and peak-hour travel times may be determined from this data.

To begin observing travel habits in a more trendline view and realizing the value of this information, the Des Moines Area MPO hopes to participate in the NHTS add-on program to be conducted around the year 2010.

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Performance of SPS-1 Project in Kansas

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ABSTRACT

The Long-Term Pavement Performance (LTPP) SPS-1 experiment entitled "Strategic Study of Structural Factors for Flexible Pavements" was developed to determine and evaluate the factors affecting the performance of flexible pavements. The experimental design was aimed at determining the effects of following specific pavement design features: (1) base type, (2) base thickness, (3) pavement thickness, and (4) in-pavement drainage systems. The SPS-1 experimental project in Kansas, constructed in 1993, was located in the eastbound driving lane of US-54, a major rural arterial. The experiment consisted of 12 standard SPS-1 sections, 1 Kansas DOT (KDOT) control section, and 5 supplemental sections. The service life of these sections was estimated by KDOT to be from 2 to 120 years. However, because of premature distresses, some sections were rehabilitated in 1996 and 1997. This paper discusses causes of premature distresses on some sections and the performance of other sections.

Key words: flexible pavements—SPS-1—rutting

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INTRODUCTION

The Long-Term Pavement Performance (LTPP) program within the Strategic Highway Research Program (SHRP) was aimed at determining the effects of various design features and environmental and material performance by recording performance and construction data for a large number of in-service and new pavement sections. The LTPP program was divided into two subprograms, General Pavement Studies (GPS), representing pavements that used material and structural design practices in the United States and Canada from the 1960s to the 1980s; and Specific Pavement Studies (SPS), representing performance and specific structural factors of interest in the 1990s.

The SPS-1 project, “Strategic Study of Structural Factors for Flexible Pavements,” was designed to determine and evaluate factors affecting the performance of flexible pavements (FHWA 1993). The experiment examines the effects of climatic region, subgrade soil (fine or coarse), and traffic rate (as covariate) on pavement sections incorporating different levels of structural factors. These factors include drainage (presence or lack), asphalt concrete (AC) surface thickness (4 in. and 7 in.); base type (dense-graded untreated aggregate and/or dense-graded asphalt-treated), and base thickness (8 in. and 12 in. for undrained sections; and 8 in., 12 in., and 16 in. for drained sections). The design stipulated a traffic loading level in the study lane in excess of 100,000 equivalent single axle loads (ESALs) per year (FHWA 1993). This paper examines the performance of the SPS-1 project in Kansas, which incorporated some of the above factors for a particular soil type and traffic.

THE SPS-1 PROJECT IN KANSAS

Project Layout and Description of Test Sections

The SPS-1 project in Kansas, constructed in 1993, is located in the eastbound driving lane of US-54 in Kiowa County. The experiment consisted of 12 SPS-1 standard sections, 1 KDOT control section, and 5 supplemental sections, as shown in Figure 1. The project involved construction of a new two-way asphalt-surfaced roadway, offset from the original alignment by approximately 50 feet. The sections were laid out on the base type and surface layer thickness criteria (Johnson 1994). Three KDOT supplemental sections were constructed first, followed by the KDOT control section. Six sections with dense-graded aggregate base (DGAB) were built next, followed by six sections with asphalt-treated base (ATB). Two KDOT supplemental sections with asphalt surfacing over dense-graded aggregate base were placed last. The permeable asphalt-treated base (PATB), designed to provide in-pavement drainage, was placed on six sections (SHRP #4 to SHRP #9). These sections also had a drainage blanket and longitudinal drains. Longitudinal drain was also provided for KDOT Sections 1 and 2. The KDOT control section did not have any base and was built directly over fly ash-treated subgrade.

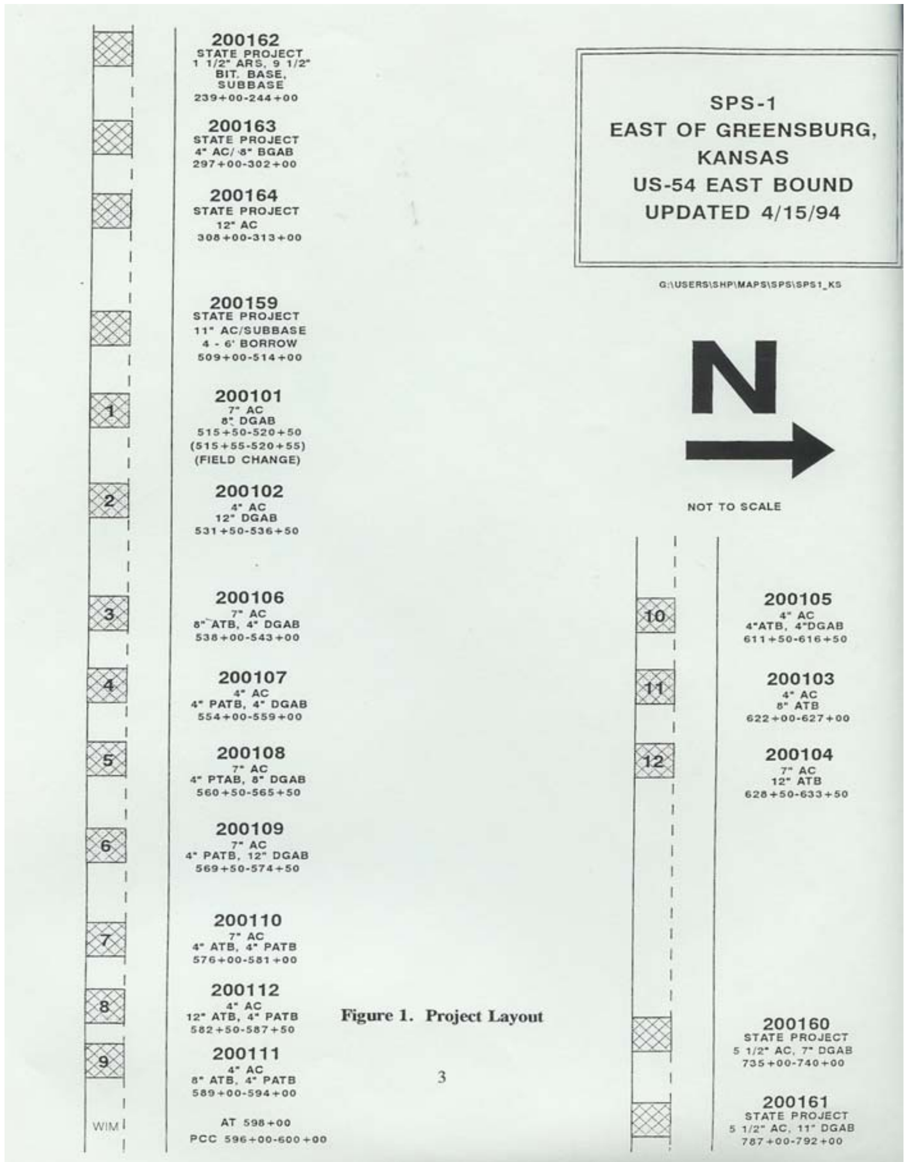


Figure 1. Kansas SPS-1 project layout

Table 1 summarizes the design features and section attributes of the SPS-1 project in Kansas. As mentioned earlier, the sections were built with varying base types (DGAB, ATB, or PATB) and with or without longitudinal drains. The thicknesses of the asphalt mixture surface and base courses were also varied. The sections had dense-graded asphalt mixtures designed by the Marshall method on most sections. Three sections (KDOT #3, #5, and #6) had asphalt-rubber mixtures in the asphalt base and/or surface courses. The total asphalt layer thickness varied from 4 in. to 11 in. for the KDOT sections and 4 in. to 7 in. for the SHRP test sections.

Traffic

The annual average daily traffic (AADT) of US-54 was 5,000 with 28% trucks in 1993. The AADT decreased to 4,648 with 22% trucks in 1996, then increased to 5,006 with 30% trucks in 2000. The calculated ESALs decreased from 230,000 in 1993 to 73,000 in 1996, then increased to 351,000 in 2000. The applied cumulative ESALs up to 1996 and 2000 were 670,000 and 1,937,000, respectively.

Table 1. Section attributes of the SPS-1 project in Kansas

Section no.	Layer thickness (in.)	Drainage type	Design life (years)	Construction year	Rehabilitation year
KDOT #3	1.5"ARS/9.9"BM-2C/6"FASB	No sub.	10	1993	1996
KDOT #5	1.5"ARS/2.5"ARB/8"DGAB/ 6"FASB	No sub.	13	1993	1996
KDOT #6	1.5"ARS/2.5"ARB/8"BM-2C/ 6"FASB	No sub.	20	1993	1996
SPS-1 CS	2"BM-1B/8.8"BM-2C/6"FASB	No sub.	10	1993	1997
SHRP #1	7.6"BM-1B/8.5"DGAB/6"FASB	No sub.	13	1993	1996
SHRP #2	4"BM-1B/12.3"DGAB/6"FASB	No sub.	4.5	1993	1996
SHRP #3	7.3"BM-1B/7.3"ATB/ 4"DGAB/6"FASB	No sub.	87	1993	
SHRP #4	4.1"BM-1B/4.1"PATB/ 3.7"DGAB/6"FASB	B	6.5	1993	1996
SHRP #5	7.6"BM-1B/3.6"PATB/ 7.9"DGAB/6"FASB	B	20	1993	
SHRP #6	7"BM-1B/3.6"PATB/ 11.9"DGAB/ /6"FASB	B	120	1993	
SHRP #7	7"BM-1B/3.8"ATB/ 3.9"PATB/6"FASB	B	36	1993	
SHRP #8	5"BM-1B/12"ATB/ 3.6”PATB /6"FASB	B	95	1993	
SHRP #9	4"BM-1B/8.5"ATB/ 3.6”PATB /6"FASB	B	39	1993	
SHRP #10	3.9"BM-1B/3.8"ATB/ 4.1”DGAB/6"FASB	No sub.	2	1993	
SHRP #11	3.6"BM-1B/7.7"ATB/6"FASB	No sub.	28	1993	
SHRP #12	6.8"BM-1B/12.1"ATB/6"FASB	No sub.	47	1993	
KDOT #1	1.5"BM-1B/4"BM-2C/ 7"DGAB/GG/6"FASB	L	10	1993	1997
KDOT #2	1.5"BM-1B/4"BM-2C/ 11"DGAB/6"FASB	L	10	1993	1997

Table 1 Key

Drainage type No sub.: no sub-drainage layer
B: drainage blanket with long drains
L: longitudinal drains

Layer type

ARS: asphalt rubber surface	DGAB: dense graded aggregate base
BM-1B: dense graded surface mix	PATB: permeable asphalt treated base
BM-2C: dense graded base mix	GG: geogrid, engineering fabric
ARB: asphalt rubber base	FASB: fly ash-stabilized base
ATB: asphalt treated Base	SPS-1 CS: KDOT control section

Construction Overview

Project construction started in the fall of 1992 with the required removals and utility relocations. The preparation of base and subbase courses was scheduled to begin in May 1993. However, the project was delayed due to weather conditions (rain) (Johnson 1994).

Preparation and Compaction of Subgrade

Fly ash was introduced into the subgrade, which was sandy silt, to dry out the soil. Class C fly ash was mixed into the upper six inches of the soil at a target application rate of 8% by weight. Compaction was achieved through moisture-density relationships. This provided uniform material with uniform thickness along the test section and ensured that the compaction specifications were met for forming a stable working platform.

Base Layer

As mentioned earlier, two base types were used for the SPS-1 project: undrained (unstabilized and stabilized) and drained (stabilized). The undrained bases were relatively impermeable layers consisting of DGAB and/or ATB. Segregation or degradation of materials during placement of DGAB was controlled and the quality of in-place density was monitored. Low-viscosity asphalt cement was used to prime the surface of the DGAB in the test sections where a permeable ATB layer was placed over DGAB. The ATB layer was constructed to control elevations and minimize surface irregularities. Low-viscosity asphalt cement was also used to tack the ATB layer before placing the surface course.

The drainable base layers incorporated a permeable layer (PATB) with edge drains to permit water to drain out of the pavement structure. The PATB mixture, with relatively open gradation, was central plant-mixed, hot-laid, and contained 2% to 2.5% asphalt with the same grade as that used in a hot-mix asphalt (HMA) surface course. Edge drains were installed in the shoulders of the pavement sections with PATB. Both inside and outside edge drains were constructed for crowned pavements, while only one edge drain was required for the cross-sloped pavements. The PATB was used as a backfill in the edge drain trench. Transverse collector sub-drains were located in the transition zones between the drained and undrained sections whenever there was a longitudinal slope.

Hot-Mix Asphalt Concrete

The asphalt concrete surface mixtures were designed to meet the specifications provided by the Marshall mix design method. Most of the SHRP and KDOT sections had a KDOT BM-1B mixture (1/2-in. nominal maximum aggregate size [NMAS]) as the surface course and a KDOT BM-2C (3/4-in. NMAS) as the base course. Table 2 shows the details of these mixtures and Figure 2 shows the gradations.

The BM-2C mixture was designed with 75 blows of a Marshall hammer. The design asphalt content was 4.25% (AC-10). The mixture had 15% CS-1A (1 1/2-in. NMAS), 21% CS-1D (3/4-in. NMAS), 39% limestone screening (CS-2C, 1/4-in. NMAS) and 25% sand and sand gravel. The specifications required 50% of the coarse aggregates be crushed. The design gradation provided 75% crushed materials. As shown in Figure 2, the mixture has a dense gradation.

The BM-1B mixture was also designed by the Marshall method, but with 50 blows. This mixture had 4.75% (AC-10) design asphalt content. The blend consisted of 35% CS-1D (3/4-in. NMAS), 16% CS-1K (3/8-in. NMAS), 24% limestone screening (CS-2C, 1/4-in. NMAS) and 25% sand and sand gravel. The specifications required 75% of the coarse aggregates be crushed. The design gradation provided 75% crushed materials. As shown in Figure 2, this mixture is also dense-graded.

The gradations for the asphalt rubber base (ARB) and asphalt rubber surface (ARS), also shown in Figure 2, are similar to those for BM-2C and BM-1B, respectively. The target asphalt rubber contents for the ARB and ARS mixtures were 7.5% and 9.9%, respectively.

Table 2. Hot-mix asphalt mixture design details

Mixture/aggregate	**BM-1B**		**BM-2C**	
blend properties	**Required**	**Design**	**Required**	**Design**
Asphalt content (%)	-	4.75	-	4.25
Air voids (%)	3-5	3.12	3-5	3.15
VMA (%)	KDOT Zone II	11.4	KDOT Zone II	9.26
VFA (%)	65-80	72	65-80	65.9
Stability (lbs)	750+	2402	1500+	3490
Density (cft)	-	148	-	148.7
Filler-to-binder ratio	-	1.48	-	1.66
Crushed aggregates	75% min	75%	50% min	75%

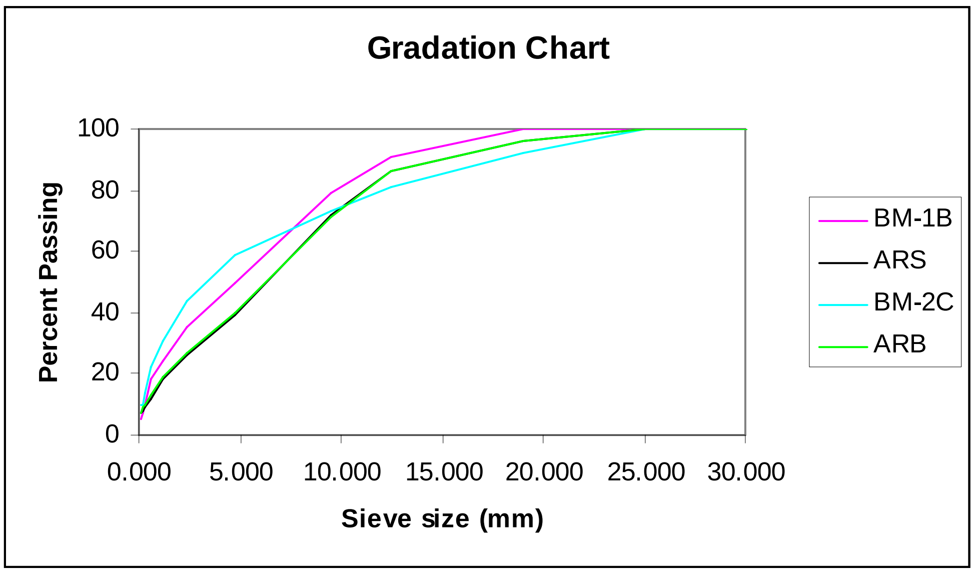


Figure 2. Blend gradations of the hot-mix asphalt mixtures

PERFORMANCE EVALUATION OF THE TEST SECTIONS

As early as in 1995, some test sections were experiencing premature failure. The predominant distresses reported were rutting followed by fatigue cracking.

Rutting

Rutting is a longitudinal depression in the wheel paths of the pavement surface that develops gradually with increasing load repetitions. Rutting can result from the permanent deformation in any or all of the pavement layers and subgrade. Three types of rutting were considered in the evaluation of the SPS-1 test sections: densification or one-dimensional consolidation; plastic/shear flow of HMA materials from wheel loads; and mechanical deformation or subsidence in the base, subbase, or subgrade, accompanied by distress patterns at the surface when the mix is too stiff or rigid.

Soon after construction was completed, two sections with an ARS, KDOT #3 and KDOT #5, started to show the first signs of rutting (Gisi and Dietz 1997). The severity of rutting varied from section to section. Sections SHRP #1 (7.6-in. BM-1B/8.5-in. DGAB), SHRP #2 (4-in. BM-1B/12.3-in. DGAB), SHRP #4 (4.1-in. BM-1B/4.1-in.PATB/3.7in. DGAB), KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG), and KDOT #2 (1.5-in. BM-1B/4-in. BM-2C/11-in. DGAB) were experiencing one-dimensional consolidation (densification)-type of rutting. This may indicate rutting in any or all layers of the sections.

Sections KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG) and KDOT #2 (1.5-in. BM-1B/4-in. BM-2C /11-in. DGAB) had rutting in the wheel paths with ride quality in some areas being affected. Sections KDOT #3 (1.5-in. ARS/9.9-in. BM-2C), KDOT #5 (1.5-in. ARS/2.5-in. ARB/8-in. DGAB), and KDOT #6 (1.5-in. ARS/2.5-in. ARB/8-in. BM-2C), which are the KDOT asphalt rubber sections, had consolidation as well as lateral movement of asphalt mixture materials (shear displacement).

Some of the rutting in section KDOT #3 (1.5-in. ARS/9.9-in. BM-2C) was due to densification of the ARS on the wheel path, but not all was associated with it. Hot weather during the summer also contributed to rutting on this section. Rutting in section KDOT #6 (1.5-in. ARS/2.5-in. ARB/8-in. BM-2C) was due to an unstable base and ARS densification in the wheel paths. Some of the rutting in sections SHRP #1 (7.6-in. BM-1B/8.5-in. DGAB) and SHRP #2 (4-in. BM-1B/12.3-in. DGAB) was due to asphalt mixture densification on the wheel paths, and some of the rutting in SHRP #2 (4-in. BM-1B/12.3-in. DGAB) was attributed to poor subgrade compaction. These sections were repaired primarily by milling and overlay (mill 4 inches and inlay with 4 inches of BM-2). Figures 3 through 10 show the measured rut depth trends on some of the sections up to 1996.

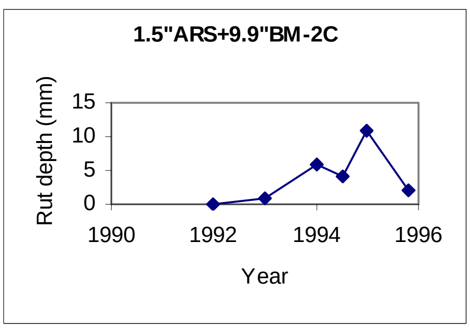


Figure 3. Rutting on section KDOT #3

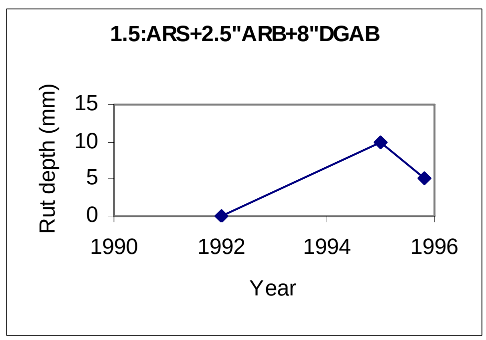


Figure 4. Rutting on section KDOT #5

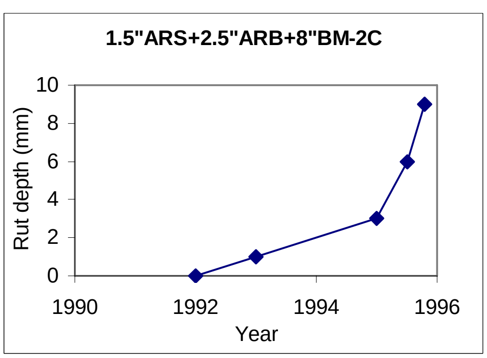


Figure 5. Rutting on section KDOT #6

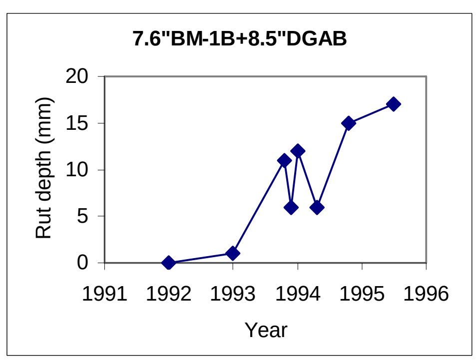
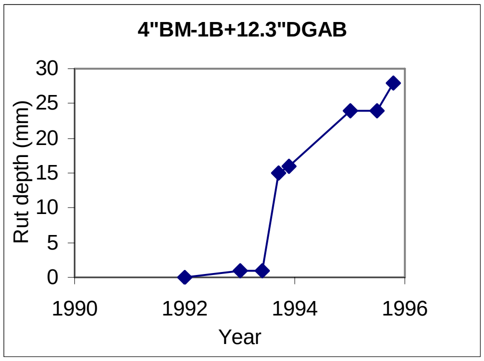


Figure 6. Rutting on section SHRP #1



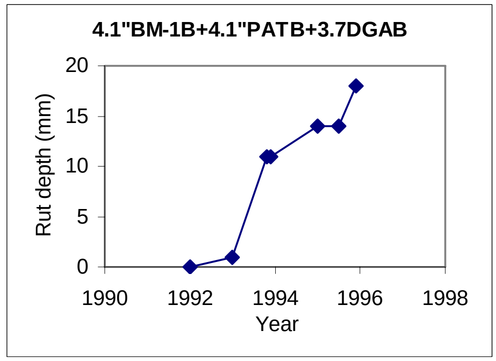
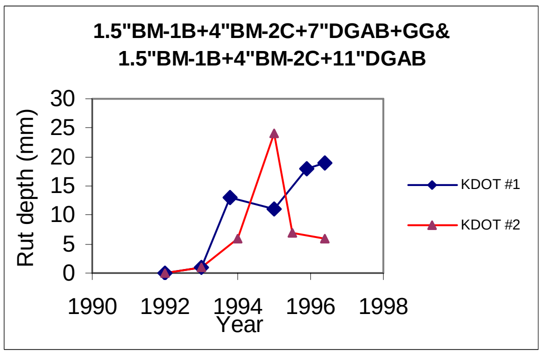


Figure 7. Rutting on section SHRP #2 **Figure 8. Rutting on section SHRP #4**



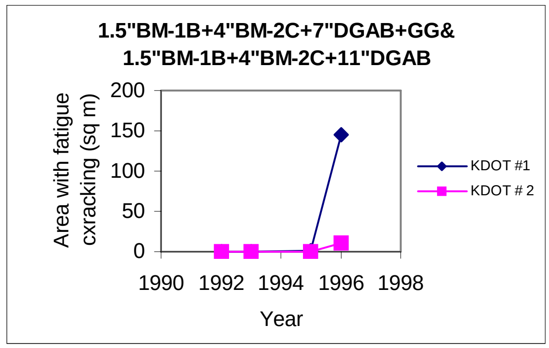


Figure 9. Rutting on section KDOT #1 & #2 **Figure 10. Fatigue cracking on section KDOT #1 & #2**

Cracking

Sections KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG) and KDOT #2 (1.5-in. BM-1B/4-in. BM-2C/11-in. DGAB) were observed to have major cracking problems. The DGAB fines appeared through the cracks in the ruts. These sections had indicated rutting failure and cracking. Laboratory testing on the cores showed that the rutting in section KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG) was due to unstable surface and base layers. KDOT #1 and KDOT #2 were milled up to four inches and inlaid with a BM-2 mix in 1997. A three-inch BM-2 overlay was then placed on both sections.

Sections SHRP #1 (7.6-in. BM-1B/8.5-in. DGAB), SHRP #2 (4-in. BM-1B/12.3-in. DGAB), and SHRP #4 (4.1-in. BM-1B/4.1-in. PATB/3.7-in. DGAB) indicated rutting as well as map or alligator cracking. Section SHRP #2 (4-in. BM-1B/12.3-in. DGAB) had one-inch–deep ruts in both wheel paths in 1996. It was suggested that a four-inch mill and inlay be used if the deterioration continued. The deficiencies mentioned above were summarized by the KDOT Geotechnical Unit in Table 3.

Sections KDOT #3 (1.5-in. ARS/9.9-in. BM-2C) and KDOT #5 (1.5-in. ARS/2.5-in. ARB/8-in. DGAB) were milled and overlaid in December of 1995. Test sections KDOT #6 (1.5-in. ARS/2.5-in. ARB/8-in. BM-2C), SHRP #1 (7.6-in. BM-1B/8.5-in. DGAB), SHRP #2 (4-in. BM-1B/12.3-in. DGAB), and SHRP #4 (4.1-in. BM-1B/4.1-in. PATB/3.7-in. GAB) showed signs of rutting along with block cracking and were repaired in the summer of 1996.

Most of the sections that experienced premature distresses by 1996 had a design life of more than 10 years (Table 1) except for section SHRP #2 (4-in. BM-1B/12.3-in. DGAB), which had an estimated life of 4.5 years. The whole SPS-1 project was overlaid with a one-inch BM-1T overlay in the fall of 2000.

Table 3. Assessment of deficiencies for SHRP and KDOT sections (Gisi and Dietz 1997)

Test section	Deficiencies
SPS-1 control section	Minor rutting, longitudinal cracking in middle of the lane, map cracking near the west end of the west bound lane
SHRP #1 (7.6-in. BM-1B/8.5-in. DGAB)	Milled 4 inches, inlaid 4 inches (BM-2) in 1996; no visible distresses
SHRP #2 (4-in. BM-1B/12.3-in. DGAB)	Milled 4 inches, inlaid 4 inches (BM-2) in 1996; no visible distresses in 1997
SHRP #3 (7.3-in. BM-1B/ 7.3-in. ATB/4-in. DGAB)	Map cracking in the wheel paths
SHRP #4 (4.1-in. BM-1B/ 4.1-in. PATB/ 3.7-in. DGAB)	Milled 4 inches, inlaid 4 inches (BM-2) in 1996; no visible distresses in 1997
SHRP #5 (7.6-in. BM-1B/3.6-in. PATB/ 7.9-in. DGAB)	Minor rutting and map cracking in the wheel paths
SHRP #6 (7-in. BM-1B/3.6-in. PATB/ 11.9-in. DGAB/)	Minor rutting and map cracking in the wheel paths
SHRP #7 (7-in. BM-1B/3.8-in. ATB/ 3.9-in. PATB)	Minor rutting and map cracking in the wheel path
SHRP #8 (5"BM-1B/12"ATB/3.6PATB)	Map cracking in the wheel paths
SHRP #9 (4-in. BM-1B/ 8.5-in. ATB/3.6-in. PATB)	0.5-in. rutting and map cracking in the wheel paths
SHRP #10 (3.9-in. BM-1B/ 3.8-in. ATB/4.1-in. DGAB)	1.0-in. rutting and map cracking in the wheel paths
SHRP #11 (3.6-in. BM-1B/7.7-in. ATB)	Map cracking in the wheel paths
SHRP #12 (6.8-in. "BM-1B/12.1-in. ATB)	Map cracking in the wheel paths
KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/ 7-in. DGAB/GG)	1.5-in. rutting, map cracking in wheel paths; surface staining, and maintenance patching has been done; scheduled for milling, inlaying, and overlaying in summer 1997
KDOT #2 (1.5-in. BM-1B/ 4-in. BM-2C/11-in. DGAB)	1-in. rutting, map cracking in wheel ruts, longitudinal cracking, surface staining; minor maintenance patching has been done; scheduled for milling, inlaying, and overlaying in summer 1997
KDOT #3 (1.5-in. ARS/9.9-in. BM-2C/ 8-in. DGAB)	Milled 4 inches, inlaid 4 inches (BM-2) in 1995; rutting is beginning to appear
KDOT #5 (1.5-in. ARS/2.5-in. ARB/ 8-in. DGAB)	Milled 4 inches, inlaid 4 inches (BM-2), overlaid 3 inches
KDOT #6 (1.5-in. ARS/2.5-in. ARB/ 8-in. BM-2C)	Milled 4 inches, inlaid 4 inches (BM-2) in 1996; no visible distresses

International Roughness Index

The ride quality of the test sections was assessed by the International Roughness Index (IRI) values. Test sections with thicker and/or treated base layers had slightly lower IRI values than those with thin and/or untreated base layers. Figures 11 to 17 show the measured IRI values up to 1997.

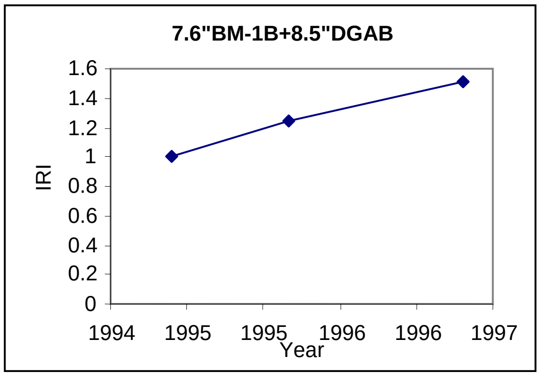


Figure 11. IRI on section SHRP #1

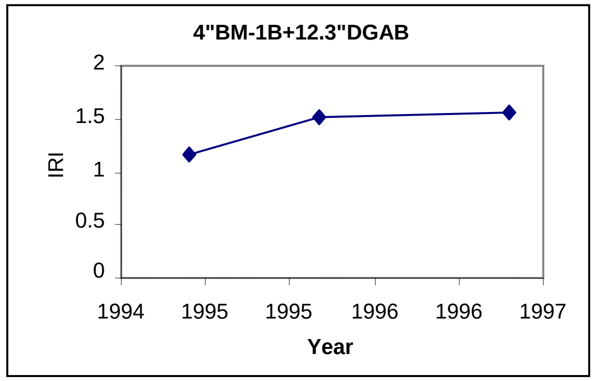


Figure 12. IRI on section SHRP #2

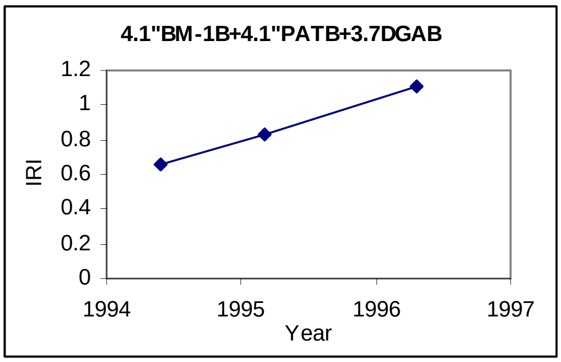


Figure 12. IRI on section SHRP #4

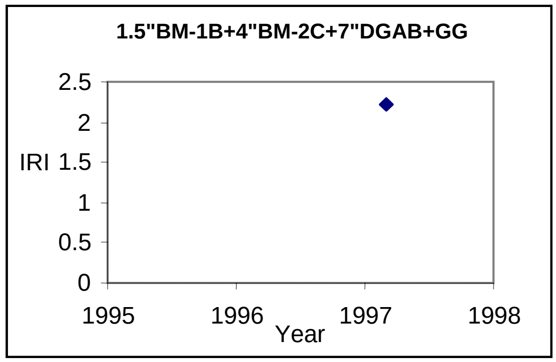


Figure 13. IRI on section KDOT #1

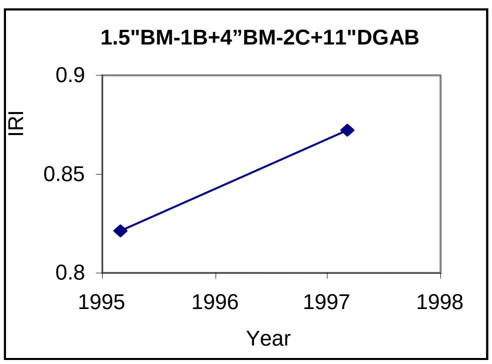


Figure 14. IRI on section KDOT #2

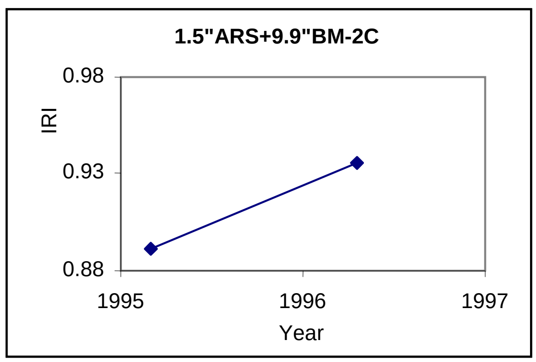


Figure 15. IRI on section KDOT #3

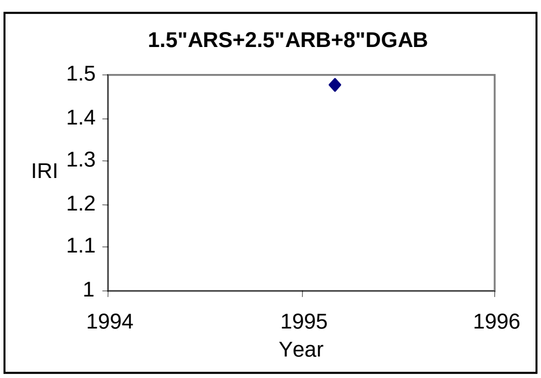


Figure 16. IRI on section KDOT #5

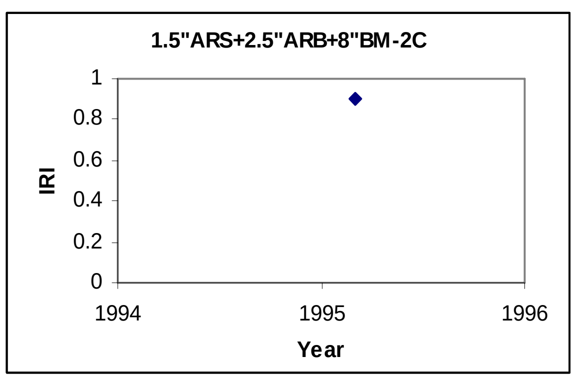


Figure 17. IRI on section KDOT #6

ASSESSMENT OF OBSERVED DEFICIENCIES

Field and Laboratory Measurements

Field Measurements

A rut-depth survey was done in January of 1996 with a rod and level. Cores from the inner and outer wheel paths were also retrieved. The falling weight deflectometer (FWD) tests were carried out on sections KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG) and KDOT #2 (1.5-in. BM-1B/4-in. BM-2C /11-in. DGAB) in March 1997 (Gisi and Dietz 1997). DARWin 3.0 software was used to backcalculate the effective pavement and subgrade resilient moduli using the 1993 AASHTO design guide algorithms. The deflections obtained at both the outer wheel path (OWP) and the mid-lane path (MLP) locations were analyzed.

For section KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB), the subgrade modulus at the OWP (5,330 psi) was somewhat lower than that at the MLP location (6,750 psi). The effective pavement modulus at the OWP location (43,740 psi) was more than 50% lower than that at the MLP location (78,230 psi). Thus, the effective structural number (SN_{eff}) at the OWP location was much lower than that at the MLP location (1.98 vs. 2.41). On section KDOT #2 (1.5-in. BM-1B/4-in. BM-2C/11-in. DGAB), the backcalculated subgrade moduli at both locations were comparable (6,960 psi and 7,100 psi). The effective pavement moduli (93,170 psi and 87,100 psi) were also comparable, as were the effective structural numbers (3.37 vs. 3.29). It is to be noted that the geogrid in section KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG) was assumed to be equivalent to 4 inches of DGAB in the original design. The distress survey and FWD test results do not appear to support this assumption.

From the FWD analysis, it was determined that section KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB/GG) would require an additional 6 inches of hot mix asphalt overlay to support the traffic (2,962,000 ESALs) for the next ten years and section KDOT #2 (1.5-in. BM-1B/4-in. BM-2C/11-in. DGAB) would require an additional 2 inches for the same time period.

Physical Properties

The density and permeability tests conducted on the cores retrieved from the test sections were used to analyze the void characteristics. Test results showed that the density was greater in the wheel path than outside of the wheel paths. This indicates that there were fewer air voids in the wheel path, and shear flow of asphalt materials might be responsible for this. Density comparisons were also used to measure rutting resulting from densification of AC material. The amount of rutting from the laboratory cores showed slightly lower rutting potential when compared to the transverse roadway profiles obtained from the rod-and-level survey.

Mechanical Tests

Stability tests were performed after the permeability tests. Cores with low permeability were remixed and compacted to the nearest field density by the Corps of Engineers Gyratory Testing Machine (GTM). The GTM results are useful for identifying mixtures susceptible to localized shear failure or lateral movement of hot-mix asphalt material. A gyratory stability index (GSI) was measured from the plot of the compaction curve by empirically relating the number of revolutions required to reproduce the in-place density. If the GSI value exceeds one, the mix is unstable on the rich side of the compaction curve, and progressive rutting would occur under hot weather conditions. The results showed that the ARB mix in section KDOT #5 (1.5-in. ARS/2.5-in. ARB/8-in. DGAB) and the BM-1B and BM-2C mixes in section

KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB) were unstable and might have contributed to rutting on these sections.

Gradation tests were run on the DGAB materials for sections KDOT #1 (1.5-in. BM-1B/4-in. BM-2C/7-in. DGAB) and KDOT #2 (1.5-in. BM-1B/4-in. BM-2C/11-in. DGAB). The results showed that the materials have similar gradations and that they fall well within the required DGAB specifications. However, material passing the No. 200 sieve lies near the maximum percent passing of the grading band.

On both KDOT #1 and KDOT #2, the tensile strength ratios (TSR) determined from the modified Lottman tests on the cores were initially found to be very low for the BM-2C mixes (42% for KDOT #1 and 30% for KDOT #2). The cores were then reheated and recompacted to densities comparable to the core densities and the tests were rerun. For the recompacted BM-2C mixes, the TSR was 57% corresponding to 57% saturation, which was lower the required minimum value. The TSR for BM-1B was 78.9%, which is slightly lower than the minimum 80% required for the Superpave mixes. Therefore the surface mix BM-1B indicated non-susceptibility to stripping, while the base mix BM-2C might be susceptible to stripping.

LTPP EVALUATION OF KANSAS SPS-1 PROJECT

The LTPP program recently published a report on the initial evaluation of all SPS-1 projects (Von Quintus and Simpson, 2003). The report contains the following findings related to the performance of the SPS-1 project in Kansas:

- The excessive moisture in the subgrade caused problems throughout the project. The contractor had difficulty compacting the material to proper density. Fly ash was used to stabilize the subgrade. This resulted in sections not having uniform, homogeneous platforms.
- The elevation survey indicated that all sections except SHRP #7, SHRP #10, and SHRP #12 had thickness deviations of more than 0.24 in. (6 mm).
- On average, sections with unbound aggregate base layers were found to have higher rut depths, while those with ATB layers had the lowest.
- Some sections had high IRI values immediately after construction because of fine-grained subgrade soil and construction difficulties.
- Sections with unbound aggregate layers had slightly more fatigue cracking than those with ATB.
- Sections with a thick HMA surface layer exhibited a greater average area of fatigue cracking than did the companion sections with a thin HMA surface layer.

CONCLUSION

The results of this study show that rutting failure on most sections in the Kansas SPS-1 project can be attributed to a deficiency in the mixtures. Cracking was also observed on some of the sections that failed due to excessive rutting. Some valuable conclusions can be drawn from this study:

- The SPS-1 project faced construction difficulties due to excessive moisture in the subgrade and subbase. Variable densities were obtained for those layers.
- Although traffic is known to be a major contributor to pavement failure, the traffic data for these sections indicates that the failures that occurred in some of the sections by 1995 had very little to do with traffic.
- The asphalt rubber mixtures performed very poorly. The asphalt rubber sections rutted immediately after construction, and rutting severity increased rapidly during the first three years. These sections were the first ones to be rehabilitated.
- Most of the sections with dense-graded aggregate bases experienced premature distresses. The sections with ATB layers had lower measured rut values than those with DGAB. The HMA

pavements with aggregate base layers had slightly more fatigue cracking than those with ATB layers. The HMA pavements with thicker base layers are smoother than those with thin layers.

- BM-1B and BM-2C were reported as unstable mixes and BM-2C did not provide good moisture resistance. Furthermore, the geogrid used in one section did not make any significant difference, compared to a companion section without it.
- Most sections, which had in-pavement drainage, did not experience premature failure within the first three years of evaluation, compared to the failed sections without in-pavement drainage. These sections were smoother with less fatigue cracking and lower rut depths.
- The test sections with thick HMA surface layer exhibited a greater average area of fatigue cracking than did the companion sections with thin HMA surface layers.

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Transportation Security (Invited Presentation)

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ABSTRACT

This presentation will provide an overview of current issues in transportation security research.

Note: Contact the presenter above for more information on this topic.

Key words: security—transportation

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

Effect of Four-Lane to Three-Lane Conversions on Crash Frequencies and Crash Rates (Invited Presentation)

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ABSTRACT

This presentation reports on a recent full Bayes analysis of the safety effectiveness of four-lane to three-lane conversions. The study was performed as part of a creative component by a graduate student, Wen Li, working with Dr. Alicia Carriquiry of the Iowa State University Department of Statistics and Dr. Michael Pawlovich of the Iowa Department Transportation Office of Traffic and Safety.

The Iowa study refutes a recent Federal Highway Administration (FHWA) Highway Safety Information System (HSIS) study that found little safety benefit from four-lane to three-lane conversions. The FHWA study fitted negative binomial regression models to the crash frequencies at each project site. The Iowa study was based on a much richer database and a superior statistical analysis. This was the first time a full Bayes analysis has been applied to a safety countermeasure. The full Bayes analysis shows some very distinct advantages over the results obtained from fitting negative binomial regression models.

Note: Contact the presenter above for more information on this topic.

Key words: four-lane to three-lane conversions—full Bayes—negative binomial regression—safety countermeasures—safety effectiveness—statistical analysis

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INTRODUCTION

As part of a creative component from the Iowa State University statistics department, in cooperation with the Iowa Department of Transportation, Office of Traffic and Safety (TAS), a full Bayes analysis of the reduction in crash frequency due to four-lane to three-lane conversions in Iowa was conducted. The study utilized monthly crash data and estimated volumes obtained from TAS for 30 sites, 15 treatment (sites 1 through 15) and 15 control (sites 18 through 32), over 23 years (1982–2004). The sites had volumes ranging from 2,030 to 15,350 during that timespan and were largely located in smaller urbanized areas (see Table 1 for full site descriptions).

Table 1. Site locations

Site ID	City	Site description	City pop. (2000)	ADT (2000)
1	Storm Lake	Iowa 7 from Lake Ave. to Lakeshore Dr.	10,076	7,503
2	Clear Lake	US 18 from N 16 st. W to N 8th St.	8,161	10,403
3	Mason City	Iowa 122 from West intersection of Birch Drive to a Driveway	29,172	7,800
4	Osceola	US 34 from Corporate limits on east side to where highway divides to 4 lanes on west side	4,659	8,172
5	Manchester	Iowa 13 from River St. to Butler St.	5,257	9,400
6	Iowa Falls	US 65 from City Limits - ? to Park Ave.	5,193	10,609
7	Rock Rapids	Iowa 9 from S Greene St. to Tama St.	2,573	4,766
8	Glenwood	US 275 from MP 36.2 to MP 37.42	5,358	6,410
9	Des Moines	Beaver Ave from Urbandale Ave. to Aurora Ave.	198,682	13,695
10	Council Bluffs	US 6 from McKenzie Ave. west 1300 ft.	58,268	11,000
11	Blue Grass	Old US 61 from Oak Lane to 400' W of Terrace Drive	1,169	9,155
12	Sioux Center	US 75 from 200' South of 10th St. S. to 250' North of 9th St. NW	6,002	8,942
13	Indianola	Iowa 92 from South R St. to Jct. of US 65/69	12,998	13,288
14	Lawton	US 20 from 100' east of Co. Rd. Eastland Ave. to 1130' West of Co. Rd. Emmet Ave.	697	9,237
15	Sioux City	Transit Ave. from Vine Ave. to just west of Paxton St. at curve	85,013	9,608
18	Storm Lake	Iowa 7 from Lake Ave. to Barton St	10,076	8,790
19	Le Mars	US 75 from 0.01 miles north of 3rd St NW to 0.36 miles SW of 12th St SW	9,237	10,880
20	Cedar Falls	Green Hill Road from 0.10 miles east of IA 58 to 0.09 miles west of Cedar Heights Dr.	36,145	2,768
21	Jefferson	Iowa 4 from National Ave to 0.13 miles north of 250th Ave	4,626	5,685
22	Harlan	Iowa 44 from US 59 to 6th St	5,282	6,981
23	Norwalk	Iowa 28 from 0.03 miles south of Gordon Ave to 0.04 miles south of North Ave	6,884	7,679
24	Belmond	US 69 from 0.38 miles north of Main St to 0.58 miles south of Main St	2,560	3,734
25	Harlan	Iowa 44 from US 59 to 6th St	5,282	6,981
26	Des Moines	Hickman Road - 40th Place east to 0.07 miles west of W 18th St	198,682	13,953
27	Ames	13th Street from 0.09 miles east of Stange Road to 0.07 miles west of Crescent Circle Dr.	50,731	10,711
28	Mapleton	Iowa 141 from 0.02 miles north of Sioux St. to 0.08 miles south of Oak St.	1,322	3,007
29	Algona	US 169 from 0.07 miles south of US 18 to 0.23 miles south of Irvington Rd.	5,741	7,263
30	Oskaloosa	Iowa 92 from 0.12 miles east of IA 432 to 0.07 miles west of Hillcrest Dr	10,938	11,143
31	Merrill	US 75 from 0.05 miles north of 2nd St to 0.18 miles north of Jackson St	754	7,774
32	Sioux City	S. Lakeport from 4th Ave to Lincoln Way	85,013	15,333

Each treatment site had different known intervention dates (see Table 2); therefore, the number of before and after crash records varied from site to site. Individual control sites were matched to each treatment site to provide a control sample similar to the treatment sample; these matches are shown in Table 2 using the site ID and paired ID columns.

Table 2. Site descriptive information

Site ID	Paired ID	Route	Lanes	Length	Comp. year
1	18	IA 7	3	1.41	1993
2	19	US 18	3	1.51	2003
3	20	IA 122	3	1.78	2001
4	21	US 34	3	2.04	2001
5	22	IA 13	3	0.35	2001
6	23	US 65	3	1.23	2002
7	24	IA 9	3	0.35	1998
8	25	US 275	3	1.09	1998
9	26	Local	3	1.19	1999
10	27	US 6	3	0.20	2000
11	28	Local	3	0.72	1999
12	29	US 75	3	1.52	1999
13	30	IA 92	3	1.57	1999
14	31	US 20	3	0.64	2000
15	32	Local	3	0.77	2000
18	1	IA 7	4	0.71	1993
19	2	US 75	4	1.80	2003
20	3	Local	4	1.80	2001
21	4	IA 4	4	2.40	2001
22	5&8	IA 44	4	1.20	2001
23	6	IA 28	4	0.80	2002
24	7	US 69	4	0.90	1998
25	5&8	IA 44	4	1.20	1998
26	9	Local	4	1.50	1999
27	10	Local	4	0.33	2000
28	11	IA 141	4	0.70	1999
29	12	US 169	4	2.00	1999
30	13	IA 92	4	1.50	1999
31	14	US 75	4	0.50	2000
32	15	Local	4	1.20	2000

In general, both the treatment and control site crash histories show a reduction in crashes.
However, the reduction in treatment site crash frequency and rate after intervention are significantly more marked than at the comparison sites (see Figures 1 and 2, where the vertical bar represents the intervention date for the treatment site). This differs from a previous four-lane to three-lane study (Huang et al. 2002; HSIS 2004; HSIS 2005), recently published in *ITE Journal*, whose data, even from a descriptive statistical standpoint, indicated very little reduction or difference between the two groups. Additionally, because monthly crash frequencies were used for the present analysis, it was possible to account for the seasonal effects on crashes, which should be expected given the seasonal weather patterns in Iowa.

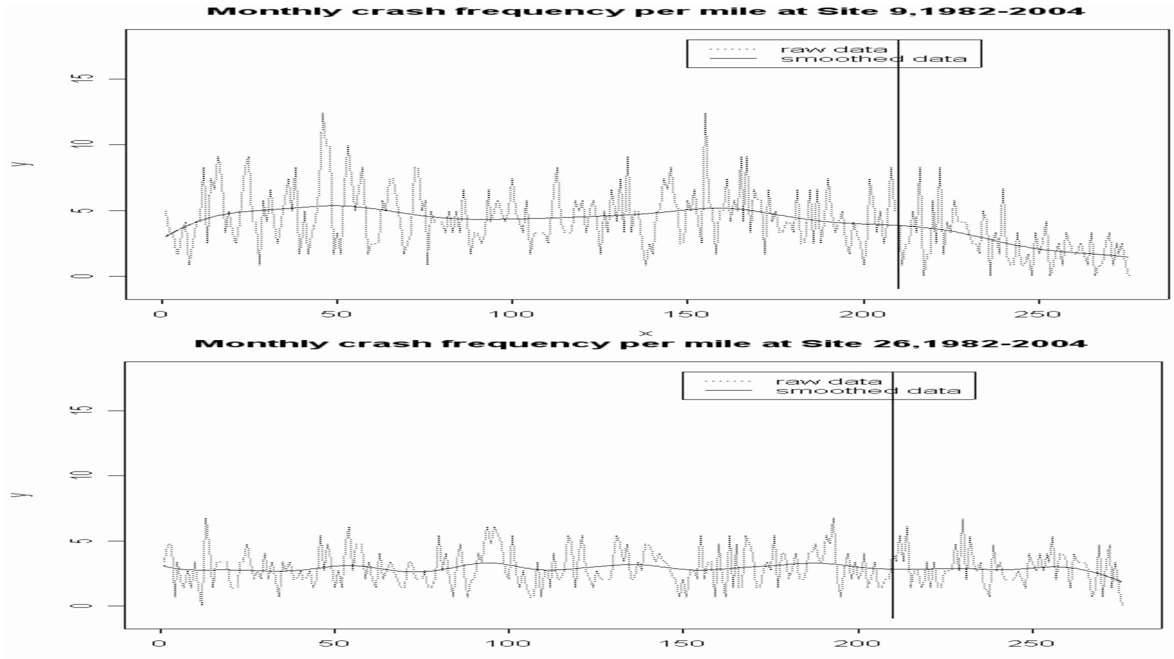

Figure 1. Observed and smoothed estimated monthly crash density for a sample pair of sites

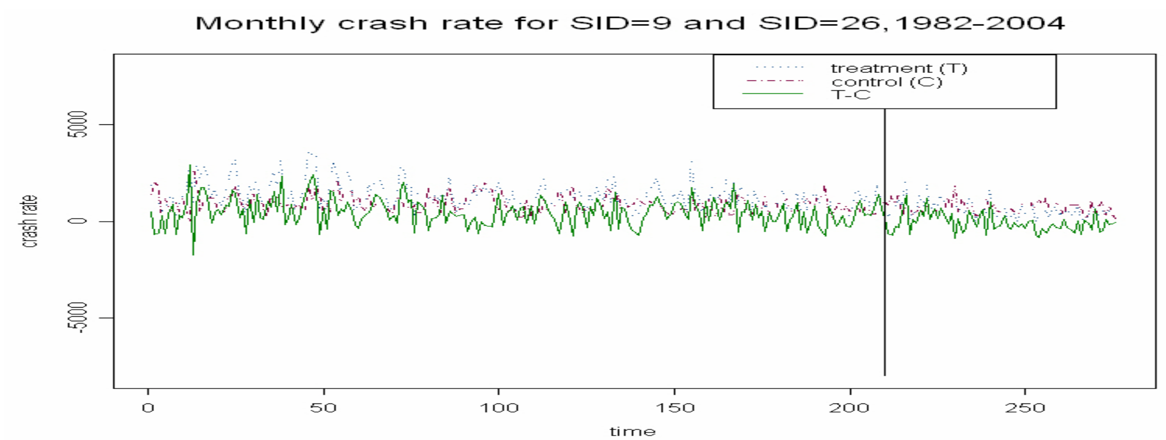

Figure 2: Difference in monthly crash rate for a sample pair of sites

Given the random and rare nature of crash events, a hierarchical Poisson model, where the log mean was expressed as a function of time period, seasonal effects, and a random effect corresponding to each included site, was fitted to the crash frequencies. To formulate the model, notation is first defined:

- i denotes site and takes on values 1 to 30
- t denotes month (or time period) and takes on values 1 to 276 (for i=17, t is from 1 to 252)
- y_{it} denotes the number of monthly crashes at site *i* during time period (month) *t*
- v_{it} is the estimated monthly average daily traffic (MADT) for site *i* at time period *t*
- id_i is a random effect corresponding to site *i*
- t_{0i} denotes the time period during which the intervention is complete for treated site *i* and (fictitiously) for the corresponding matched site

- $X_{1it} := \begin{cases} 1, \text{if site } i \text{ is treated at some time} \\ 0, \text{if site } i \text{ is in control group} \end{cases}$
- $S_{it} := \begin{cases} 1, \text{if t belongs to Winter (December, January, and February)} \\ 2, \text{if t belongs to Spring (March, April, and May)} \\ 3, \text{if t belongs to Summer (June, July, and August)} \\ 4, \text{if t belongs to Fall (September, October and November)} \end{cases}$
- $X_{2it} = \cos\left(\frac{2\pi \times S_{it}}{4}\right),\ X_{3it} = \cos\left(\frac{4\pi \times S_{it}}{4}\right),\ X_{4it} = \sin\left(\frac{2\pi \times S_{it}}{4}\right)$

We postulate that the number of monthly crashes at a site y_{it} is a Poisson random variable with mean $\lambda_{it}v_{it}$, and divided by 1,000 for numerical convenience:

$$y_{it} \sim Poi\left(\frac{\lambda_{it}v_{it}}{1000}\right) \tag{1}$$

At the second level, we model the log crash rate as a piecewise linear function of the covariates defined above, such that the function is continuous at the change point. The model is

$$\begin{aligned} \log(\lambda_{it}) = {} & \beta_1 + \beta_2 X_{1it} + \beta_3 t + \beta_4 (t - t_{0i}) I_{(t > t_{0i})} + \beta_5 X_{1t} t + \beta_6 X_{1t} (t - t_{0i}) I_{(t > t_{0i})} \\ & + \beta_7 X_{2it} + \beta_8 X_{3it} + \beta_9 X_{4it} + id_i \end{aligned} \tag{2}$$

where

$$id_i \sim N\left(0, \left(\tau_{bw}^2\right)^{-1}\right),\ I_{(t > t_{0i})} = \begin{cases} 1, \text{if } t > t_{0i} \\ 0, \text{if } t \le t_{0i} \end{cases} \tag{3}$$

and τ_{bw}^2 is the between-site precision, defined as the inverse of the between-site variance in monthly number of crashes.

A full Bayesian (FB) approach (Gelman et al. 2004; Pawlovich 2003) was adopted to estimate the model parameters. In the Bayesian approach, model parameters are treated as random variables and the goal is to estimate the distribution of the likely values of the parameters given prior and data information. The approach differs from classical methods in that distributions of likely values, rather than point estimates and standard errors of parameters, are obtained, and all results are conditional on the sample at hand. The empirical Bayes (EB) approach (Hauer, 1997) is a variant in which prior distributions are partially based on the sample.

Results indicate a 25.2% (23.2%–27.8%) reduction in crash frequency per mile and an 18.8% (17.9%–20.0%) reduction in crash rate. This differs again from the Huang study (Huang et al. 2002; HSIS 2004; HSIS 2005), which reported a 6% reduction in crash frequency per mile and an insignificant indication for crash rate effects. This difference is evident just by comparing the raw data from the two studies. The Iowa data, when graphed, indicates marked reductions, whereas the Huang data indicate very little difference. Based on these Iowa FB results and results from a simple before/after analysis done as part of a separate causal study, we are comfortable with the 25% and 19% reductions, especially as they fit practitioner expectations. Other benefits shown from a previous internal Iowa study on speeds, travel times, and delays on the Sioux Center conversion during a.m. and p.m. peak periods, indicate a 4–5 mph reduction in 85th percentile free flow speed and a 12–14% reduction in vehicles traveling more than 5 mph over the speed limit (i.e., vehicles traveling 35 mph or higher).

The differences between our analysis and the analysis performed by Huang (Huang et al. 2002; HSIS 2004; HSIS 2005) are several and may explain the diverging results. First, even the descriptive analysis of the raw data suggests that the effect of conversion on Iowa roads was much more dramatic than on the roads considered in the Huang study. See, for example, the descriptive statistics presented in Table 3 of that report. Second, Huang fitted an ordinary linear regression model to the expected crash frequencies, meaning that a single slope for expected frequency on time was assumed for the entire study period. We extended the model and allowed for different slopes during the before and the after periods explicitly by including a change-point in the model and for the interaction of treatment and slope. Notice that as a result, our model allows for a slight increase in crash frequency during the months immediately preceding the conversion and during the months immediately following the conversion. Finally, we included a longer time series of crash frequencies, as we included 23 years of data on almost all sites in the study. By analyzing monthly data, we were also able to account for seasonal variability in crash frequency and traffic volume; while essential in Iowa, where seasonal variation in driving conditions is marked, seasonal variability may not be as critical in a study conducted in the northwestern region of the country. Huang's study, though it began with 12 treatment sites and 25 control sites, was reduced to 8 treatment sites and 14 control sites due to the unavailability of data. Additionally, Huang utilized only 3 years of data for both the before and after periods.

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Access Management Plan and Program for the Des Moines Metropolitan Area

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ABSTRACT

According to the Iowa Department of Transportation's crash database (GIS-ALAS), between 5% and 10% of all crashes in Iowa occur at commercial driveways. Most of these occur at arterials within municipalities. In recent years, nearly one-quarter of these crashes occurred in the Des Moines metropolitan area, making the area a prime candidate for improved access management. Case study research in Iowa has shown that access management is an effective highway safety tool: well-managed routes can be 40% to 50% safer than poorly managed routes. The Des Moines metropolitan area has many miles of older arterials constructed before access management was considered.

This paper describes a cooperative effort of the Des Moines Area Metropolitan Planning Organization (DMAMPO) and the Center for Transportation Research and Education (CTRE) at Iowa State University to develop an access management study and program for the metro area. The overall goal of the study is to use the knowledge developed to make improvements that will reduce access-related crashes. This project will also help local officials better consider access to avoid future safety and operational problems.

The major task described in this paper was to identify and rank the Des Moines metro area's access management problems using Iowa's GIS-ALAS database and other geospatial data. This process was developed to identify arterial segments in the metro area most promising for further investigation in terms of access management.

Key words: access management—problem identification—corridor ranking

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BACKGROUND

The Importance of Access Management

Access management has been shown to be highly valuable in improving highway safety and maintaining traffic flow and travel speeds while having a neutral impact on adjacent land and businesses. Well-managed urban arterial roadways have been shown to be around 40% to 50% safer than poorly managed urban arterials on a per-vehicle-mile basis. Mean travel speeds are significantly higher on well-managed urban streets and traffic service levels are significantly higher. On the other hand, studies that have documented the business impacts of higher levels of access management have found little or no impact on business sales, business turnover, new business development, or customer satisfaction (Maze and Plazak 1997). Businesspersons often express great concern about access management, but the actual impacts appear rather benign. This means that stakeholder education is a very important component of a comprehensive access management program (Maze and Plazak 1997).

Access Management in Iowa

The Iowa Department of Transportation has been very active in the area of access management for the past several years. The Iowa Safety Management System Steering Committee has considered access management as an important tool for improving highway safety in Iowa. As such, it has commissioned several extensive research and education projects designed to determine the impacts of access management and to promote access management. Iowa's state highway design standards and unique Statewide Urban Design and Specifications (SUDAS) program both contain extensive access management material.

RESEARCH OBJECTIVES AND PROJECT OVERVIEW

The main purpose of the Des Moines Access Management Plan and Program project was to develop a systematic approach to access management in a medium-sized metropolitan area. Des Moines (with a metropolitan population of around 500 thousand persons) is the largest such area located entirely within Iowa, but is a mid-sized metropolitan area by national standards.

The major tasks of this project were as follows:

- Problem identification using Iowa's crash database (GIS-ALAS) and other geospatial data
- Development of access management elements for the metropolitan planning organization's (MPO) long-range transportation plan and the transportation program
- Integration of access management planning with local land use planning
- Education of local transportation and land use professionals and local elected officials regarding access management
- Documentation of the planning and programming process

The Des Moines Metropolitan Access Management Plan and Program project represented a three-way partnership between the Iowa Department of Transportation (Iowa DOT) Office of Traffic and Safety, the Des Moines Area Metropolitan Planning Organization (DMAMPO), and the Center for Transportation Research and Education (CTRE) at Iowa State University. The Iowa DOT was the main source of funding for the project (with help from the Midwest Transportation Consortium); the Iowa DOT also served as the technical monitor for the project. The DMAMPO staff concentrated on two issues in the project: development of access management elements for its planning and programming activities and integration of access management and local land use planning. CTRE focused its activities on problem identification and education of stakeholders, two areas in which it had previous experience.

This paper concentrates on describing task 1, the problem identification and ranking process. This process was developed to identify arterial segments in the metro area most promising for further investigation in terms of access management. The project, begun in Summer 2003 and completed in Summer 2004, was designed and documented to be replicable in the other Iowa metropolitan areas. Other metropolitan areas in Iowa with high proportions of the statewide total of access-related crashes are Cedar Rapids, Davenport, Iowa City, Ames, Council Bluffs, Sioux City, and Dubuque.

RESEARCH METHODOLOGY

To determine the most promising corridors in the Des Moines metropolitan area for access management treatments, access-related crash severity, access-related crash rates, and commercial driveway densities were calculated for 180 arterial roadway segments in the metro area. These segments were a maximum of two miles in length. The top 20 highest commercial driveway densities, top 20 highest access-related crash rates, and top 20 access-related crash severities were identified from these calculations. The following material briefly describes the process for each step and the top 20 results. The analysis for the Des Moines Metro area included all major arterial surface streets in the metro. This system was broken down into corridors. The corridors were further broken up primarily to identify specific locations that might be most promising for access management treatments. This resulted in a total of 180 analysis segments. The average segment length of the 180 segments was 2.04 miles.

KEY FINDINGS

Commercial Driveway Density

High commercial driveway densities are highly correlated with access-related crashes. Common probable access-related crashes along urban arterials with a high density of driveways are rear-end, right-turn, and left-turn crashes. Driveway density indicates how many driveways per mile exist along each corridor. Commercial driveways have been found in past research to be critical, as commercial land uses generate more trips compared to residential driveways. As the driveway density increases along a corridor, more conflict points are produced.

Commercial driveway densities were calculated for each corridor by taking the total commercial driveway count (per corridor) and dividing it by the total length of the corridor (in miles). This process utilized geographic information system (GIS) software, more specifically ESRI ArcGIS 8.3. Using one-meter resolution infrared orthophotos taken in 2002, driveway locations were identified manually along each corridor. The orthophotos were obtained from the Iowa State University Geographic Information Systems Support and Research Facility website (http://www.gis.iastate.edu/).

An ArcGIS shapefile was created, using points to indicate all driveway locations along the entire arterial system for the metro area. A current metropolitan land use coverage created by the MPO was then added to the GIS project to determine the driveways that provided access to commercial land use parcels. This was indicated in the GIS database for the total driveway inventory, thus allowing commercial driveways to be selected. Table 1 shows the top 20 highest ranked driveway densities for the Des Moines metropolitan area.

Table 1. Top 20 highest commercial driveway densities

Road Name	Corridor ID	Corridor Length (miles)	Driveway Count	Driveway Density
Ingersoll Ave.	64	3.05	92	30.14
Southwest 9th St.	7	6.31	145	22.98
Merle Hay Rd.	71	4.28	80	18.69
Southeast 14th St.	8	17.14	306	17.85
Euclid Ave.	47	5.35	95	17.77
Douglas Ave.	85	6.66	107	16.06
Delaware Ave./Northeast 22nd St.	46	6.53	103	15.77
East 30th St.	17	1.73	26	15.06
Beaver Ave.	41	4.92	72	14.63
Northeast 46 Ave./Broadway Ave./Southwest 8th St.	43	8.09	115	14.22
Crocker St.	45	0.22	3	13.39
2nd Ave.	3	9.45	123	13.01
University Ave.	80	20.28	246	12.13
Grand Ave.	52	13.58	157	11.56
Army Post Rd. (east)	38	9.62	111	11.54
Northeast 23rd Ave./Easton Boulevard	48	4.34	48	11.07
41st/42nd St.	22	1.76	19	10.80
Hubbell Ave.	55	10.87	115	10.58
72nd/73rd/8th Ave.	33	4.19	44	10.51
Adventureland Dr.	37	1.92	20	10.42

Access-Related Crash Rates

Crash rates per million vehicle miles traveled were figured for each study corridor. Crash rates were calculated using summed vehicle miles traveled (VMT) and crash frequencies for each corridor. ArcGIS was used to calculate crash rates for total crashes and probable access-related crashes. See Table 2 for a listing of the top 20 highest crash rates for probable access-related crashes. Only probable access-related crashes (e.g., multi-vehicle crashes at right angles or left angles or rear-end collisions) were used in these calculations. The calculation used to determine crash rates was as follows:

Access-related crash rate (per million vehicle miles)

=Frequency of probable access-related crashes (by segment)

* 1,000,000 summed VMT (by segment)

Table 2. Top 20 highest crash rates for probable access-related crashes, 1997–2000

Road	Corridor ID	Crash frequency: probable access-related crashes	VMT	Crash rate: probable access-related crashes
A21st St.	12	48	3,091,965	15.52
2nd/State St.	3	490	41,438,132	11.82
60th St.	27	47	4,057,939	11.58
15th St.	9	271	24,486,390	11.07
31st St.	18	137	13,550,260	10.11
West 19th St.	11	147	15,711,584	9.36
28th St.	14	48	6,460,900	7.43
74th St.	34	42	5,848,044	7.18
Guthrie Ave.	53	137	20,827,179	6.58
Northeast 64th St.	30	7	1,076,882	6.50
Crocker St.	45	18	2,928,316	6.15
Martin Luther King Jr Pkwy	72	320	59,626,721	5.37
42nd/41st St.	22	164	31,864,208	5.15
Keo Way	66	116	22,865,060	5.07
54th Ave.	24	45	9,591,980	4.69
35th St.	20	37	8,174,087	4.53
70th St.	31	31	6,910,472	4.49
56th St.	26	105	23,901,034	4.39
30th St.	16	73	17,071,780	4.28
West 15th St.	10	23	5,536,399	4.15

Access-Related Crash Severity

The severity of a crash was determined by placing dollar values on each crash. This allowed each corridor segment to be compared in terms of the highest dollar amount in crash severity as related to access issues. The crash data were obtained from the Iowa DOT's geospatial database. ArcGIS was used to query total crashes and probable access-related crashes from 1997 to 2000. Probable access-related crashes are those in which a rear-end, right-angle, or left-turn collision occurred. Crashes that occurred within the functional area of intersections were included, since they may or may not be access related, depending on the location of driveways.

The Iowa DOT assigns a severity ranking for each crash that represents an estimated cost to society of the crash. Table 3 shows each crash severity value, its description, and the dollar amount assigned. For this study, the crash locations were identified and crash severities were summed per corridor for both total and probable access-related crashes. See Table 4 for the top 20 highest ranked crash severities for probable access-related crashes.

Table 3. Description of loss values assigned to severity rankings

Severity Ranking	Level of Severity	Dollar Amount
1	Fatal	$1,000,000
2	Major Injury	$150,000
3	Minor Injury	$10,000
4	Possible Injury or Property Damage	$2500 each

Note: Data from Iowa DOT Office of Traffic and Safety

Table 4. Top 20 highest crash severities for probable access-related crashes, 1997–2000

Road Name	Corridor ID	Crash Severity
Southeast 14th S.	8	$143,650,000
University Ave.	80	$129,080,000
Hickman Rd.	54	$65,257,500
Grand Ave.	52	$64,632,500
Douglas Ave.	85	$46,087,500
Euclid Ave.	47	$42,659,000
SW 9th St.	7	$42,532,500
2nd/State St.	3	$41,355,000
Merle Hay Rd.	71	$40,490,000
86th St.	83	$39,732,500
Hubbell Ave.	55	$36,475,000
Army Post Rd. (east)	38	$32,122,500
Martin Luther King Jr Pkwy	72	$29,072,500
63rd St.	29	$28,530,000
15th St.	9	$23,577,500
Broadway Ave.	43	$22,615,000
Fleur Dr.	51	$21,917,500
100th St.	35	$21,570,000
Park Ave.	76	$19,912,500
6th Dr.	5	$19,167,500

Most Promising Segment Locations for Further Analysis

To identify the segments with the most promise for further analysis, the three rankings (crash rate, crash severity, and commercial driveway density) were summed for each corridor. This summed ranking served as the overall index ranking, which was then used to determine the top 20 high priority access management investigation segments in the Des Moines metropolitan area. See Tables 5 and 6 for the top 20 ranking system and resultant top 20 segment locations. Figures 1 and 2 show the locations of these segments within the Des Moines metropolitan area. Most of the top 20 segments are located in the older portions of the metro area where a common development pattern involves small frontage commercial lots along four-lane undivided arterial roadways. This includes the core areas of the cities of Ankeny, Clive, Des Moines, West Des Moines, and Windsor Heights. The top 20 ranked segments account for about one-third of all the access-related crash severity in the metro area, even though these segments include only 10% of the arterial mileage.

Table 5. Most promising segments for further analysis

Road Name	Severity Ranking	Crash Rate Ranking	Commercial Driveway Density Ranking	Sum of Rankings	Overall Ranking
Southwest 9th St.	7	26	2	35	1
Grand Ave.	1	9	30	40	2
Southwest 9th St.	25	15	1	41	3
Hubbell Ave.	21	20	5	46	4
Merle Hay Rd./ Merklin Way	6	27	18	51	5
Southeast 14th St.	4	41	8	53	6
East Euclid Ave.	11	32	13	56	7
University Ave.	13	17	29	59	8
Grand Ave.	19	36	6	61	9
East 14th St.	8	28	28	64	10
Douglas Ave.	12	43	14	69	11
Ankeny Blvd.	16	40	17	73	12
University Ave.	36	13	25	74	13
University Ave.	20	31	24	75	14
MLK Jr Pkwy	5	19	56	80	15
Hickman Rd.	10	39	34	83	16
Ingersoll Ave.	44	22	19	85	17
University Ave.	34	35	16	85	18
East Army Post Rd.	17	65	4	86	19
Southeast 14th St.	3	61	22	86	20

Table 6. Top 20 ranked segments

Rank	Road Name	Location	City
1	Southwest 9th St.	Thomas Beck Rd. to McKinley Ave.	Des Moines
2	Grand Ave.	29th St. to Hubbell Ave.	Des Moines
3	Southwest 9th St.	McKinley Ave. to East Army Post Rd.	Des Moines
4	Hubbell Ave.	East Grand Ave. to Guthrie Ave.	Des Moines
5	Merle Hay Rd./ Merklin Way	Meredith Dr. to Franklin Ave.	Des Moines
6	Southeast 14th St.	East Park Ave. to Army Post Rd.	Des Moines
7	East Euclid Ave.	2nd Ave. to East 14th St.	Des Moines
8	University Ave.	2nd Ave. to I-235	Des Moines
9	Grand Ave.	E.P. True Pkwy. to 56th St.	West Des Moines, Des Moines
10	East 14th St.	East Euclid Ave. to I-235	Des Moines
11	Douglas Ave.	Merle Hay Rd. to MLK Jr Pkwy.	Des Moines
12	Ankeny Blvd.	Northeast 11th Place to Oralabor Rd.	Ankeny
13	University Ave.	MLK Jr Pkwy. to 2nd Ave.	Des Moines
14	University Ave.	55th St. to MLK Jr Pkwy.	Des Moines
15	MLK Jr Pkwy.	Euclid Ave. to Ingersoll Ave.	Des Moines
16	Hickman Rd.	73rd St. to Beaver Ave.	Des Moines
17	Ingersoll Ave.	56th St. to 31st St.	Des Moines
18	University Ave.	86th St. to 55th St.	Clive, Windsor Heights, Des Moines
19	East Army Post Rd.	Fleur Dr. to Southeast 5th St.	Des Moines
20	Southeast 14th St.	I-235 to East Park Ave.	Des Moines

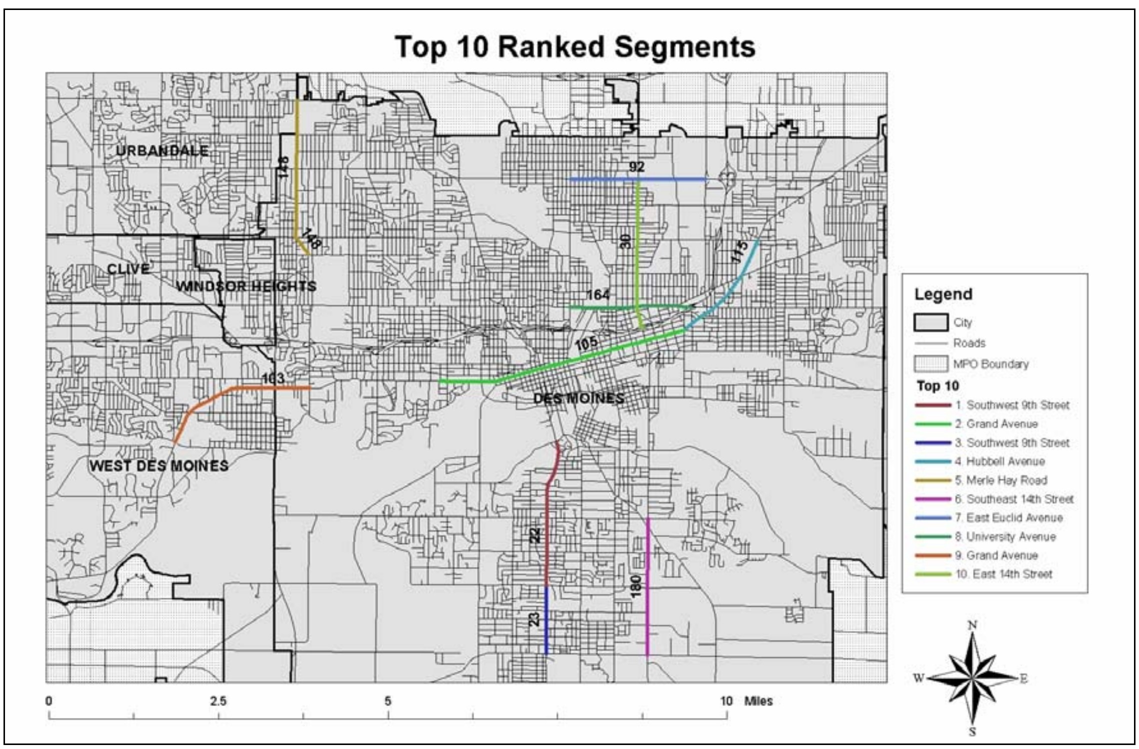

Figure 1. Top 10 ranked segment locations

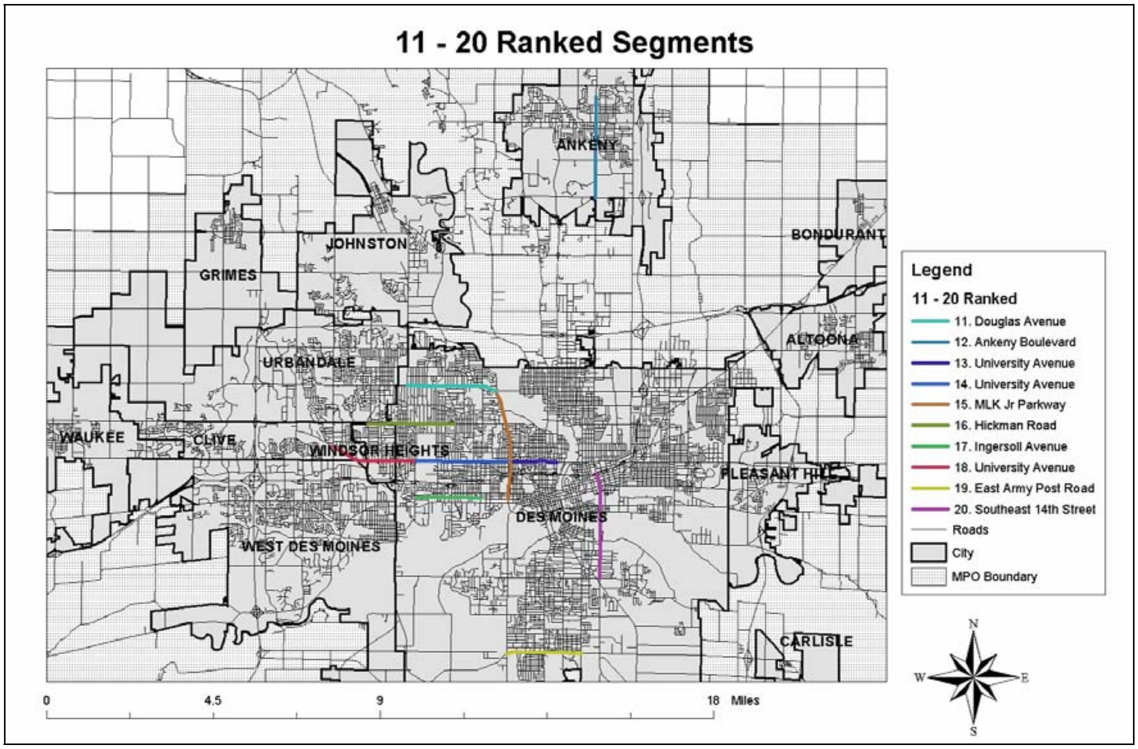

Figure 2. Number 11–20 ranked segment locations

CONCLUSIONS

Access management has been shown to have significant benefits in terms of highway safety and traffic flow and in protecting valuable public investments in roadway assets. Most access management issues occur on arterial roadways in metropolitan and other urban areas. Metropolitan planning organizations can play a key role in planning for and implementing access management through such means as problem identification, programming, coordination of transportation and land use planning, and education of stakeholders and decision makers. The project described in this paper represents a partnership between a state DOT, an MPO, and a university transportation research center to advance access management in one typical metropolitan area.

RECOMMENDATIONS

This paper presented the process and results from the first task of the Des Moines metro area access management planning process. The first task simply identified arterial roadway segments that appear promising for further investigation for access management improvements. Each of these segments will be further analyzed in terms of the types of access management issues present and the potential for treating them. Potential treatments to be examined may include driveway consolidation, addition of dedicated turn lanes, and the addition of two-way left-turn lanes or raised medians. For some of the corridor segments identified, potential solutions may be limited due to a lack of available right-of-way. As part of another task of this research project, the Des Moines MPO will develop the means to include access management improvement potential as a factor in its project programming process. The MPO will also incorporate access management considerations into its long-range planning process and educate decision makers about the value of access management.

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The complete research report for this project is available on-line at http://www.ctre.iastate.edu/.

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Long-Term Impacts of Access Management on Business and Land Development along Minnesota Interstate 394

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ABSTRACT

Understanding the relationship between changes in transportation infrastructure and the surrounding commercial economy is important for completing improvement projects successfully. Owners of businesses located along major highway corridors considered for improvements often suggest that changes to the existing street network will reduce property values, reduce retail sales, or cause the business to fail. This is particularly true when direct access between the roadway and commercial land parcels is modified and controlled. Little information exists about the economic impacts of roadway improvements, though studies in Iowa, Texas, and Kansas have indicated that there are few or no adverse economic impacts to most businesses.

Recently, the Minnesota Department of Transportation (Mn/DOT) comprehensively and systematically analyzed the economic impacts associated with converting arterial US Highway 12 to freeway-standard I-394, between Minneapolis and Wayzata in the Twin Cities metro area. The I-394 study first developed an overview covering both the transportation and business conditions in the entire corridor before and after conversion. The second step focused on the details (travel times/distances, land use/values, business turnover) of a representative sample of parcels in the corridor. The selected parcels represent a cross section of corridor business types, including offices, auto dealerships, retail, hospitality, restaurants, and gas stations. Secondary data were gathered and in-depth interviews of business owners were conducted.

The I-394 study found that all transportation performance measures improved when US 12 converted to a limited-access interstate freeway, though traffic volumes almost doubled due to regional and corridor growth. The business performance measures also improved: the amount of vacant land in the corridor has significantly declined, new businesses have been added, business turnover was below statewide and national averages, and employment and adjacent commercial land values are up. Interviews with 14 of the selected business owners/managers indicated that most are doing well and most agreed that the I-394 corridor is a good place to do business, even after much greater access control was put in place. These results are consistent with the findings of the previous research and indicate that the dire predictions of a few of the business owners prior to construction about long-term adverse economic impacts associated with the conversion of US Highway 12 to I-394 did not prove to be true.

Key words: access management—business performance measures—land development impact—traffic performance measures

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Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

PROBLEM STATEMENT

Access management is a process that involves carefully designing and controlling direct access from adjacent land parcels to major highway transportation facilities, such as arterials. A freeway is a fully access-controlled facility in that there are no direct driveway accesses or at-grade public road intersections. All access to adjacent land parcels occurs indirectly via grade-separated interchanges and other roadways, including a comprehensive system of frontage and backage roads.

Literature on the safety and operational benefits of managing access is extensive, with consistently positive results. Managing access leads to significantly lower highway crash rates and is one of the best ways to preserve traffic flow and travel speeds on highways, particularly in urban areas with high levels of traffic volume. One of the most thorough studies of the impact of access management on safety was conducted in Minnesota and showed roughly a 50% reduction in crash rates on well-managed highways.

Although the safety and traffic flow benefits of access management have been well-documented and are well-known, the literature on the impacts of access management projects on adjacent commercial businesses and land parcels is much less abundant. There are very few previous research projects of this sort, although, as in the case of safety studies, they do have similar findings. Access management projects do not seem to cause inordinate damage to either business vitality or commercial land values. A study conducted in Iowa in the 1990s is the most relevant to Minnesota conditions, although the facilities it examined were urban and small city arterials with traffic volumes far lower than those found on interstates in major metropolitan areas.

Because of the scarcity of research, the long-term impact of major access management projects on commercial development and land remains a controversial issue. Business owners and managers and property developers often oppose access management projects due to the perceived impact on business activity and potential future revenues from land development. Legal disputes involving loss of property value and business activity are often the result of such projects. This research is intended to provide a comprehensive long-term evaluation of the transportation, business, and land development impacts resulting from a major access management project in the Twin Cities metropolitan area in Minnesota.

INTERSTATE 394 CORRIDOR BACKGROUND

Interstate 394 is a major east-west freeway facility running between downtown Minneapolis and the western suburbs of the Twin Cities metro area in Minnesota. Prior to the mid-1980s, the highway serving this corridor was a high-speed, at-grade arterial designated as Trunk Highway 12 (TH 12, also called US 12) and locally known as Wayzata Boulevard. This facility had at-grade intersections with major public roadways and a number of slip ramps that provided nearly direct accesses to some adjacent land parcels and commercial businesses. At this time on TH 12, there were short sections built to freeway standards with interchanges.

Between 1985 and 1993, the corridor was extensively reconstructed as a freeway built to urban interstate standards with no at-grade intersections and no direct driveway accesses or slip ramps. See Figure 1.

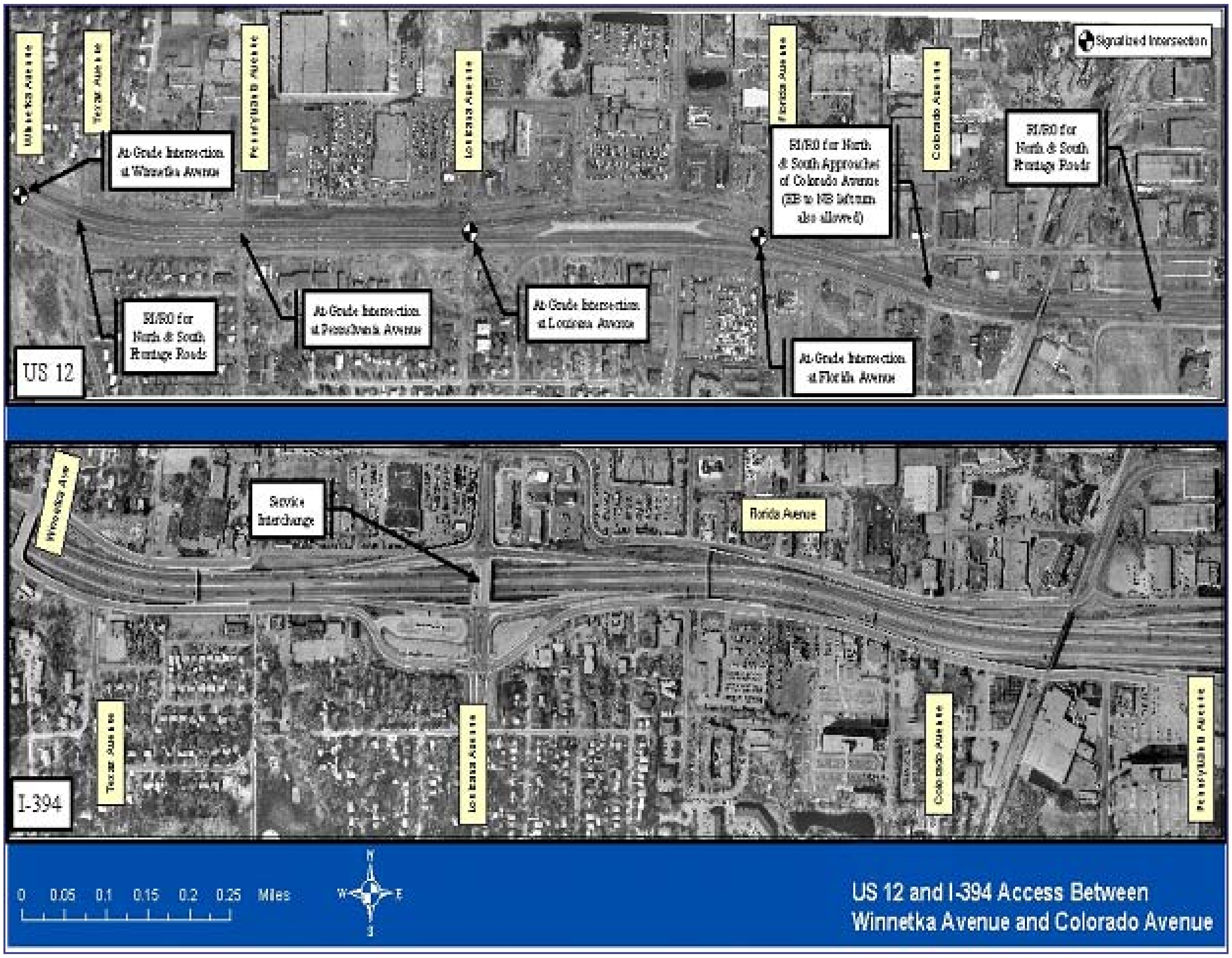

Figure 1. TH 12 (top) and I-394 (bottom) aerial views before and after construction

RESEARCH METHODOLOGY

The research for this study was conducted on two levels. First, overall transportation, land use, demographic, and economic trends along the I-394 corridor were assessed for the time period between 1980 and the early 2000s using a variety of secondary data sources. Major data sources included Mn/DOT, the Metropolitan Council Regional Planning Agency, the Minnesota Department of Revenue, the United States Census Bureau, several private business directories, and the archives of a local commercial real estate brokerage and appraisal firm. All of these data paint of picture of the transportation, land use, and business environments for the corridor as a whole before, during, and after the I-394 reconstruction project.

The second line of research was conducted at the individual land parcel and business level to gain more insight regarding how individual firms fared during the transition from arterial roadway to interstate with its associated higher level of land access control. Collected parcel data included information from condemnation hearing transcripts, property tax assessment databases, and in-depth interviews with selected business owners and managers. The business interviews were selected by the research project advisory committee to represent a cross-section of business types found in the corridor. The interviews concentrated on businesses that have been in the corridor long enough to have experienced both the before and after highway conditions, although some newer businesses were interviewed as well.

KEY FINDINGS

The key research findings for the I-394 corridor study are presented below. The results are presented by topic for the overall corridor trends and by business type for the detailed parcel studies. The findings are consistent with previous research literature about the impacts of access management on business and land development. Overall, the I-394 project was a success in meeting its objectives of adding traffic capacity, preserving traffic flow, and improving traffic safety dramatically. Overall economic trends along the corridor have also remained very positive throughout the study period. Business and land development have continued at a good pace and business turnover rates have been relatively low. Over the past two decades, there has been a noticeable shift in the corridor from residential development to retailing and then to office and service sector development. Commercial land values have also appreciated quickly along the I-394 corridor.

As has been found in previous studies, the experiences of individual businesses have varied. Most businesses interviewed were positive about the results of the I-394 project. Others needed to make a transition or change their business practices as a result of the project. Some businesses studied in detail have failed, although the failures appear to be unrelated to the highway project and changes in access. Other businesses are still in place but have specific complaints about the design of the project, in particular the system of frontage roads along the south side of I-394. These roads confuse business customers, particularly first-time customers.

Key Findings at the Corridor Level

Traffic Volumes

In 1990, TH 12 carried 40,000 to 80,000 vehicles per day. By the year 2000, I-394 carried 109,000 to 145,000 vehicles per day, essentially double the traffic volume (see Figure 2). Handling this anticipated increase in traffic was one of the main motivations for the highway upgrade.

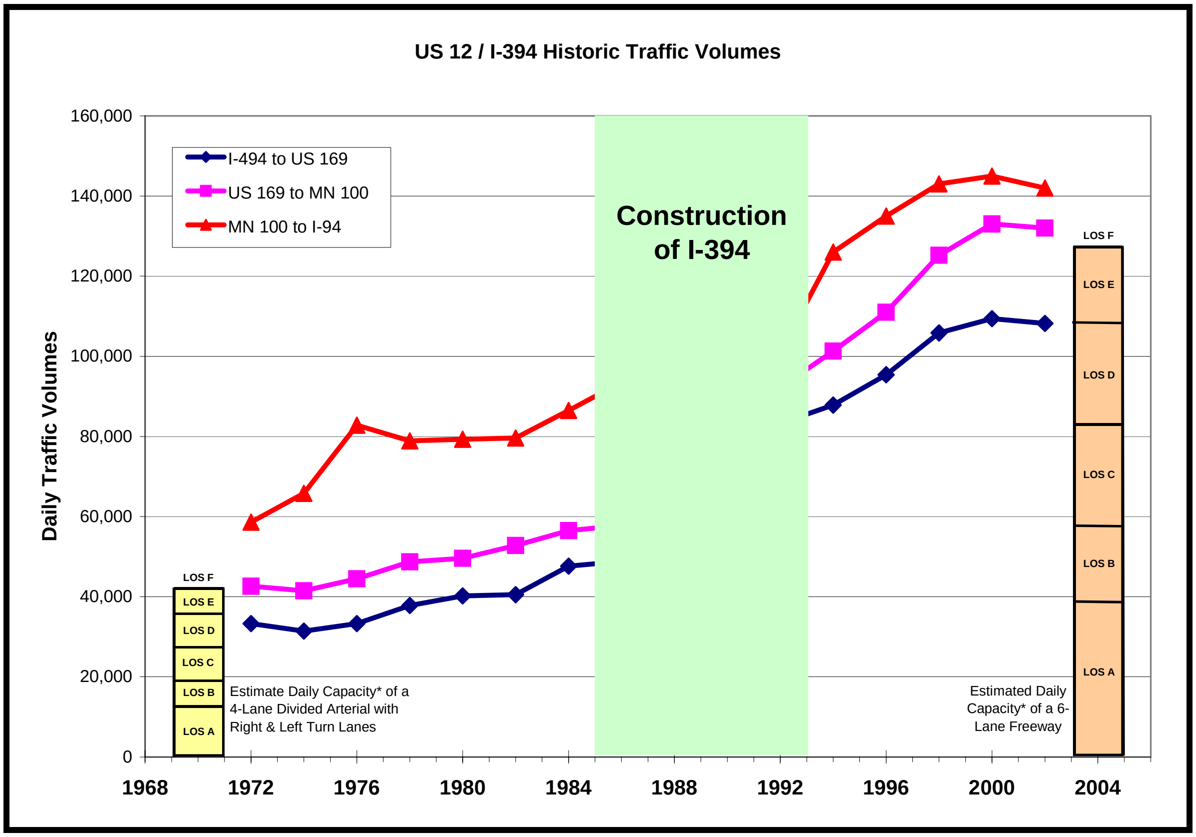

Figure 2. US 12/I-394 historic traffic volumes

Travel Speeds and Traffic Flow

Peak hour travel speeds along I-394 are from 2 to 25 miles per hour (3.2 to 35.4 kilometers per hour) faster along I-394 today than they were along TH 12 before the upgrade, even with the doubling of traffic. Travel times for typical-length trips along the route have generally dropped, even when more indirect access to commercial properties is factored in. The upgrade from an arterial to a freeway clearly maintained the level of service for traffic in the corridor, even under conditions of high traffic growth. However, peak travel speeds on some parts of the current I-394 corridor are now beginning to drop below the Mn/DOT minimum performance level target of 45 miles per hour (72.4 kilometers per hour). This analysis is shown below in Table 1.

Table 1. Total travel time analysis before and after project for 17 sampled locations (1980 vs. 2000)

Indicator	To/from east	To/from west	Typical total travel time change
Mean change	**-6%**	**-5%**	About 1 minute faster
Median change	**-7%**	**1%**	About 1.5 minutes faster to 10 seconds slower
Improved	**12**	**8**	
Worse	**4**	**9**	
No change	**1**	**0**	
Large positive change	**6**	**3**	Over 2 minutes faster
Large negative change	**2**	**4**	Over 2 minutes slower
Small changes (LT 10%)	**9**	**10**	Within 2 minutes faster or slower

Note: Typical total trip lengths in the I-394 corridor are between 15 to 20 minutes

Traffic Safety

The I-394 corridor has significantly fewer fatal and injury crashes than TH 12 had, even though traffic volumes have doubled. Average annual fatal crashes have declined from two to one. The rate of fatal and injury crashes (crashes normalized by traffic volume) has declined considerably. Clearly, the I-394 project was very beneficial in terms of traffic safety. See Figure 3.

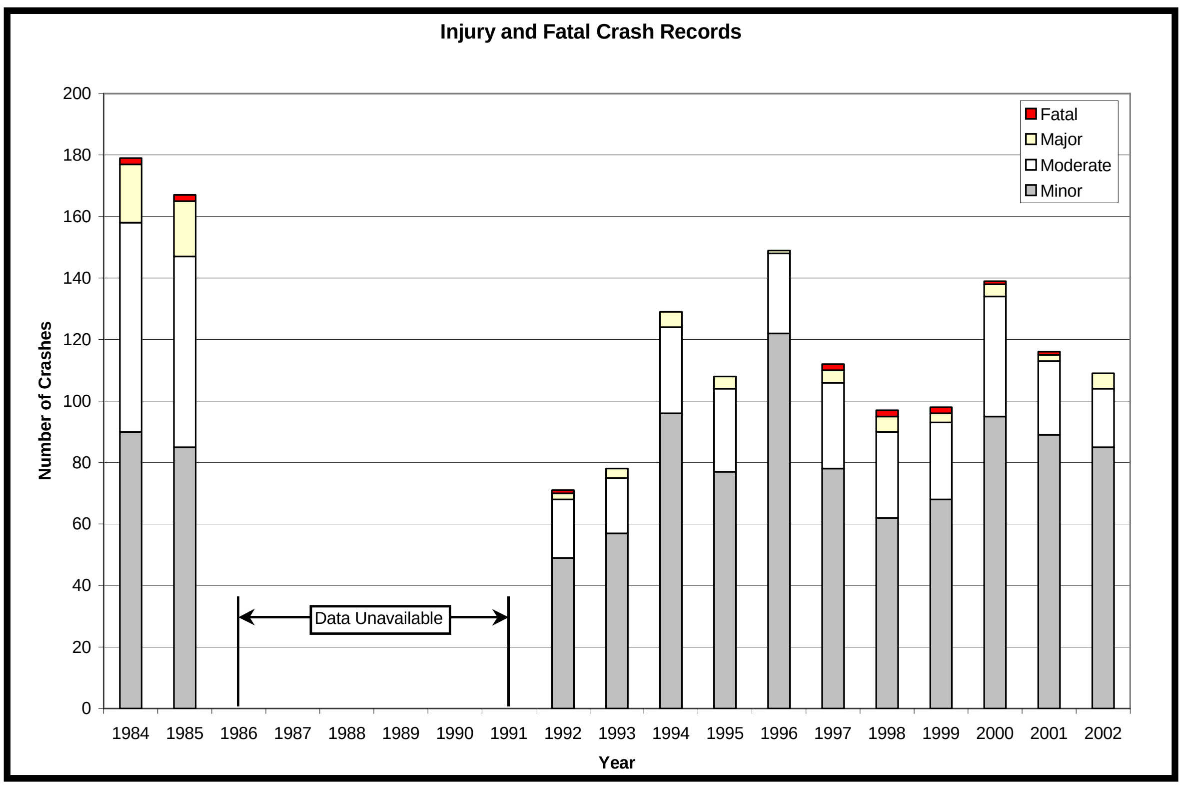

Figure 3. US 12/I-394 traffic safety trend

Land Use

There are roughly 1,300 acres (525 hectares) of developable land immediately adjacent to the I-394 corridor. Land use was compared for this quarter-mile (400 meter) buffer zone for the period between 1984 and 2000 using remote sensing data obtained from the metropolitan council. The results show that land use along the corridor has become more intensive, with significant decreases in residential and agricultural/vacant land and significant increases in commercial and industrial land. Commercial land uses now make up 40% of the land adjacent to the corridor. Land use change was most pronounced in the middle of the corridor near the interchanges with US 169 and TH 100. These are locations with high levels of accessibility, traffic, and visibility. This land is now being used far more intensively than before I-394 was constructed.

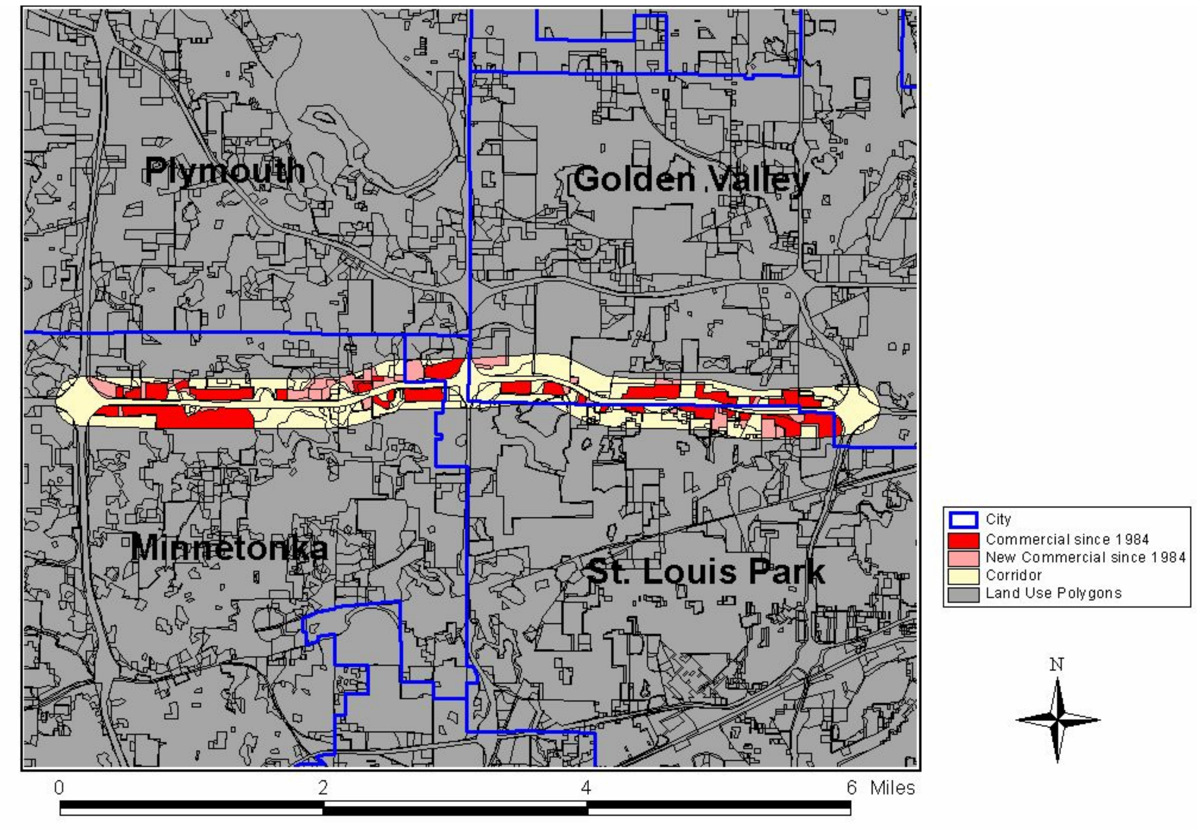

Figure 4. Commercial land use change along the study corridor, 1984–2000

Population and Income

As the I-394 corridor has transitioned from residential land use to commercial/industrial land use, the population of the census tracts along the corridor have declined somewhat. However, the population that remains has become more affluent as measured by statistics such as median household income. The area has a relatively high median household income by Minnesota standards, which makes the area an attractive market for both retailers and service businesses.

Retail Trade Activity

The number of retail firms located in the cities that surround the I-394 corridor has fluctuated over time; this sort of ebb and flow is commonplace in retailing and is mostly related to overall economic conditions. The three suburban cities that include the I-394 corridor have become somewhat less dependent on retail businesses for their commercial base over the past few decades. There has been a rise in service businesses, including services for households and other businesses. Gross retail sales in the corridor (which includes taxable services) grew substantially in the area, suggesting an overall healthy business climate, but one that is becoming more services-oriented.

Employment

Employment in the area immediately surrounding the I-394 corridor grew by almost 30% between 1990 and 2000. Unfortunately, comparable data were not available for 1980. The density of employment (employees per acre of land) also grew, reflecting the growing intensity of land use along the corridor. The composition of employment along I-394 changed dramatically as direct employment in retailing declined while employment in service and office sectors grew. These changes were most pronounced in the middle of the corridor, the same area where the most significant changes in land use occurred. See Figure 5.

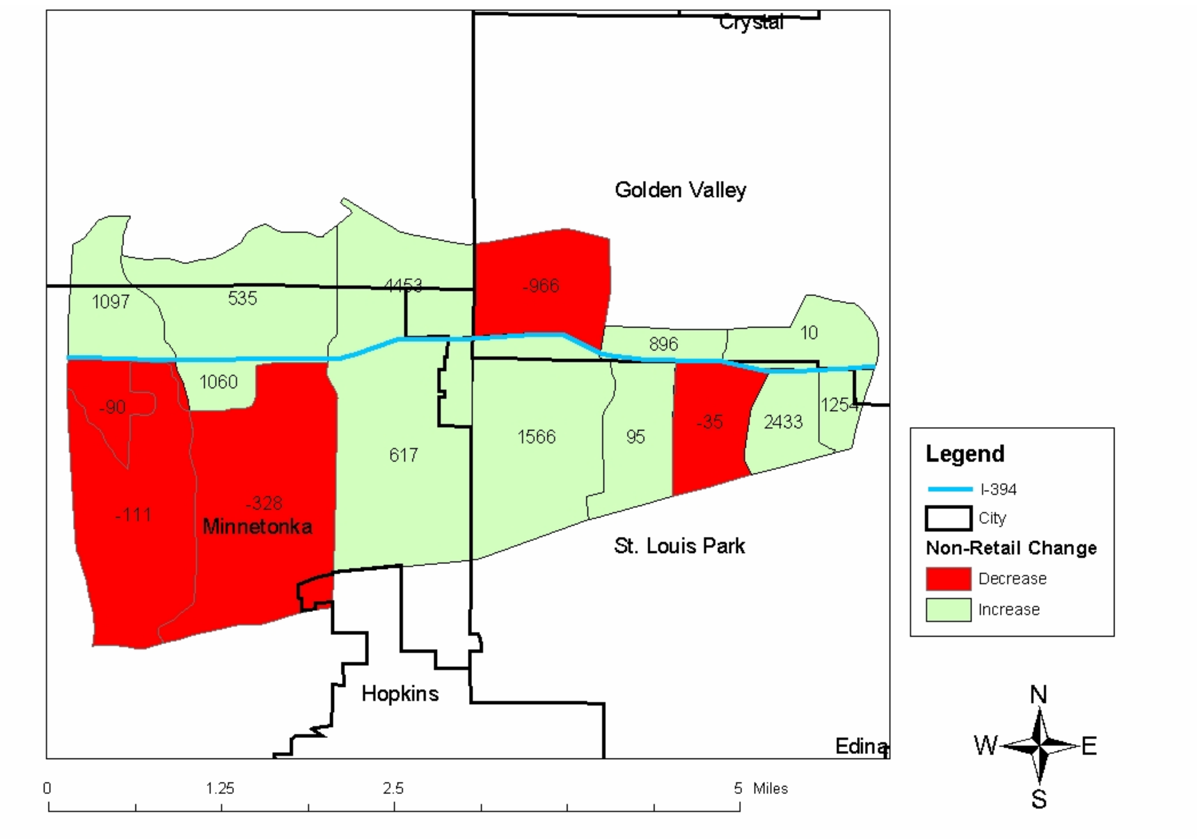

Figure 5. Change in non-retail employment on I-394 by traffic analysis zone, 1990–2000

Business Turnover

An analysis of business turnover was conducted using published business directories for the period between 1980 and 2003 for addresses along Wayzata Boulevard. (Wayzata Boulevard is the local street name for TH 12 and the frontage road system that now serves businesses along I-394.) For this analysis, a business was considered to have turned over if it went out of business, moved out of the study corridor, or changed its name such that it could not be positively identified as the same business. The analysis indicated that there has been substantial new development; there are now many more commercial postal addresses, while vacant addresses have declined dramatically. The 2003 commercial vacancy rate along the corridor was very low. The most significant change that has occurred in business activity over time is a large increase in multi-tenant buildings, including strip malls and office buildings. These properties are now often leased by service sector businesses. The overall rate of business turnover for the corridor has been lower than typical annual rates for Minnesota and the nation as a whole. The highest turnover rates have been for service and office businesses rather than retail businesses, which include all types of restaurants.

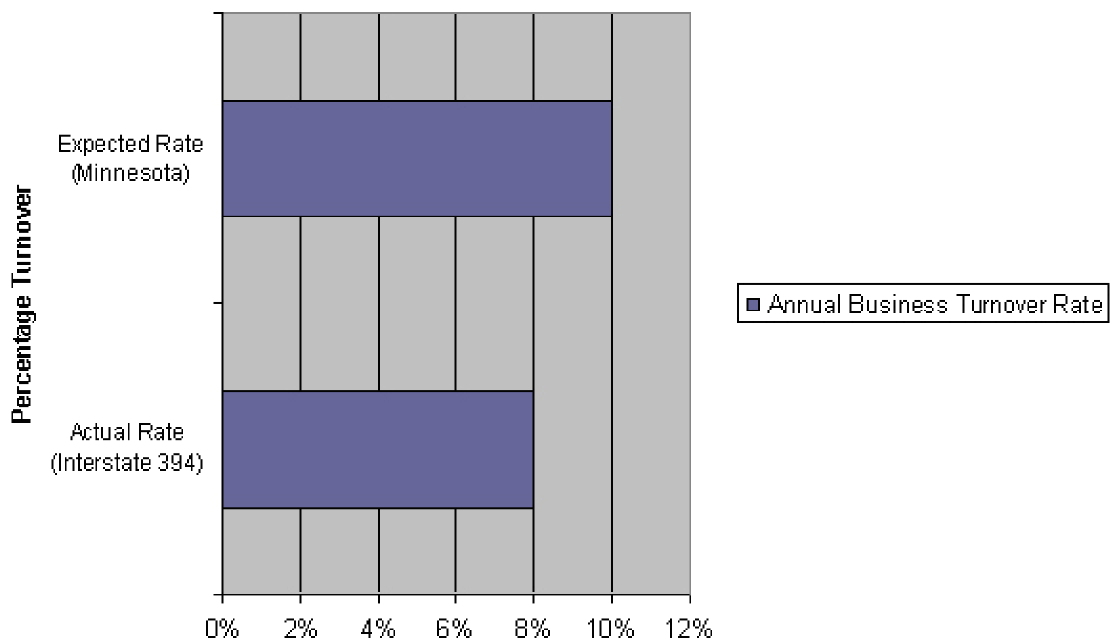

Figure 6. Annual business turnover rate, I-394 study corridor vs. all of Minnesota

Commercial Land Values

Raw commercial land values in the I-394 corridor were assessed over a three-decade period (roughly 1970 through 2003) using sales transaction records from a local commercial realty and appraisal company. Land values along I-394 have grown substantially, from about $2.00 per square foot (0.09 square meters) in 1970 to about $15.00 per square foot today. The price trend for the I-394 corridor has been very similar to that for another highly developed commercial corridor in the Twin Cities metropolitan area for which comparable data are available, I-494 through Bloomington near the airport and the Mall of America, which is the largest shopping mall in North America.

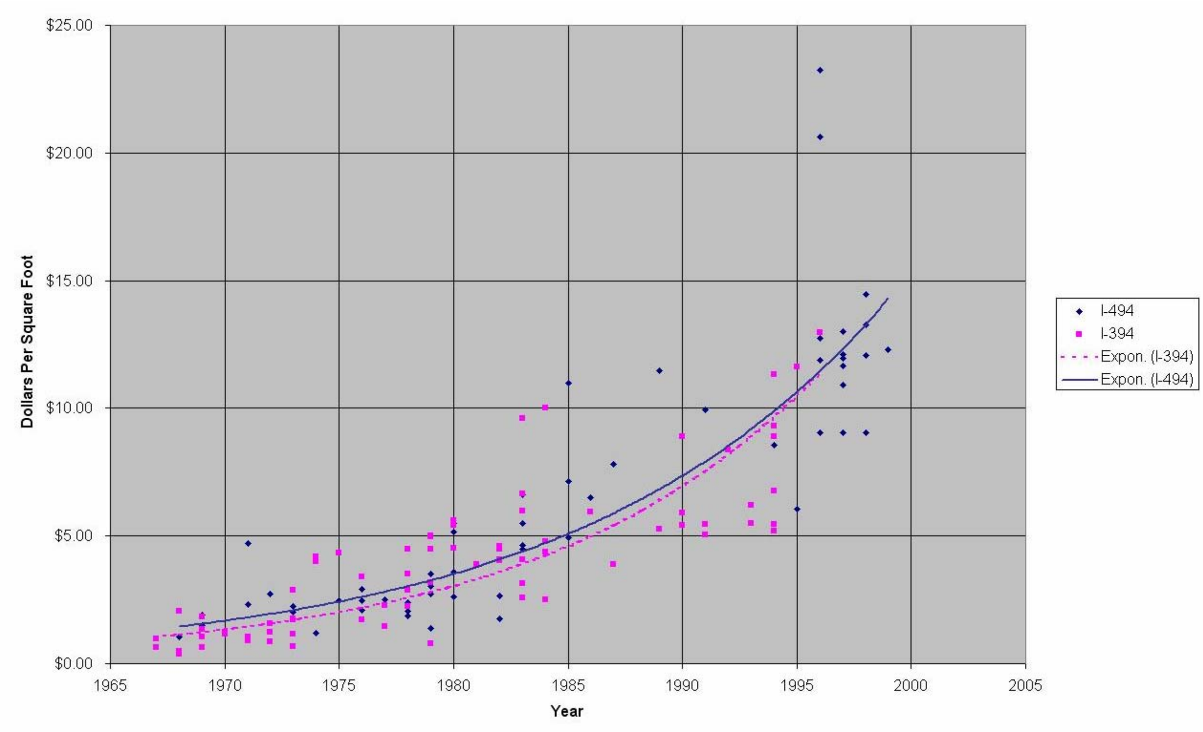

Figure 7. I-394 raw commercial land value trend vs. I-494 corridor

Key Findings at the Business and Parcel Level by Business Type

Office Buildings

Four office developments were studied in detail for this research. Several of the locations studied were in place at the time of reconstruction and noted difficulties during the time the project was underway. Several were involved in condemnation proceedings for the project. Since the project has been completed, the results from travel time studies, assessor data analysis, and interviews with building owners and managers indicate that this business type has fared extremely well along I-394. Travel times to these buildings have mainly declined, property values have risen, and managers were very positive about the I-394 corridor as a place to do business. In many ways, the I-394 corridor has become an ideal location for offices, in that it is close to the Minneapolis central business district, but without its problems of congestion, high costs, and limited parking. The attractiveness of the corridor for offices and office employment has helped other business categories, especially fast food restaurants. Office land uses appear to be much less dependent on access and visibility factors than many other uses and are more dependent on the overall economic characteristics of the corridor and its overall location.

Automobile Dealerships

Two auto dealerships were studied in detail. Many of the parcels owned by these dealers were involved in condemnation proceedings during the I-394 construction project and the dealers that could be interviewed were adamant that the project would be damaging to them while it was in progress and afterward due to direct access restrictions and losses in visibility. Although auto purchases would not appear to be impulse purchases due to their expensive nature, dealers maintain that a significant percentage of these purchases are made quickly and depend greatly on visibility from the highway. One dealer went so far as to state that an auto dealership would no longer be viable at its current location due to changes in access and visibility. However, both dealerships currently remain at the same location along I-394. Also, travel times have declined or remained fairly stable at these locations. Property values continue to increase and the I-394 location is still a good location for dealerships. One of the dealers is now somewhat positive about the highway project. A key to keeping these types of businesses healthy appears to be their ability to transition during the construction project when customer access is complicated and visibility is hindered.

Sit-Down Restaurants

Five sit-down restaurants were studied in depth. Two of these were in business along old TH 12. One went out of business for reasons that appear to be unrelated to the highway project. The other two are new to the corridor. Like many of the older businesses along TH 12, the two oldest sit-down restaurants studied were very opposed to the I-394 upgrading project at the time due to the perceived impact it could have on their business. Both were involved in condemnation proceedings. Both argued that their restaurants would no longer be viable once the I-394 project was completed and direct accesses were no longer available. Today, both properties are prosperous restaurants in the same chain. Both parcels have travel-time access similar to their condition prior to the upgrade. Both of the current managers are fairly positive about I-394 as a location today, although one has made adjustments in the type of customer the restaurant caters to (now local rather than drive-by customers) and the other has significant concerns about the confusing system of frontage roads that customers need to navigate to reach these restaurants.

The sit-down restaurant studied along the corridor that went out of business (when the entire chain of restaurants failed) was replaced on the same site by two new sit-down restaurants. Managers of these two restaurants were positive about the I-394 location in general, but were also frustrated by the complex frontage road system on the south side of I-394. They noted that access can be circuitous and that the frontage roads are far enough away from the freeway to limit their visibility. Clearly, sit-down restaurants depend both on efficient and straightforward access and the overall economic health of the corridor.

Fast Food Restaurants

Two fast food restaurants were studied in detail. Both existed before completion of the I-394 upgrade. Similar to office buildings, fast food restaurants were the business type that experienced the most positive results. Both restaurants have had excellent sales activities, partly as a result of the great increase in employment and traffic along the corridor since 1980. Both businesses were in locations such that travel times have improved or stayed stable and where visibility remained very good. As with office buildings, fast food restaurants find the I-394 corridor a nearly ideal location. Although efficient access and visibility are very important to fast food restaurants, what really seems to matter for them is a strong customer base, which has grown in the many new offices along I-394 since reconstruction.

Strip Commercial (Small Shopping) Centers

The detailed information gathered regarding this business category through interviews and other means was very limited and does not indicate either negative or positive impacts of the I-394 project. However, the I-394 corridor does appear to be a hospitable environment for such businesses, since many new strip shopping centers have been developed along the corridor.

General Retail

Two specialty retailers were studied in this category. Both were in place prior to the construction project. One was involved in condemnation proceedings. Both remain highly successful businesses today, although one changed its marketing strategy to make the store more of a “destination” business (one less dependent on drive-by customers). Both businesses are clearly taking advantage of the prosperous customer base found along the corridor today. This is the most important factor for businesses in the general retail category.

Big-Box Retail

Two parcels were studied in detail. These large, specialty retailers are clearly “destination” businesses (meaning that customers primarily seek them out rather than impulsively decide to patronize them). One of them has seen small increases in travel time for typical trips after project completion. One owner declined to be interviewed, while the other noted that peak period congestion on I-394 is a much greater concern for his customers than is additional access distance. This store noted it benefits overall from the I-394 corridor location, its prosperous customer base, and the visibility the freeway creates.

Hospitality

Two hotels were studied in detail. One went out of business for reasons unrelated to the highway project. (Its convention-based business went elsewhere when new convention facilities were constructed away from the study corridor.) It was replaced by several restaurants. The other is still in business at the original location, but the management chose not to participate in the interview process. As with the strip commercial properties, it is difficult to draw any conclusions from studying these two businesses.

Convenience Stores and Gas Stations

One such parcel was studied in detail. Travel time to and from this location was largely unchanged by the I-394 project and the gas station remains in business today.

CONCLUSIONS

The results of the I-394 research indicate that the economic impacts of upgrading the highway from an arterial facility to a freeway were largely positive. Clearly, additional traffic capacity, improved travel speeds, and improved traffic safety resulted. The overall economy of the corridor improved, as measured by such indicators as employment, income, business turnover, and retail sales taxes. A considerable amount of land development occurred and developed land transitioned from lower uses, such as agriculture and residential, to higher uses, such as commercial/office. Commercial land values increased significantly and the trend was in line with a comparable high traffic volume corridor, I-494.

As has been shown in the literature, even though the overall business turnover rate in the corridor has been relatively low, the experience of individual businesses and land parcels does vary when projects such as I-394 are constructed and direct access to land is controlled more tightly. Some individual businesses do not fare as well as others. However, the impacts of the I-394 project that business persons expected have proven to be much greater than the actual impacts (and often in the opposite direction). Statements made in condemnation proceedings, that locations would no longer be viable for business, turned out to be wrong in all cases. There are two reasons for this. First, the overall economic environment of the corridor improved greatly after the project was put in place. Second, travel times for typical trips to and from parcels along the corridor generally declined because travel speeds along the mainline of I-394 increased.

Certain types of businesses studied along I-394 appear to have greatly benefited from the highway project and the resultant increase in traffic along the corridor. These include office buildings, fast food restaurants, and big-box retailers. For these businesses, the macro-level economic trends following the project seem to have been very important. These businesses have thrived because the corridor is a healthy environment in general, with abundant customers and buying power. For other business types, the project was more of a mixed blessing, although on the whole the positives have outweighed the negatives. Table 2 summarizes the research results by indicator and by business type.

Table 2. Summary of impact by indicator

Indicator	Direction of impact	Comments
Transportation		
Traffic volume	Positive	Traffic doubled
Travel speed	Positive	Peak travel speeds up
Traffic safety	Very positive	Large decline in serious crash rate
Economic and demographic		
Land use	Positive	Land developed more intensively
Population	Neutral to negative	Slight population loss due to land use changes away from residential
Income	Neutral to positive	Area consumers more affluent
Retail trade activity	Neutral	Mixed trends
Employment	Positive	Large office jobs gain
Business turnover	Neutral to positive	Below state turnover rate
Commercial land values	Neutral	Trend similar to I-494 corridor
Business type		
Office buildings	Very positive	Large increase in activity
Automobile dealerships	Neutral	Remained viable after transition
Sit-down restaurants	Neutral	Remained viable with adjustments
Fast food restaurants	Very positive	Large increase in business
Strip commercial centers	Neutral to positive	Attractive location
General retail	Neutral	Remained viable with adjustments
Big box retail	Very positive	Very attractive customer base
Hospitality	Neutral	Insufficient data
Convenience stores and gas stations	Neutral	Remained viable

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Structural Dowel Bar Alternatives and Gaps of Knowledge

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ABSTRACT

Dowel bars are subject to large cycles of fatigue loading as the bars transmit loads from one portion to another of a concrete pavement, bridge, or other structural components. As the continued cycling occurs, the bearing of the contact from the bar on the concrete can cause an “oblonging” of the hole surrounding the bar for a typical circular bar. Thus, a need exists to reduce the bearing stresses between the dowel bar and the concrete.

The combination of the corrosion and bearing fatigue problems for dowel bars leads to the need to consider alternative shapes and materials for dowel bars. Research has been on-going at Iowa State University (ISU) on both the alternative shapes and the alternative materials for dowel bars. Several types of structural tests and analyses have been conducted at ISU. Recently, an extensive study was made of the gaps of knowledge that exists for design and research for all types of dowel bars and the associated parameters.

The structural laboratory work at ISU has focused on many different potential dowel bars of various shapes and materials, including elliptically-shaped fiber reinforced polymer (FRP) and steel dowels. The goal of the elliptically-shaped dowel bars was to reduce the maximum contact bearing stress between the bar and the concrete.

This paper will present some of the key results of earlier laboratory tests that have been conducted over a period of approximately 15 years, including a brief summary of the theory, as background. Key sections of the paper include knowledge gaps and needed research for FRP in concrete.

Key words: concrete pavement—corrosion—dowels—dowel bars—fiber reinforced polymer material

INTRODUCTION

A vast majority of the nation's highways and roads are made of jointed concrete pavement. These joints allow for deformation and movement due to thermal and environmental conditions. Joints may either be longitudinal joints, parallel with traffic, or transverse joints, perpendicular to traffic. Transverse joints are placed at regular intervals creating discontinuities in the pavement and forming a series of slabs. Load transfer within a series of concrete slabs takes place across these joints. An effective load transfer device, therefore, must be present in order to transfer load between adjacent slabs.

For a typical concrete paved road, these joints are assumed to be approximately 1/8 in. gaps between two adjacent slabs. Dowel bars are located at these joints and used to transfer load from one slab to an adjacent slab. After a significant number of vehicles have passed over the joint, an oblonging where the dowel bar contacts the concrete can occur. This oblonging creates a void space. This void space is formed due to a stress concentration where the dowel contacts the concrete at the joint face directly above and below the dowel. Over time, the repeated process of traffic traveling over the joint crushes the concrete surrounding the dowel bar and causes a void in the concrete. This void inhibits the dowels ability to effectively transfer load across the joint.

Possible corrosion of the dowel bar can potentially bind or lock the joint. When locking of the joint occurs, no thermal expansion is allowed and new cracks parallel to the joint are formed directly behind the dowel bars in the concrete. As temperature decreases, contraction of the concrete will occur resulting in the new cracks becoming wider and a resulting load transfer failure. Once there is no longer load transferred across the joint, the entire load is then transferred to the subgrade and differential settlement of the adjacent slabs occurs. Differential settlement of the slabs creates a vertical discontinuity at the joints, making vehicle travel uncomfortable, and requires that the slab be repaired or replaced.

A majority of the dowel bars used today for load transfer are epoxy coated. This epoxy coating aids in the reduction of the exposure to corrosive agents. However, many times this coating is nicked or scraped before installation leaving the uncoated steel susceptible to deterioration.

As was mentioned previously, a void around a dowel bar is formed by stress concentrations crushing the concrete directly in contact with the dowel. When a wheel load is applied to the concrete slab, the force is supported only by the top or bottom of the dowel bar, not the sides. Since the stress concentration region lays on the top or bottom of the dowel bar, the smaller the dowel, the higher the stress concentration. The sides of the dowel bar do not aid in the distribution of the wheel load from the concrete. Therefore, the top and bottom of the dowel bar at the face of the joint is where the stress concentration is located and is directly related to the width and/or shape of the dowel bar. While round dowel bars handle these stress concentrations relatively well, other shapes and materials may provide a better distribution.

Iowa State University researchers have been actively performing continuous research in the area of dowel bars for pavement slabs since 1991. Interest in this work was generated by the utilization of alternative dowel bar shapes and materials. A significant amount of research was funded by the Iowa Department of Transportation (IDOT) in two fairly significant projects, resulting in several research reports, the most notable of which are Report #HR343 "Non-Corrosive Tie Reinforcing and Dowel Bars For Highway Pavement Slabs" (Porter et al. 1993) and #TR408 "Investigation of Glass Fiber Composite Dowel Bars For Highway Pavement Slabs" (Porter et al. 2001). These reports serve as examples of the work done by Iowa State University.

Additional work has been done at ISU on a compilation of preliminary needs for dowel bars for highway pavement slab joints. A number of other reports have also been prepared for the Iowa Department of Transportation, American Highway Technology (AHT), Highway Innovative Technology Evaluation

Center (HITEC), and others concerning dowel bar performance. In combining past and present knowledge, gaps found within dowel bar research can be closed and a universal test may be developed in order to properly evaluate dowel bars.

During the time that ISU has been conducting the IDOT-sponsored work, other states have also begun to conduct additional studies on both laboratory specimens and field applications of alternative dowel bars. The various studies, however, have not been coordinated among state or federal agencies. Therefore, over the recent years, apparent gaps in knowledge exist as to what is yet needed and as to what areas of research may have been duplicated. The purpose of this paper is to summarize the identified gaps in the knowledge of dowel bars, which was part of an extensive investigation funded by the IDOT via report # HR-1080, “Assessment of Highway Pavement Slab Dowel Bar Research” (Porter and Guinn 2003). In the pages to follow the identified “gaps” of knowledge will be discussed. A “gap” in dowel bar knowledge is any piece of information that is not already known which may pertain to the effectiveness of the dowel bar as a load transfer device.

BACKGROUND

In order to determine the technology gaps in dowel bar research, a collection of previous reports, studies, and interviews were obtained so that each may be reviewed. From the review of this information, the technology gaps and duplications in dowel bar knowledge were determined. Details of this study are shown in the report by Porter and Guinn “Assessment of Highway Pavement Slab Dowel Bar Research” (2002). References (Porter et al. 1993; Porter et al. 2001) provide the results of full-scale tests and analysis. An example of the full-scale test is shown in Figure 1.

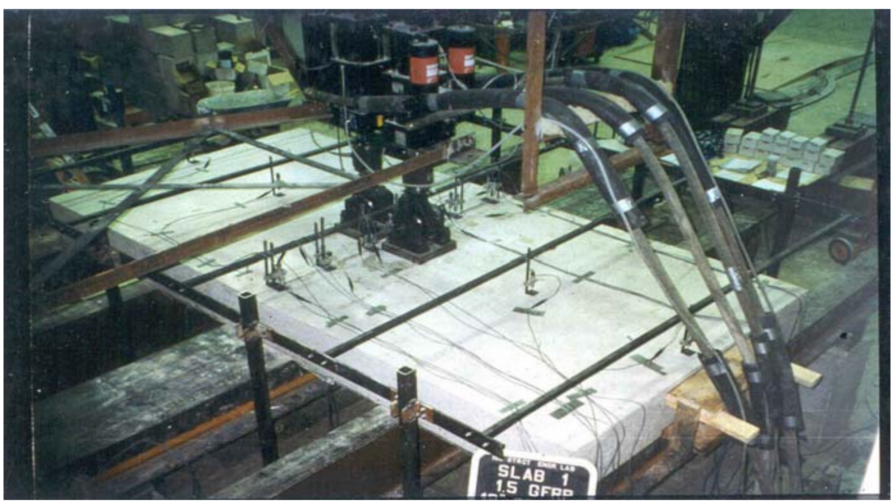

Figure 1. Full-scale test of pavement slab containing dowel bars

Some of the highlights from the IDOT Reports are as follows:

- The results indicated that the elliptical dowel bars behaved as predicted. When comparing the 1-1/2 in. round epoxy coated steel dowel bars to the large elliptical steel dowel bars, the large elliptical steel dowel bars produce bearing stresses on the concrete that are greatly reduced while the increase in relative deflection is minimal.
- The large elliptical steel dowel bars have an increase in cross-sectional area of nearly 18% but provide a reduction in bearing stress of over 26%. In contrast, the 1-1/2 in. round epoxy coated steel dowel bars have a 44% increase in cross-sectional area over the smaller 1-1/4 in. round epoxy coated steel dowel bars, yet only provide a 25% reduction in bearing stress.

- The round dowel bars did retain a slight advantage in the stiffness over elliptical dowel bars of a similar cross-sectional area due to their shape. However, this difference in stiffness is insignificant based on the small variance in the deflection of the slabs. The difference in magnitude of the deflections is so small that the dowel bars could be considered as having roughly the same deflection.
- This research has shown that the 1.5 in. round epoxy coated steel dowel bars have roughly same bearing stress as the medium elliptical dowel steel bars. This occurrence could be beneficial if the load transfer efficiency was determined.
- Dowel bar spacing is a method to distribute load to the dowel bars. The smaller the spacing of the dowel bars, the smaller the load on the dowel bars. A decrease in pavement thickness will lower the number of bars available for load transfer and a smaller spacing may be required.
- The 1.5 in. diameter GFRP dowels spaced at 12 in. on center were inadequate in transferring load.
- The 1.5 in. diameter GFRP dowels spaced at 6 in. on center were effective in transferring load over the design life of the pavement.
- The current design guideline for steel dowels cannot be applied to GFRP dowels.
- The 1.75 in. FC dowels spaced at 8 in. performed at least as well as 1.5 in. steel dowels at 12 in. for transferring static loads across the joint in the full-scale pavement test specimens. The performance of the 1.75 in. FC dowels spaced at 12 in. was similar to that of the 1.5 in. steel dowels spaced at 12 in. with any difference being attributed to dowel diameter.
- The load transfer efficiency of 1.75 in. FC dowels spaced at 8 in. for a full-scale pavement slab was nearly constant (approximately 44.5% load transfer) through two million applied load cycles with a maximum of 9,000 pounds.
- The load transfer efficiency of 1.5 in. steel dowels spaced at 12 in. for a full-scale pavement slab decreased (approximately from 43.5% to 41.0% load transfer) over the first two million cycles.
- The load transfer efficiency of 1.75 in. FC dowels spaced at 12 in. for a full-scale pavement slab decreased from an initial value of approximately 44% to a final value of approximately 41% after 10 million cycles.

The structural laboratory work at ISU has focused on many different potential dowel bars of various shapes and materials. The different types of dowel bars investigated include the following:

- 1.5 in. diameter standard epoxy coated steel
- 1.5 in. diameter stainless steel
- 1.5 in. diameter FRP
- 1.875 in. diameter FRP
- 1.5 in. diameter aluminum
- 1.957 in. diameter aluminum
- 1.714 in. diameter copper
- 1.5 in. diameter copper
- 1.5 in. diameter stainless steel
- 1.5 in. diameter hollow-filled steel
- 1.75 in. diameter FRP
- aged FRP
- special-sized shaved FRP
- 1.5 in. diameter plain steel
- several sizes of elliptically-shaped steel
- several sizes of elliptically-shaped FRP

The goal of the elliptically-shaped dowel bars was to reduce the maximum contact bearing stress between the bar and the concrete. The FRP acronym is for fiber reinforced polymer material and these bars consisted of glass fibers and various types of polymers.

The types of structural laboratory tests conducted include the following:

- full-scale pavement sections subjected to fatigue loading
- Iosipescu elemental shear (static)
- AASHTO T-253 elemental shear (static and fatigue)
- pull-out
- alkalinity aging
- chemical properties

All of the above-mentioned tests were conducted in the Structural Engineering Laboratory at ISU. The elliptical shape has been used for steel and FRP dowels for laboratory testing.

ANALYSIS SUMMARY

On-going work at ISU is currently investigating the results of field applications and additional laboratory analysis. The laboratory analysis includes an investigation into the type of test element that can be used to determine property values needed in the analysis. Figure 2 shows the two test types under investigation to determine a needed parameter, k_o, for the analysis.

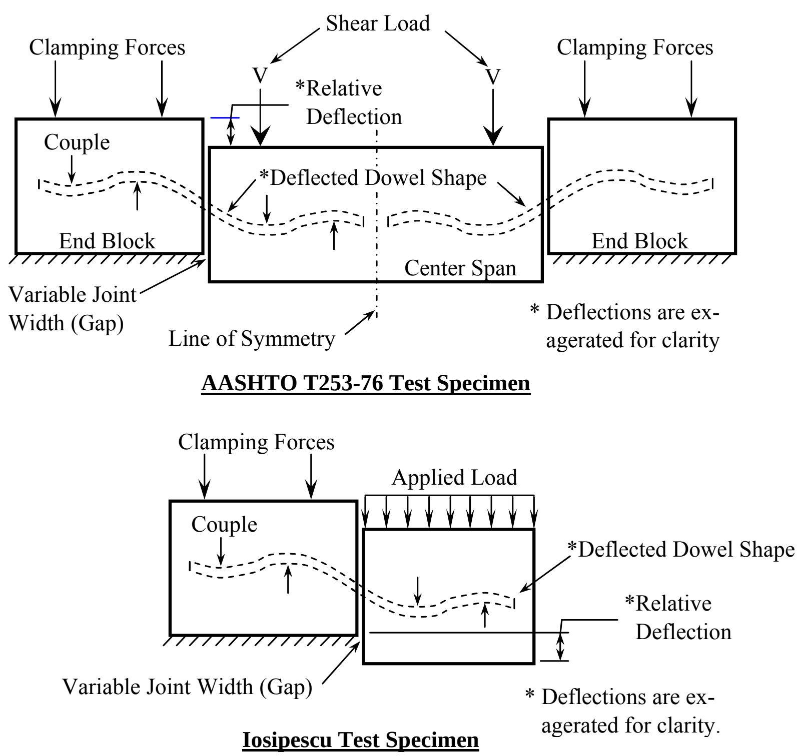

Figure 2. Testing schematics

The analysis has focused on determination of the deflection and the related bearing stress properties. The deflected shape and the key parameters are shown in Figure 3.

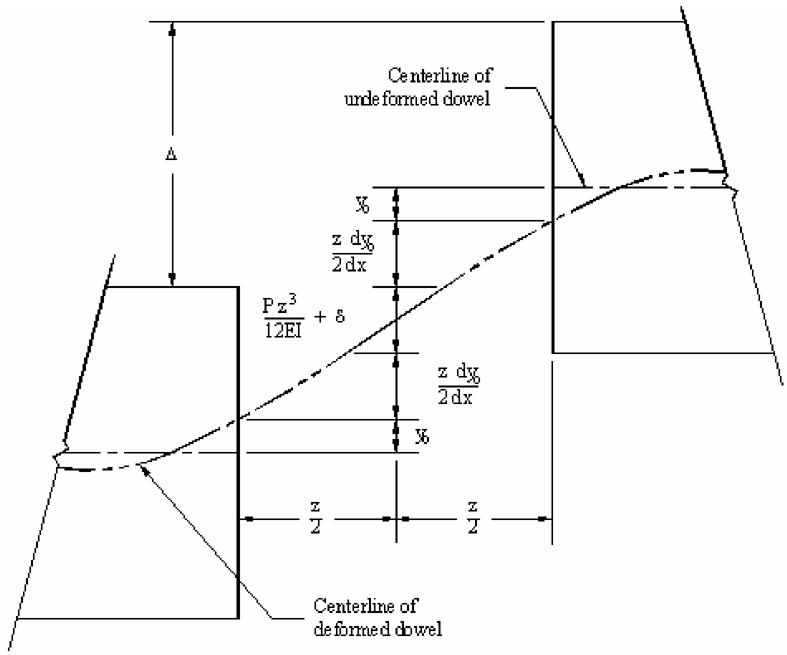

Figure 3. Deflection relationships across a joint width z

The deflection relationships shown in Figure 3 result in a predictive deflection shown by Equation 1.

$$\Delta_{relative} = 2 \cdot (y_o) + 2 \cdot \left(\frac{z}{2}\right) \cdot \left(\frac{dy_o}{dx}\right) + \frac{\lambda V \cdot z}{A_d \cdot G} + \frac{V \cdot z^3}{12 \cdot E \cdot I_z} \qquad (1)$$

y_o = dowel deflection relative to or within concrete at the face (in.)
z = gap width (in.)
V (or P in Figure 3) = shear transferred across the joint, including the
center span weight in Figure 2 (lb.)
λ = form (or shear shape) factor
δ = shear deflection as shown in Figure 3 (in.)
A_d = dowel cross sectional area (in.^{2})
G = shear modulus (psi)
E = flexural modulus of elasticity (psi)
I_z = moment of inertia (in.^{4})

The AASHTO test shown in Figure 2 is being investigated to determine its ability to predict k_o. Previous work at ISU has utilized Friberg's semi-infinite beam theory (Friberg 1940), where

$$y_o = \frac{V}{4\cdot\beta^3\cdot EI_z}\cdot(2+\beta\cdot z)$$

$$\beta = \sqrt[4]{\frac{k_o\cdot d}{4\cdot EI_z}}$$ = relative stiffness of the dowel bar encased in concrete (in.$^{-1}$)

y_o = deflection at the face of the joint (in.)
k_o = modulus of dowel support or reaction (pci)
d = dowel width (in.)
V = shear load transferred through the dowel (lb.)
z = joint width (in.)

The relative deflection across the joint in the AASHTO test is given by Equation 2. This equation differs from Equation 1 by the flexural deflection (i.e., no inflection point at the center of the gap) and y_o terms. For large joint widths, these changes can be significant when solving for k_o. Slight differences in dowel slope at each face were determined theoretically for small joint widths. These differences are extremely small and were neglected in Equation 2.

$$\Delta_{relative} = (y1_o + y2_o) + 2\cdot\left(\frac{z}{2}\right)\cdot\left(\frac{dy_o}{dx}\right) + \frac{\lambda V\cdot z}{A_d\cdot G} + \left(\frac{V\cdot z^3}{3\cdot E\cdot I} - \frac{M_2\cdot z^2}{2\cdot E\cdot I}\right) \quad (2)$$

All other terms are defined above with exception of

$y1_o$, $y2_o$ = dowel deflection relative to the concrete for the end block and center span, respectively (in.)
M_2 = moment in the dowel at the center-span face or load-side face (in.-lb.)

The bearing stress at the face of the joint is calculated using the following formula (assuming that the stress distribution across the dowel width is uniform):

$$\sigma_b = k_o\cdot y_o$$

Current ISU work is investigating these analysis equations for FRP, steel, and other materials of several shapes, including the new elliptical shape. The elliptical shape is receiving significant attention due to the better bearing stress distribution at the concrete-to-dowel-bar interface.

GAPS IN KNOWLEDGE

The "gaps" in knowledge that have been determined by the recently completed IDOT work (Porter and Guinn 2002) will be presented in this section of the paper. These gaps will utilize the information presented in the Background and Analysis Sections above. These gaps are the result of an extensive background investigation of work around the world. The compilation of the total work done versus what information is needed or questions that have been raised has led to the listing of the significant gaps of knowledge. These gaps have been grouped into seven major categories:

1. Bearing stress
2. Corrosion/aging/environment
3. Theory with respect to load transfer
4. Test procedures
5. Design procedures
6. Parameter changes
7. Other items

The earmarked items designated for each category are shown below.

Bearing stress

- Acceptable corrosion of steel dowel bars
- Investigation of wheel load at pavement edge
- Investigation of uneven dowel bar placement
- Curvature of slab effects on dowel when load placed in middle of slab
- Bearing and contact surface stresses for shapes other than circular
- Investigation of load transfer efficiency of different shaped dowels at different spacing
- The relationship between the modulus of foundation versus bearing stress for different dowel bar shapes
- Investigation into the effects of oblonging of the hole for a large number of cycles
- Fatigue for a large number of cycles correlated with field cyclic results

Corrosion/aging/environment

- Effects of moisture on FRP dowels
- Aging of FRP dowels
- Effects of road chemicals on FRP resin
- Acceptable corrosion of steel dowel bars
- Fatigue for a large number of cycles correlated with field cyclic results

Theory with respect to load transfer

- Investigation of wheel load at pavement edge
- Bearing and contact surface stresses for shapes other than circular
- Modulus of dowel support, K_o, values for all shapes and sizes
- Investigation of form factors
- Investigation into the effects of oblonging of the hole for a large number of cycles
- The theory change for dowels used as expansion joints and larger joint widths

Test procedures

- Modulus of dowel support, K_o, values for all shapes and sizes
- Modifications to the AASHTO T253 test procedure
- Standardizing testing procedures and ASTM tests for dowel bars
- A distinction between whether laboratory and field measurements are true needs to be made
- Fatigue for a large number of cycles correlated with field cyclic results

Design procedure

- Development of FRP design procedure
- Investigation of form factors
- Development of a universal design procedure taking into account spacing and size of dowels
- Modulus of dowel support, K_o, values for all shapes and sizes

Parameter changes

- Modulus of dowel support, K_o, values for all shapes and sizes
- Investigation of load transfer efficiency of different shaped dowels at different spacings and sizes
- Investigation of form factors
- Fatigue for a large number of cycles correlated with field cyclic results

Other items

- Investigate criteria for large planes on runways and taxiways

FRP GAPS IN KNOWLEDGE

This past October a special workshop was conducted in San Francisco, CA, and sponsored by the National Science Foundation, American Concrete Institute, and ISIS of Canada. This NSF workshop concentrated on the use of FRP in conjunction with concrete structures, which includes topics affecting FRP dowel bars in concrete pavement slabs. In April 2005, a summary report was issued by Max Porter and Kent Harries, co-chairs for the workshop (Porter and Harries 2005). An Executive Summary is given below for reference, followed by items of interest to FRP dowel bar applications.

Workshop Executive Summary

> The research community has made great progress identifying and quantifying the characteristics of fiber reinforced polymer (FRP) composite materials used in infrastructure applications. A growing number of demonstration projects and commercial applications contribute to the knowledge base in this area. The use of FRP materials in infrastructure is still in its infancy, however, and a number of issues remain to be adequately addressed. In some cases, the dearth of understanding and/or design guidance represents a significant limitation to the broader implementation of FRP materials in infrastructure applications. The objective of this Workshop is to identify and prioritize research areas and issues which industry, practitioners and academia identify as requiring further attention in order to improve our understanding of the behavior of FRP materials and FRP structural and repair systems.

The objectives of the Workshop were as follows:

- Develop a consensus of the current state-of-the-art in the application FRP composites for infrastructure applications through a review of previous and pending research projects and field applications.
- Identify critical research needs affecting the implementation of FRP composites in construction applications and develop a consensus on the priority of these needs.
- Identify emerging and novel applications for FRP composites in infrastructure and identify additional research needs associated with these.
- Develop a coordinated plan for a unified research approach to addressing research needs and matching research needs with appropriate funding agencies/opportunities.
- Identify improved mechanisms by which research results may be disseminated in a manner appropriate to the implementation of FRP composites in infrastructure.
- Provide a brief assessment of research facilities and capabilities in the United States pursuant to the identified research needs.

The Workshop was held October 22-23, 2004 at the San Francisco Hilton immediately preceding the 2004 ACI Fall Convention. Forty nine participants representing academe, industry, and government from the United States, Canada, Great Britain, and Belgium were in attendance.

Seven workshop sessions were held addressing relatively broad topic headings. Topic-specific research priorities were established and refined in a final plenary session from which the final priority of research needs was established. This report documents the activities of the Workshop."

Workshop Items of Interest for FRP Dowels (and FRP with Concrete)

The following excerpt is taken from the NSF report by Porter and Harries (2005). Most of this applies in some way to FRP dowel or tie rods used in pavement slabs.

Current Research Needs

In order to address the immediate needs of the use of FRP in concrete construction, the following research topics are ***highly recommended*** for investigation. These topics address needs based on widely accepted applications of FRP in civil infrastructure.

Durability and Performance-Related Topics

Identification of Appropriate Environments for Durability Testing

There remains considerable confusion and disagreement among researchers and practitioners as to exactly what environmental parameters need to be considered when using FRP materials. Additionally, it should be clear that intended use, regional climates and maintenance practices will significantly impact which parameters affect a particular application. Such a research study must include participation from all primary climatic regions of the US (NE, NW, SE, SW, Mountains, Alaska, coastal exposure).

Development of Standardized Durability/Environmental Exposure Test Methods

Coupled to the previous topic, a consensus on accelerated environmental conditioning techniques and subsequent durability test methods is required. Methods are required for both external FRP applications and internal FRP reinforcement applications

Durability Studies of Externally Bonded FRP Repair/Retrofit Measures

Identify time dependent effects and factors affecting them (including fatigue). Clearly the durability of the adhesive bond and/or substrate-FRP interface is of primary concern.

Durability Studies of Internal FRP Reinforcement
Identify time dependent effects and factors affecting them (including fatigue). The critical issue here is the behavior of FRP *in situ*, thus studies must account for the concrete environment in which the FRP is embedded, the expected behavior (cracking) of that environment (which may differ from steel-reinforced members) and the environmental factors of importance (which also differ somewhat from those of importance for steel-reinforced members).
Service Life Prediction of Structures using FRP
Development of models to extrapolate short term test results to long term service life models. This topic also deals primarily with durability-related parameters and should involve models of degradation processes. Fatigue life of bonded FRP has been shown to of particular concern and predictive models of this behavior are required.
Fire Resistance/Protection of FRP
The behavior of FRP materials, whether imbedded in concrete or externally applied, subject to fire loading is largely unknown. Modeling techniques must be developed and verified for predicting fire performance of structures.
Seismic and Blast Resistance of FRP Systems
FRP is very often used for structural retrofit including efforts to mitigate the effects of earthquake or blast loads. Beyond pseudo-static testing, which does not capture strain rate effects, little is known of the performance of such retrofit systems under such extreme loads. Methods of assessing the appropriateness of existing and innovative FRP systems at mitigating the effects of extreme loading need to be developed.

New Materials and Systems
Innovative and Hybrid Materials
FRP materials are often not mechanically or hygrothermally suited to applications in concrete infrastructure. Research aimed at developing new and hybrid FRP materials having properties better matched to concrete is necessary. Such systems may be a simple as composite CFRP, GFRP and AFRP products or as innovative as polymer-free chemically prestressed systems.
Innovative Reinforcing Schemes
Taking proper advantage of the FRP materials should involve getting away from the paradigm of “replacing steel with FRP” and toward the development of innovative reinforcing schemes which should make both FRP reinforcement and concrete construction more cost-effective. One role that concrete plays in reinforced concrete systems is to protect the reinforcing system. If FRP systems can be made more robust and durable, this role for concrete becomes obsolete and should result in a savings.
Self-sensing FRP Structural Health Monitoring Systems
FRP materials are unique in terms of their properties and their fabrication which lends itself well to the development of integrated sensor systems. Such systems facilitate improved structural health monitoring and may be developed to allow the structure to interact with its occupants.

Overall, the items connected with durability received a high ranking in the gaps of knowledge. Interestingly enough to the author is that ISU started the durability research for FRP reinforcement approximately 17 years ago. A significant number of answers have been obtained, and today, FRP elliptical dowel bars have been used successfully in actual highway construction.

SUMMARY

The use of other sizes, shapes, and materials of dowel bars has been on-going at ISU. While success has been accomplished, additional information is needed. Many sizes of dowels have been included. The use of FRP dowels has merit in areas where possible corrosion exists. Elliptically-shaped FRP and steel dowel bars appear to provide benefit in the bearing contact with the concrete. Several key gaps in knowledge, as well as areas of FRP research, have been identified.

ACKNOWLEDGMENTS

The research described herein was conducted at Iowa State University in the Department of Civil, Construction and Environmental Engineering through the auspices of the Engineering Research Institute, the Center for Transportation Research and Education (CTRE), and the Center for Portland Cement Concrete Pavement (PCC center). Sponsorship of this research was provided by the Highway Division of the Iowa Department of Transportation (IDOT) and the Iowa Highway Research Board. The authors would also like to recognize and thank several people and firms for providing materials services and advice, including: James Cable and Doug Wood from Iowa State University, Doug Gremel from Hughes Brothers Inc. of Seward, Nebraska; James P. McCallion from RJD Industries Inc. of Laguna Hills, California; Technical Services Inc. of Ames, Iowa; Steve Tritsch of American Highway Technology (AHT) of Lenexa, Kansas; and John Busel of the Market Development Alliance of Harrison, New York.

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Multi-State Pooled Fund Effort to Understand Rural Intersection Crashes

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ABSTRACT

Staff from CH2M HILL worked with the University of Minnesota's Intelligent Transportation Systems Institute to understand the key contributing factors to rural intersection crashes in Minnesota. Preliminary discussions with several state traffic engineers found support for extending the research beyond Minnesota. This effort resulted in seven states (Georgia, Iowa, Michigan, New Hampshire, Nevada, North Carolina, and Wisconsin) joining a pooled fund effort to analyze crash records at their rural intersections to determine whether their patterns of crash types and contributing factors were similar to Minnesota's.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: crash data analysis—pooled fund effort—rural intersection crashes

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Performance-Based Rehabilitation and Maintenance for the District of Columbia's National Highway System

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ABSTRACT

Local and state Departments of Transportation (DOTs) face a common challenge. The Federal Highway Administration (FHWA) will provide Federal-aid funds to construct transportation assets, and they will provide Federal-aid funds to re-construct transportation assets, but maintenance funds generally must come out of the local or state agency's budget. This situation provides little incentive for effective maintenance, which can result in an expedited decline in asset conditions, necessitating reconstruction sooner than it should be needed. The objective of using Federal-aid funds for performance-based asset preservation is to break this cycle, improve the condition of the assets to a specified level, maintain the assets at or above the specified level, and thus, lengthen the life-cycle of the asset while providing better service to the public.

Performance-based asset preservation aims to rehabilitate and maintain roadway, roadside, bridge, and tunnel assets, while reducing overall rehabilitation and maintenance costs by encouraging innovative, cost-effective, flexible preservation strategies. The DC Streets contract, an experimental performance-

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based asset preservation project undertaken by the District of Columbia Department of Transportation (DDOT) and FHWA over the past five years, entails a private contractor, VMS Inc., maintaining over 75 miles of the National Highway System (NHS) in the District.

With this type of innovative contracting, DDOT has attained system condition improvements on the NHS and currently plans to continue using performance-based asset preservation contracts after the conclusion of the DC Streets effort. This paper describes the DC Streets project's daily, monthly, and annual assessments, lessons learned, innovative technologies, and contract management methods.

Key words: asset preservation—performance-based—urban

PROJECT INTRODUCTION

The DC Streets project, an experimental project undertaken by the District of Columbia Department of Transportation (DDOT) and the Federal Highway Administration (FHWA) over the past five years, entails a private contractor, VMS Inc., rehabilitating and maintaining over 75 miles of the National Highway System (NHS) in the District (Figure 1). This $70 million Federal-aid project is the first urban, performance-based asset preservation effort of its kind in the United States. Performance-based asset preservation aims to rehabilitate and maintain roadway, roadside, bridge, and tunnel assets, while reducing overall rehabilitation and maintenance costs by encouraging innovative, cost-effective, and flexible preservation strategies.

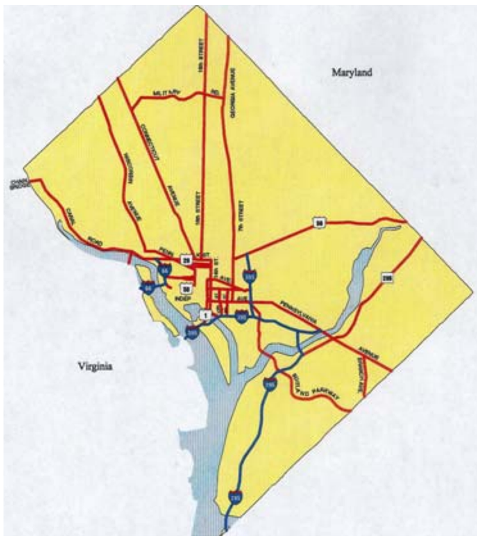

Figure 1. DC NHS roadways

PROJECT BACKGROUND

The 75 miles of the NHS in the District are heavily used by residents, commuters, businesses and tourists. The District's NHS infrastructure needs routine maintenance and timely preservation. The aging of the infrastructure and reduction in public sector staffing forced the District to look at alternatives, such as the DC Streets project, for timely preservation.

This project represents the first time that FHWA has teamed directly with a city government on a program to preserve their highway infrastructure. FHWA is providing management advice, engineering services for contract development, and annual evaluations of the asset preservation contractor with support from Science Applications International Corporation (SAIC), an employee-owned research and engineering company.

Asset preservation is the rehabilitation and maintenance of roadway and roadside infrastructure. Traditional asset preservation specifies what materials are used and what maintenance techniques and methods the contractor follows. This traditional method involves extensive engineering measurement and oversight, and the owner minimizes contractor risk. In performance-based asset preservation, the desired outcome is specified instead of the material or method. Thus, the contractor is instructed what to achieve, not how to achieve.

Assets covered by the project include tunnels, pavement, bridges, roadside assets (i.e., curbs, gutters, sidewalks, retaining walls), traffic safety assets (i.e., guardrails, barriers, attenuators, pavement markings, signs, lighting), roadway and roadside cleaning, drainage structures, roadside vegetation, pedestrian bridges, weigh-in-motion stations, and snow and ice control.

Problem Statement

Currently, local and state Departments of Transportation (DOTs) face a common challenge with transportation infrastructure elements. The Federal Highway Administration (FHWA) provides Federal-aid funds to construct these transportation assets, and they provide Federal-aid funds to reconstruct transportation assets, but maintenance funds generally must come from the local or state agency's budget. This situation provides little incentive for effective maintenance of transportation assets, which may result in an expedited decline in asset conditions, necessitating reconstruction sooner than it should be needed.

The objective of using Federal-aid funds for performance-based asset preservation is to break this cycle, improve the condition of the assets to a specified level, maintain the assets at or above the specified level, and thus, lengthen the life-cycle of the assets while providing better service to the public (Figure 2).

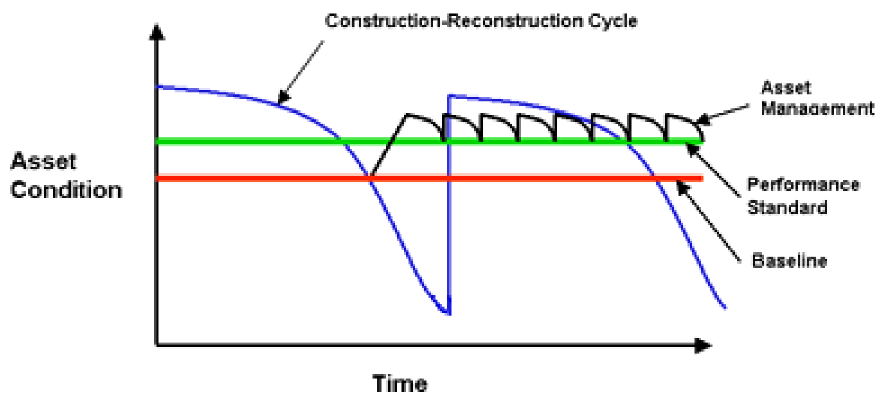

Figure 2. Traditional versus performance-based maintenance cycles

With this type of innovative contracting, performance measures and standards are specified instead of maintenance techniques. While challenges existed with developing the performance measures for such a broad range of assets from scratch, the project team was able to develop a set of measures to evaluate the condition of all assets, as well as the timeliness of the contractor's performance. These measures, along with the lessons learned from the project, may be used by agencies across the country as a basis for new performance-based programs.

The DC Streets project has allowed DDOT to both improve the condition of the assets on the NHS and allocate their in-house maintenance resources to local neighborhood streets. The effort has also provided local small businesses and residents with increased opportunities for employment.

PROJECT OBJECTIVES

Before the signing of the DC Streets contract, DDOT, FHWA, VMS, and SAIC determined that formal partnering would help to ensure the success of the project. The initial partnering session was held in 2000; the partners-to-be discussed personal views of owners, architects and engineers, the public, and contractors. This frank conversation allowed the participants to understand the paradigms that they held of each other. Given this understanding, the group developed Bold, Hairy, Audacious Goals (BHAGs) for the DC Streets project. The resulting goals became the objectives for the contract. The goals include the following:

- Enhance the NHS infrastructure by meeting or exceeding performance standards
- Receive positive feedback from the public and the press
- Revitalize the community
- Institutionalize performance-based contracting
- Produce innovative methods, procedures, and processes for infrastructure maintenance

The partners hold periodic meetings to plot the course for a successful completion of the DC Streets project. Part of this process involves recognition of notable achievements and discussion on burning issues and contract concerns; actions to resolve differences in interpretation and areas of dissatisfaction are identified.

One key aspect of setting goals is to measure progress periodically against those goals. The progress of the goals is measured primarily through evaluations. Performance measurement methodologies are described in the next section.

PERFORMANCE MEASUREMENT METHODOLOGIES

Evaluations

The project team measures performance using the performance standards on a daily, monthly and annual basis. On a daily basis, the project partners survey the system for deficiencies; the contractor maintains a daily log of activities as well. The monthly review is a subjective windshield survey, while the annual review is an objective engineering evaluation. Both time critical (i.e., timeliness of response) and condition-related performance measures are considered in the evaluations.

Monthly Evaluation Methods

Each month, members of the DC Streets team perform a subjective evaluation of the condition of the assets; Figure 3 shows the processes for the evaluations. The rating personnel are kept relatively consistent to help ensure comparability of the results from month to month. The monthly evaluations are video-recorded and distributed on DVD. A report for each evaluation is also generated. These deliverables help the contractor plan and perform work based on the deficiencies noted during the survey.

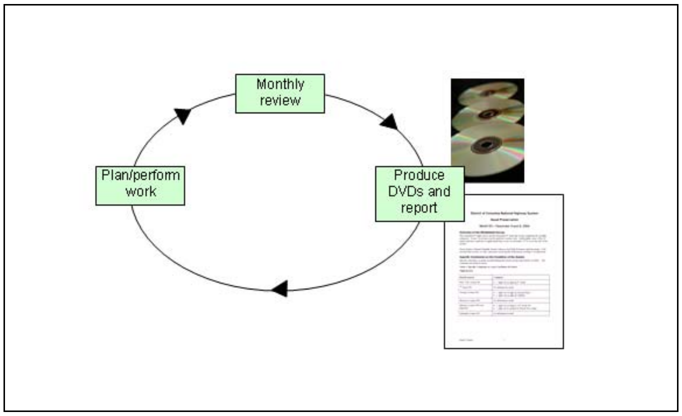

Figure 3. Monthly evaluation process

Annual Evaluation Methods

The purpose of the annual evaluation is to provide an evaluation of the condition of the assets and the timeliness of contractor response against the performance standards. Teams of subject matter experts perform this evaluation by rating the assets on randomly selected sample segments and rating time critical performance from DC NHS Tracker Database entries. Figure 4 shows the processes for the annual evaluations.

Most annual ratings are performed with an approximate 10% sampling rate. The SAIC team analyzes the data and uses the results to evaluate performance against the appropriate performance standards. For the final annual evaluation, a 20% sample may be used to provide a more comprehensive view of the project's status. A comprehensive report is provided to the project partners to present the results of each annual evaluation. Similar to the monthly deliverables, this annual deliverable helps the contractor to plan work.

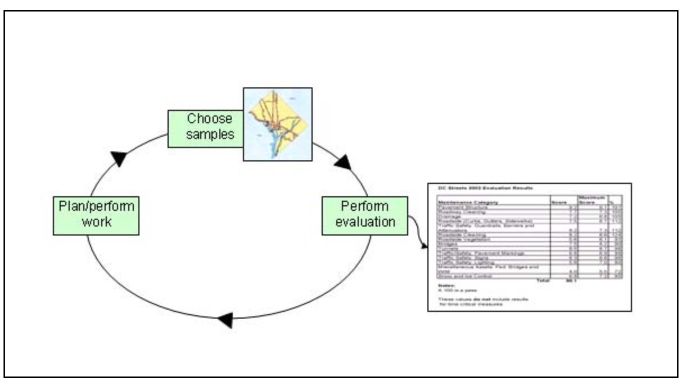

Figure 4. Annual evaluation process

Innovations

Practical solutions have been designed to address the challenges on the DC Streets project. The project partners use numerous innovative products and technologies, namely the Web Portal for project coordination and the Tracker Database for asset deficiency tracking and innovative sampling and data collection (Figure 5).

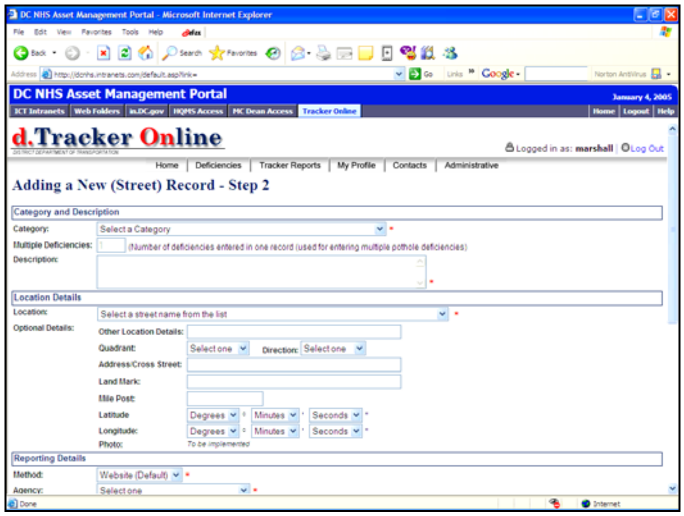

Figure 5. Tracker database

VMS began using a spray patch technology for potholes; this innovation provides a departure from the conventional cold mix operation that generally requires several people and more traditional lane closures to complete the patching safely. The innovative technique was tested by the Strategic Highway Research Program (SHRP); the technique has been used on the project with no failures to date. Previous uses of this method have lasted for several years.

VMS also introduced a new way to clean tunnel walls; it uses a three-step process, which includes a preliminary wash, sealing and an additional wash. Tunnel walls are washed a couple of times before they are sealed. Deionized water mixed with a detergent is sprayed onto the walls; this substance sticks to the walls for a specific dwelling period. A hot water rinse is the next phase. Next, the walls are sealed with a siliconized mineral, which is sprayed onto the clean wall. The primary purpose of the mineral is to remove the electrostatic charge that holds the dirt to the wall. After the walls are sealed, they are washed with a mixture of detergent and water again, but this mixture does not stick to the walls. The walls are then rinsed with plain water.

In early 2005, VMS presented a new material and method of replacing neoprene joint seals on bridges; the new material is a malleable two-part silicone. There are three main advantages of this new material. It can accommodate any joint up to 3″, including irregular widths. The same material can be used for all surfaces, crack widths, and shapes. The silicone bonds to itself; thus, there is no need to replace large areas as needed with neoprene seals.

The effort also has produced multiple visual tools (i.e., video dissemination of the infrastructure condition, snowfall tracking via interactive map) that communicate the results of the project to the various internal and external stakeholders. The interactive snowfall-tracking map is shown in Figure 6.

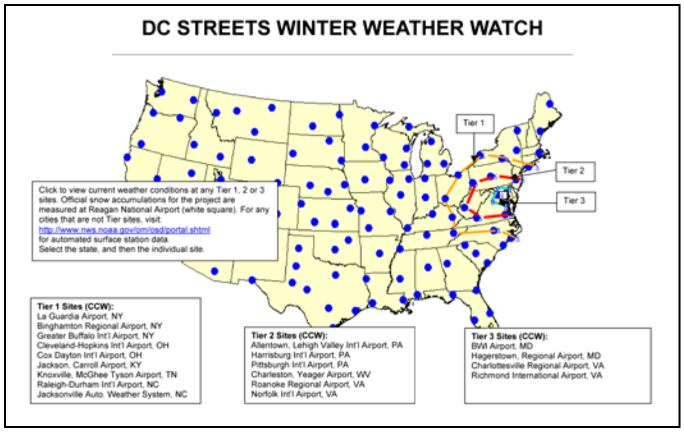

Figure 6. Interactive snowfall-tracking map

KEY FINDINGS

Results to Date

Monthly Results

In an effort to enhance the NHS infrastructure by meeting or exceeding the performance standards, the project partners track the results of the project to date. For example, an overall and well-informed subjective score is computed monthly so the project team may adjust strategies or methodologies as needed. The raters assign subjective ratings of good, fair, and poor at the maintenance element level. A rating of good is assigned if the rater felt that the maintenance element appears to meet the performance standards; fair is assigned when the maintenance element may not meet the performance standards. A rating of poor is assigned when the condition of the maintenance element is clearly below the performance standards.

A summary of the results through Month 56 shows that there has been substantial subjective improvement over the course of time (Figure 7). The worst month was Month 3 with approximately 70% poor ratings. The best month was Month 41, when 99% of the ratings fell in either the good or the fair categories. The first month covered only Interstate 295 and is not shown.

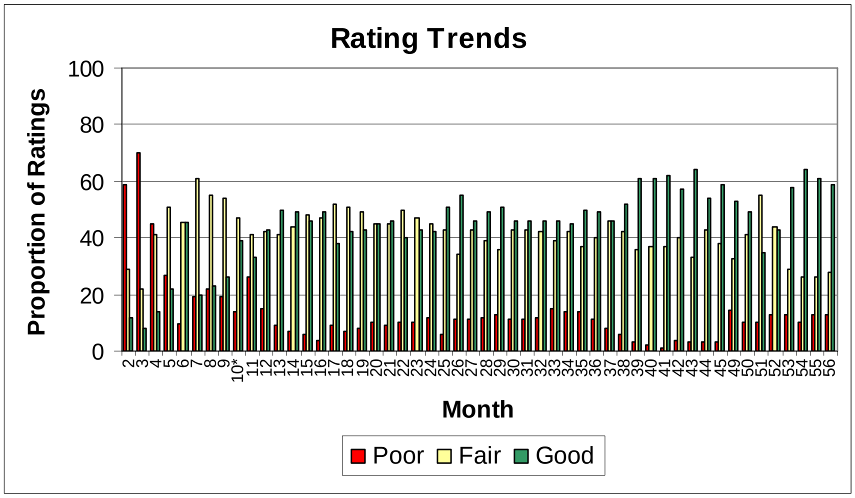

Figure 7. Monthly evaluation results

The rating trends chart is useful, but the project team desired a means of tracking monthly performance with a single measure. In response to this desire, a monthly score was computed from the proportions of subjective good, fair, and poor ratings assigned by the evaluation team (Figure 8). The proportion of good ratings is multiplied by a factor of 100, the proportion of fair ratings by 50, and the proportion of poor ratings by 0. The resulting values are added together to obtain the score out of 100.

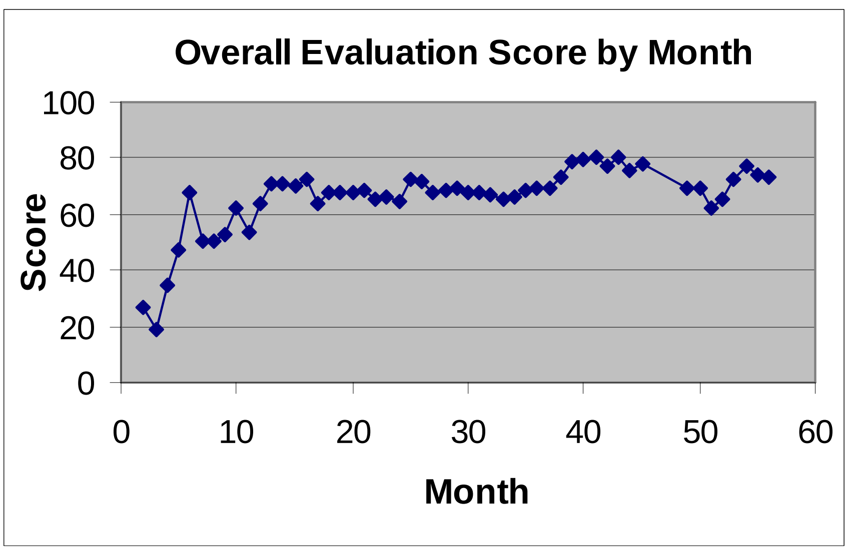

Figure 8. Overall monthly score

The overall evaluation score chart shows that the subjective score decreased between Months 44 and 53, and then increased in Month 54. It has decreased slightly in the last two months. The project partners are currently developing action plans to continually move towards exceeding the performance standards.

Annual Results

For the annual evaluation, the team enters resulting scores into a complex scoring spreadsheet, and an overall score, as well as a score for each maintenance category, is computed (Figure 9). A score of 100 indicates that, on average, the performance standards were met. For fair comparison purposes with pre-DC Streets conditions, time critical performance data are excluded in these scores.

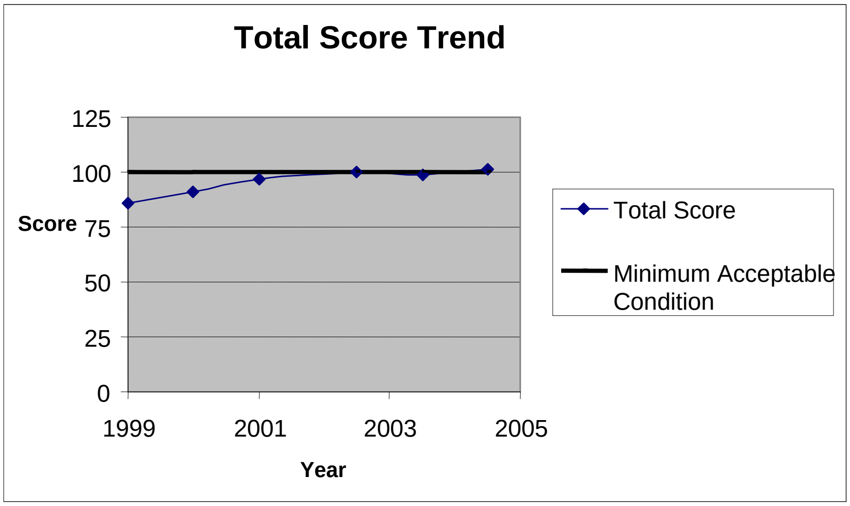

Figure 9. Overall annual score

Further Goal Results

The project partners also strive to reach the other BHAGs they developed. For example, the editor-in-chief of FHWA's Better Roads magazine attended the annual TRB conference in January 2004. She was impressed with the improvements and conditions of the District's streets and thus contacted VMS' corporate office. She wrote a four-page story in the April 2004 issue of the magazine.

In performing the DC Streets effort, VMS made a number of efforts to revitalize the community in which they work. VMS supported the Metropolitan Police Department's "Operation Fight Back" on numerous occasions. Operation Fight Back involves a number of different approaches, including carrying out building and business inspections, towing abandoned vehicles, cleaning up trash, and reaching out to residents in the community. VMS supported this operation by performing street sweeping, mulching, graffiti removal, tree trimming, and sidewalk repair.

The main project factors that will help to institutionalize performance-based contracting will be the success in meeting and exceeding the performance standards and in providing a noticeable aesthetic improvement to the transportation network. For transportation professionals and politicians to find out about the good things happening in the DC Streets project, it is helpful to present at conferences and to publish in transportation journals.

Lessons Learned

A number of lessons have been learned over the first 4.5 years of the contract. For example, formal partnering is important for keeping good relationships between partners, solving problems not easily resolved at the daily project level, and avoiding claims. Additionally, a contract cannot cover every conceivable issue; project partners must work together within the spirit of the contract to make the project work.

The urban environment is dynamic and challenging to work in due to heavy traffic, limited right-of-way space, and the large number of people/organizations to work with.

Regular team meetings help keep outstanding issues in focus until they are solved. Assigning action items to specific individuals and then following up on the items at the next meeting improves the proportion of action items accomplished.

It is helpful to choose the monthly evaluation route randomly from a set of established routes. Doing so avoids focusing solely on problem areas and includes all sections of the system. It is easier on the driver as well. Performing routes in the reverse direction provides a different and useful perspective. A videographic or photographic record of the system should be captured on the day before the project starts.

It is important to keep multiple people from each partner organization involved in and informed about the project. This activity greatly eases the transition when a key person leaves.

Management IT tools are necessary. It is also useful to have different tools, such as the DC NHS Web Portal and Tracker Database, communicate directly with each other.

CONCLUSION

DDOT plans to continue using performance-based asset preservation contracts after the conclusion of the DC Streets effort. The agency currently is expanding the application of the concept District-wide for assets, such as lighting and tunnels. DC Streets II, a follow-on contract that will include several DC NHS asset categories, also is being developed.

Workforce Recruitment Dilemma: Defining Transportation and Transportation Careers

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ABSTRACT

The ambiguous definitions of “transportation” and “transportation careers” are potential obstacles to effective recruitment of high school and college students. With the transportation industry beginning to experience a serious worker shortage at all skill and education levels, attracting young people to the field has become critical.

The purpose of this study was to learn what, if any, differences exist between college students and transportation professionals in their understanding of the term “transportation.” A secondary purpose was to have both groups evaluate a list of transportation-related job titles according to how well they fit the field of transportation. This small study included 52 first- and second-year students at Iowa State University and Des Moines Area Community College and 39 transportation professionals with experience in academia, consulting, and local, state, and federal government.

Both students and transportation professionals defined “transportation” primarily as movement of people and goods. But when the groups rated job titles, students rated jobs higher (i.e., more “transportation-ish”) that were more clearly related to the modes of transportation, e.g., truck driver. Transportation professionals, on the other hand, rated engineering jobs—jobs aligned with the transportation infrastructure—higher. This difference in understanding of transportation careers suggests why the term “transportation” is so nebulous when it comes to describing possible careers. The physical, social, and political infrastructure that enables the movement of goods and people is essentially invisible to, and therefore not comprehended by, students. This invisibility is a major roadblock to effective recruitment.

Key words: recruiting—transportation careers —workforce development

INTRODUCTION

When high school and college students hear the term “transportation,” what comes to mind? Moving goods and people? Driving somewhere? Maybe taking a plane? What generally does not come to mind is the transportation infrastructure—the roads and bridges, rail lines and runways. These elements are invisible to them.

Transportation can be a vague, even misleading, word, so it’s not surprising that workforce development efforts, especially those targeting children, avoid the term. The concept of transportation careers is nearly as ambiguous. What job titles does that term encompass? In the *Des Moines Register*’s help-wanted ads, for example, one category of jobs is “Automotive/Transportation.” The majority of the jobs advertised in this section is usually truck drivers. While moving goods across the country is important and necessary work, it does not reflect the breadth of the transportation career field. Misperceptions about transportation careers compound the recruiting problem.

Yet the field of transportation is a great industry for people looking for long-term work. According to the National LTAP (Local Technical Assistance Program) Association, nearly half of the current transportation workforce may retire by 2010. The U.S. is beginning to experience a serious worker shortage at all skill and education levels.

Attracting young people to transportation careers, particularly careers related to the transportation infrastructure, has become critical. But designing, developing, and maintaining the infrastructure can be an invisible function to young people and to their parents, teachers, and guidance counselors. Even when the work is visible, as in the case of road work zones, high school students (and most adults, for that matter) have no idea what kind of work and planning is done before and after that work zone goes up.

There is a fundamental communication gap between transportation professionals and laypeople about the work that goes on to keep this country moving. Because of this communication gap, recruiting young people into professional and non-professional careers in transportation can be particularly challenging.

PURPOSE OF THE STUDY

The purpose of this study was to show the differences between college students’ understanding of the term “transportation” and transportation professionals’ understanding. My hypothesis was that young people think of transportation in terms of modes or means of getting around whereas transportation professionals use a broader definition to include the infrastructure. A secondary purpose was to have both groups evaluate a list of transportation-related job titles according to how well they fit the field of transportation. I anticipated that students would rate jobs higher (i.e., more “transportation-ish”) that are more clearly related to the modes of transportation, e.g., truck driver. On the other hand, I anticipated that professionals would tend to rate jobs higher that are related to the transportation infrastructure, e.g., civil engineer.

BACKGROUND INFORMATION

Overview of Topic

This study focused on two things: (1) people’s definitions for the term “transportation” and (2) their understanding of how “transportation-ish” certain job titles are. The study compared responses from young people, specifically college students from Iowa State University (ISU) and Des Moines Area

Community College (DMACC—Boone Campus) enrolled in first-semester English courses, and adults working in the transportation industry.

Previous Studies

In Linguistics

Scientists such as linguists and psychologists have wondered what the relations are between words and mental categories. Prototype theory attempts to explain how categories of words can be understood as groupings of meaning. So a word like "chair," for example, may include a seat off the floor and a backrest. Does a prototypical chair need to have legs? If so, how many?

Several researchers, particularly Rosch, Armstrong, and Gleitman have conducted studies using well-defined categories of concrete nouns such as fruit, furniture, vegetables, etc. Subjects rated exemplars of a given category on a seven-point scale. Fruit, for example, might have the following exemplars: apple, banana, orange, plum, lemon, and kiwi. Each type of fruit would be rated individually according to how "fruity" it seemed to each subject.

To my knowledge, however, "transportation" has not been studied in this way.

In Workforce Development

Researchers exploring workforce development issues in the field of transportation have not tried to define "transportation." Instead they've focused on the more easily definable "civil engineer" (easier because it refers more clearly to a person rather than a whole field; i.e., *doctor* rather than *medicine*).

From a brief review of literature related to recruitment, it's clear that careers in transportation are not generally known or understood by most high school students or by the adults in their lives. Other research and outreach projects have focused on more readily identifiable careers.

In a recent project sponsored by Minnesota's Local Road Research Board, researchers developed recruiting materials about careers as a civil engineer and civil engineering technician. An older (1992) National Cooperative Highway Research Program (NCHRP) report also focused on civil engineering. According to researchers for NCHRP Report 347, *Civil Engineering Careers: Awareness, Retention, and Curriculum*, "The term 'civil engineer' means almost nothing to those who are uninformed. The image of the civil engineer is related typically to road construction or maintenance, or working for local government at a low salary."

For the past several years, the Federal Highway Administration has been promoting careers in construction by helping sponsor "Construction Career Days" for high school students across the country. These events are not restricted to transportation-related construction (dry-wall hanging and electrical work are some of the suggested hands-on activities). While these recruiting efforts are certainly useful, they ignore the breadth of transportation careers available and needed within the industry. One reason for this may be the difficulty with defining the terms *transportation* and *transportation careers*.

PROCEDURE

Two separate one-page surveys were prepared—one for students and one for professionals in transportation. The student survey included questions about gender, age, year in school, and the following questions:

- What does the word "transportation" mean to you?
- How is transportation (based on your definition above) part of your daily life?

Students were also asked to rate 14 different job titles on a scale of 1 to 7 to show how well they felt each title fit the career field of transportation. These job titles were selected to represent the breadth of the transportation industry, from those responsible for building and maintaining the infrastructure to those responsible for moving goods and people using that infrastructure, and included the following titles:

- Civil Engineer
- Truck Driver
- Traffic Safety Engineer
- Public Policy Analyst
- Land Surveyor
- Motor Grader Operator
- Urban Planner
- State Trooper
- Bridge Engineer
- Construction Inspector
- Fleet Dispatcher
- County Engineer
- Airline Pilot
- Pavement Materials Technician

The specific instructions were these: "Following is a list of job titles. Please rate each one on a scale of 1 to 7 to show how well you feel a title fits the career field of transportation. A 1 rating means you feel the job title is a very good example of your idea of a career in transportation while a 7 is a poor example. A 4 means you feel the job title fits moderately well. Use the other numbers to indicate intermediate judgments. Circle your rating for each."

The survey of transportation professionals included the same questions about gender, the definition of transportation, and the rating of job titles, as well as the following questions:

- How many years of work experience do you have in the field of transportation?
- Within the field of transportation, what types of organizations have you worked for? (options included public agency (city, county, state, federal); private; education/training; research; and other)

Subjects

One of the purposes of this study was to compare young people's definitions of transportation with those of transportation professionals. First-year college students were chosen since their high school attitudes and experiences are likely recent and their responses can be extrapolated to high school students, a primary target population for recruitment into the transportation workforce. Two separate groups of students were surveyed—ISU students and DMACC students.

Both groups of students were selected based on their enrollment in their schools' respective required first-semester English courses. At ISU, two sections of English 104 were surveyed. Thirty-nine surveys were completed and returned out of 45 enrolled in the courses. At DMACC, three sections of English 110 were surveyed, a potential enrollment of 60 students; 13 surveys were completed and returned.

In total, approximately 100 students were surveyed; 52 surveys were returned. Sixty percent of the students were female, and 40 percent were male. Of the 52 students, 75 percent were in their first year of school. The majority of the students (80 percent) were 18 or 19 years old. Nine students were in their twenties, two in their late twenties, and one was 45. Of the three non-traditional students (i.e., older than 25), two were female, including the 45-year-old.

The transportation professionals were selected based on my own knowledge of their backgrounds. Thirty surveys were distributed to staff and graduate students (with significant professional experience outside of academe) at Iowa State University's Center for Transportation Research and Education (CTRE). The same day the surveys were distributed, a course was being held at CTRE for transportation professionals (the course was Designing Pedestrian Facilities for Accessibility). On the spur of the moment, I asked the instructor if I could survey his students in order to get an even broader range of responses. In all, 52 surveys were distributed to professionals, and 39 were completed and returned.

Of the 39 professionals, 18 percent were female and 82 percent were male. Their years of experience in the transportation field range from two years to 44 years. The average number of years of experience is 16.9. Women averaged 8.6 years of experience while the men averaged 18.3 years.

Ninety-five percent of the professionals surveyed have worked in a public agency at the city, county, state, and/or federal levels. Thirty percent of the group with public sector experience has worked only in the public sector while the other 70 percent has worked in one or more additional sectors, such as private industry, education, and/or research.

RESULTS

Students' Definitions of Transportation

Students overwhelmingly defined the word "transportation" in terms of movement from one place to another. The words "move" or "moving" were used in 17 percent of the definitions while "get" or "getting," as in "get around," were used in 73 percent of the definitions. Sample student definitions are as follows:

- Means of getting from point A to point B.
- A means of getting around or moving from one place to another.
- Getting in some sort of vehicle to go somewhere. Getting from place to place.
- Transportation is a way to get from one place to another.
- Means to get from one place to the next without complications.

The fact that all of the student surveys were completed in a classroom setting may account for much of the similarity in the definitions. That is, students may have shared their thoughts. Nevertheless, it's interesting to note that a small handful of students suggested a broader definition beyond moving from one place to another. For example, some students mentioned the route and mode, as well as what or who gets transported. See the following examples:

- The way you get from one place to the next and **what route you take**.
- Getting around or going from place to place by a **means of either a car, bus, train, plane, etc**.

- This word means to me a process of getting from one place or another whether it is **people, goods, or nutrients in a plant, etc**.
- **A device used** to get you from one destination to another.

The survey question about how transportation is used in daily life sheds more light on students' individual understanding of the word. Answers to this question were more diverse. Forty-six percent of the students referred either to "class," "school," or "work" in their responses to this question as destinations and/or reasons for using transportation. Forty-eight percent referred to walking, driving, riding a bike, or taking the bus. Student responses to this question emphasized transportation as a means to move from place to place, such as from home to work or school, via several modes. One response hinted at a broader definition of transportation that would include the infrastructure: "It is everywhere whether it is cars, buses, bicycles, walking, airplanes, streetlights, roads, sidewalks."

Professionals' Definitions of Transportation

Of the 37 responses to this question, 76 percent of transportation professionals included either "get" or "move" or "movement" in the definition. Examples of the most common definitions are as follows:

- Transportation is an organized way to move people or goods.
- The movement of people or things in an efficient manner.
- Anything related to the movement of goods or people from one place to another.
- A means of getting from one location to another.

A few definitions included infrastructure-related terms such as the following:

- Pedestrian facilities, roadway design, traffic management.
- **Design and development of systems** for moving goods and people from place to place.
- Things related to roads/highways.

Students' Ratings of Job Titles

Students rated 14 job titles on a seven-point scale, with 1 being a "good" example of a transportation career and 7 being a "poor" example. Averaged together, students rated those jobs closer to 1 (good) that were involved in moving people or goods, e.g., airline pilot (1.67) and truck driver (1.96). Jobs that were rated closer to 7 (poor) were land surveyor (4.0) and public policy analyst (4.5). Ratings ranged widely between students. Most of the job titles received at least one rating on each point of the scale. See the averaged student ratings for all job titles in Figure 1. The difference in average ratings between male and female students was one point or less (on the seven-point scale). The jobs with the largest difference were state trooper, airline pilot, and truck driver.

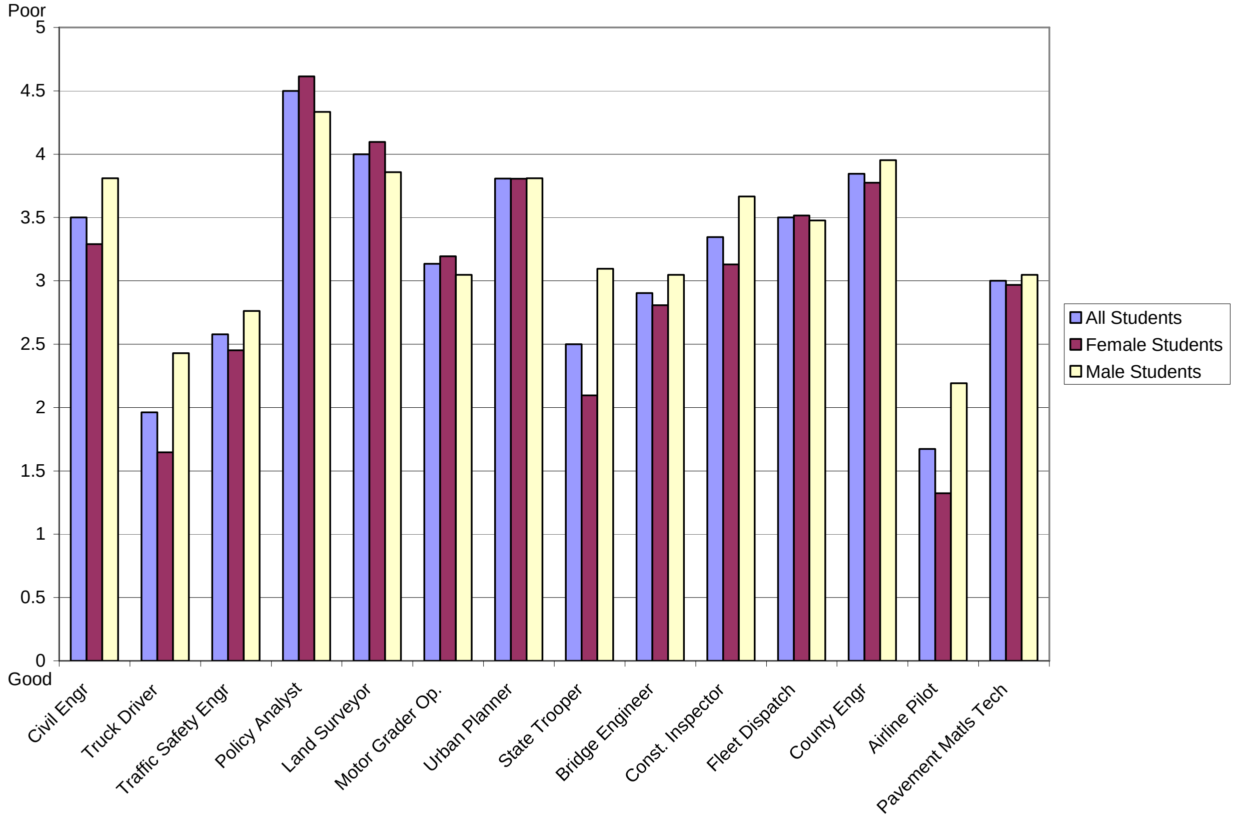

Figure 1. Students' average ratings of job titles

Professionals' Ratings of Job Titles

When their ratings were averaged, professionals tended to rate positions related to the infrastructure as "good," including traffic safety engineer, civil engineer, and bridge engineer (see Figure 2). The title of public policy analyst earned the most "poor" ratings, as it did among the students.

Since there were relatively few women professionals surveyed, it's difficult to know whether the differences in ratings show any particular trends based on gender. Other factors may be at work, such as years of experience and/or experience with different types of organizations.

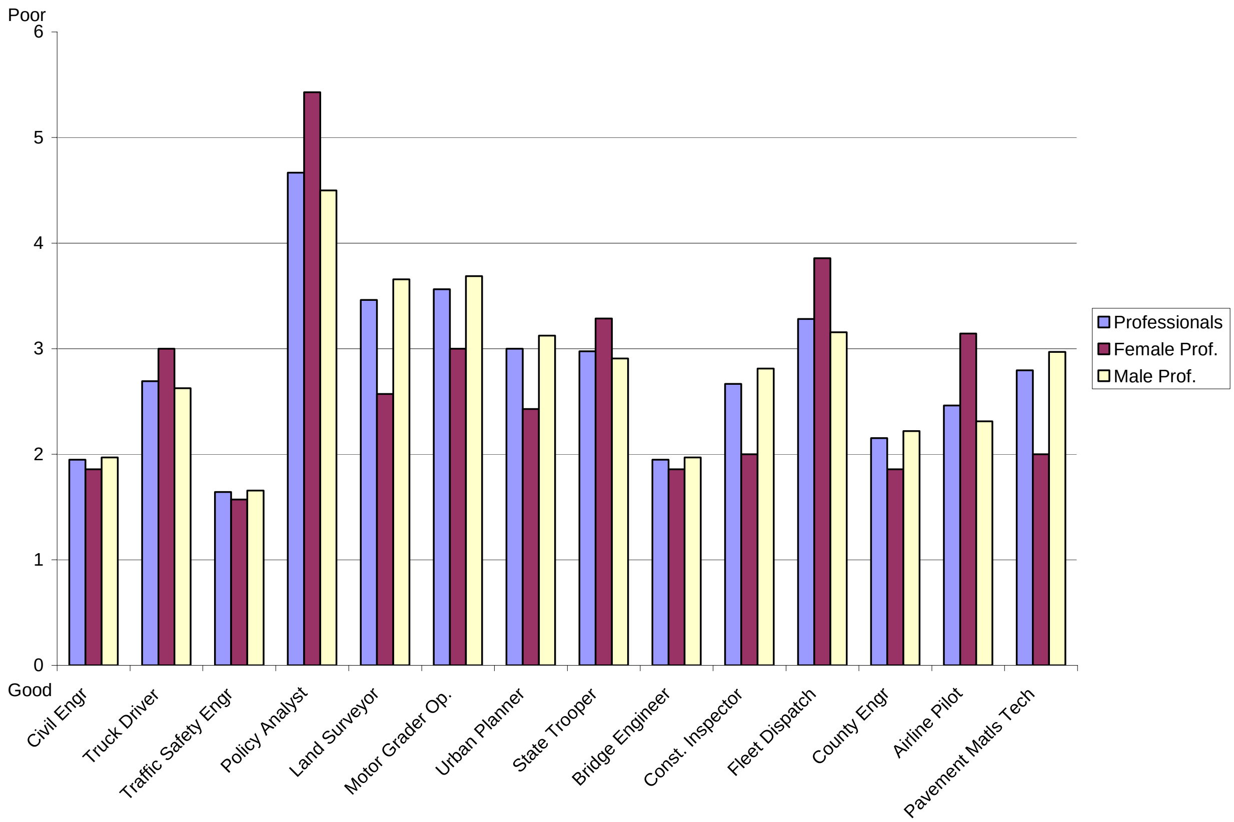

Figure 2. Professionals' average ratings of job titles

Students' and Professionals' Ratings Compared

Differences in the way students and professionals rated the various job titles stand out more clearly when the jobs are grouped together, as in Figure 3. The infrastructure careers grouping includes professional-level jobs requiring a four-year degree or more. Within this grouping, the job titles of county engineer and civil engineer had the widest difference in average rating between students and professionals. Professionals rated both titles considerably higher than students did. This difference may be due to professionals' greater familiarity with the jobs. It was noted in another study, mentioned earlier, that young people are typically unfamiliar with the term "civil engineer."

For jobs related to moving goods/people, the differences in ratings were less pronounced, although students tended to rate these jobs closer to "good" than professionals. For jobs related to road construction or maintenance, the difference in ratings was probably the least pronounced of all. Both groups rated land surveyor, motor grader operator, construction inspector, and pavement materials technician in the 2.5 to 4 range (on a seven-point scale), with slight differences between groups for any particular job.

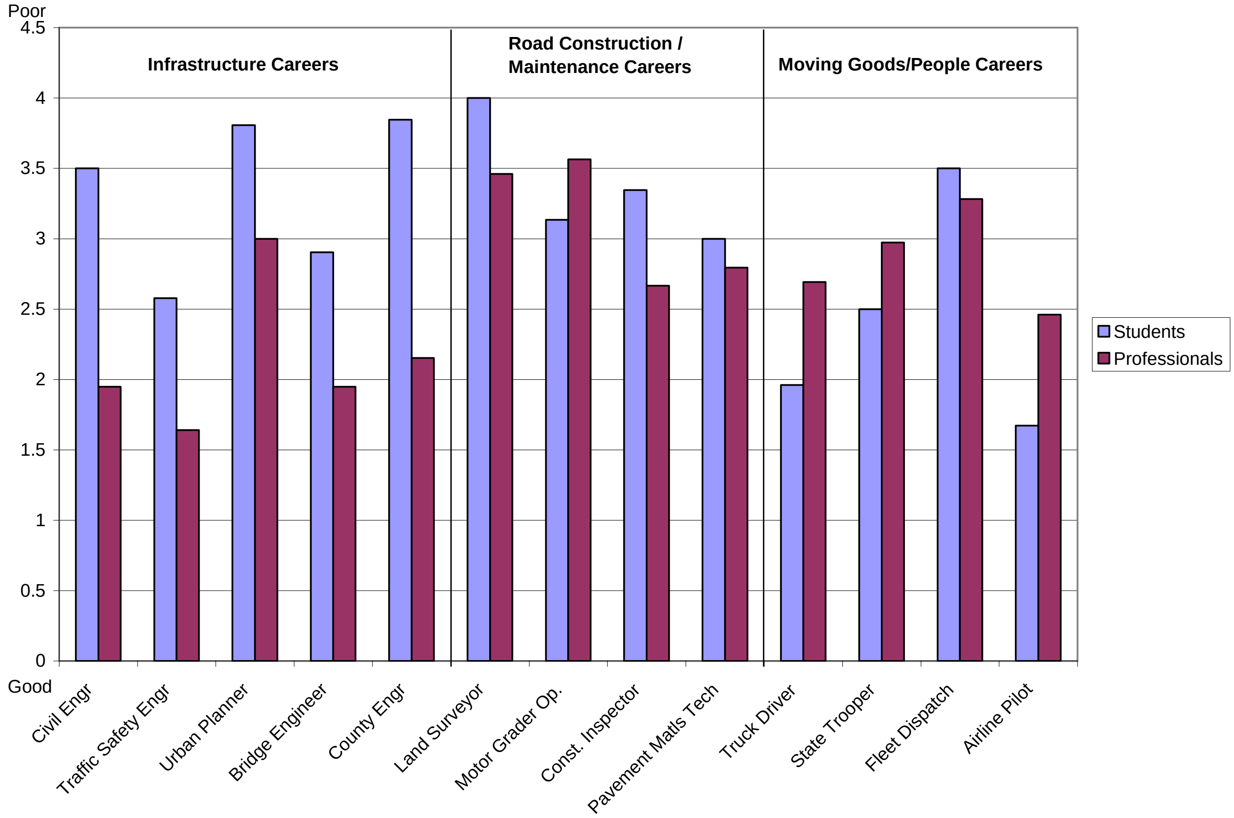

Figure 3. Students' and professionals' average ratings compared across groups of careers

DISCUSSION AND CONCLUSIONS

People already in the business of transportation infrastructure, i.e., city, county, and state transportation agencies, have a decidedly different understanding of "transportation" and "transportation careers" than young people. Knowing how laypeople, especially people who are potential recruits to the field of transportation, define "transportation" and understand transportation careers will help recruiters develop more effective recruiting materials.

Students defined "transportation" in ways that I anticipated—primarily as movement rather than as the infrastructure that supports the movement. I was somewhat surprised, however, by the transportation professionals' definitions. Generally, they defined it in terms of movement of goods and people. As people who are mostly engaged in work related to the transportation infrastructure, I expected their definitions to include more about the infrastructure or about transportation systems. So, the fact that the differences in the groups' definitions were not pronounced was surprising.

Given the professionals' general definitions of transportation, I expected their ratings of jobs related to moving people or goods to be higher (i.e., more "good" ratings). Instead, they rated engineering jobs more highly. This seems inconsistent with their definitions of transportation. It also suggests why the term "transportation" is so nebulous when it comes to describing an industry with a vast array of jobs.

This small glimpse into how students and transportation professionals think about transportation and transportation-related jobs should be useful for workforce recruiters. The only context students were provided for the job titles they rated was the term “transportation.” If students aren’t even aware of the transportation infrastructure that helps them to “get around,” it’s unlikely they’ll be drawn to careers in that sector of the industry.

Interviewing students after they completed this survey may have shed light on how and why they rated different jobs. This is something I would add to this project were I to do it again. Similar interviews with transportation professionals would shed light on their thought processes as well.

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Quantifying the Safety Benefits of Traffic Control Devices: Benefit-Cost Analysis of Traffic Sign Upgrades

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ABSTRACT

Every traffic professional has a goal of making the system safer for all drivers. However, finding new money is difficult, and existing funding sources are stretched to the limits. Combine that with most public agencies' other priorities and the safety upgrades, while valued, do not measure up to the economic development projects or other high-profile expenditures. Traffic control devices have long been understood to be some of the most cost-effective methods for addressing safety problems and hazard locations. Typically, this is framed in new installations where the signs are providing new or additional information.

In 2004, the author completed a study that identified four separate locations that had completed sign upgrades. Each location changed signs from ASTM Type 1 to ASTM Type 3 or 9. The reasoning and circumstances were different in each case but all showed positive results in their crash experience when compared to a period before the changes. Since the differences between the applications varied greatly, as did the readily available before and after information, it was difficult to identify with any certainty the crash-benefit analysis for a practitioner trying to sell a similar project to financial decision makers.

This research studies the upgrade programs implemented by Sioux City, IA, Mendocino, CA, Putnam County, NY, and the Insurance Institute of British Columbia, Canada, and identifies other projects that upgraded sign materials. The research evaluates each implementation practice and examines the crash results, including crash types, sign locations in relation to the crashes, time of day, light vs. dark, weather, and other factors that may have influenced the signs' effectiveness. In addition, state and/or national baseline crash information for the same time periods will help establish trends.

The purpose of the research is to establish and document a benefit-cost ratio that can be used by practitioners to anticipate the safety benefits compared to the costs incurred by a signing system upgrade. This information is intended to be especially useful when comparing various safety programs while evaluating the numerous demands on the financial resources of a public agency.

Key words: benefit to cost ratio— safety—signs

INTRODUCTION

With the challenge to traffic professionals to make roadways safer without increasing budgets, and being subject to strong oversight and demand for instant results, few safety improvements can have such a resounding and widespread impact as upgrading traffic signing.

According to the Manual on Uniform Traffic Control Devices (MUTCD), "The purpose of traffic control devices, as well as the principles for their use, is to promote highway safety and efficiency" (FHWA 2003). The MUTCD continues, "Traffic control devices notify road users of regulations and provide warning and guidance needed for the reasonably safe, and efficient operation of all elements of the traffic stream."

It has long been accepted that making signs more visible increases their effectiveness and therefore safety. In a 2003 research report completed for the Minnesota Department of Transportation (MN/DOT) to address safety at rural intersections (Preston and Storm 2001), some key observations were made regarding signs:

- "At intersections with crashes, the use of more and larger stop signs appears to reduce the number of ran-the-stop crashes."
- "The use of brighter retro-reflective sheeting material appears to reduce the frequency of both total crashes and right-angle crashes. The highest usage of Diamond Grade sheeting was at intersections with no crashes and the lowest usage was at intersections with multiple ran-the-stop crashes."
- "'Stop ahead' signs were in place at all but one intersection. At intersections with crashes, it appears that the use of larger, bright advance warning signs reduce the frequency of run-thru-the-stop crashes."

The MN/DOT study concludes, "Increasing the conspicuity of traffic control devises by using bigger, brighter, or additional signs and markings appears to lower the ran-the-stop crashes." The MNDOT study was focused on rural intersections and a specific type of crash, but the conclusions have broader application, in that the use of brighter signs improves the conspicuity of signs and therefore the compliance of signs resulting in improved roadway safety.

Many cities, counties, and DOTs have been following this principle for years and have felt that the practice has improved the safety of their respective roadways. This study looks at four communities that implemented sign upgrade programs in different ways and reviews the results of the improvements for comparison purposes.

The implementation methods of each organization are as diverse as the populations they represent. However, in cooperation with the organizations, this review highlights the process and results of four programs intended to improve the safety of their roadways through sign system upgrades by improving the reflectivity and visibility of material used for sign sheeting.

The agencies reviewed are spread across North America, as shown in Figure 1, and have implemented sign program upgrades in different ways. Despite the differences, each program has had similar results. The sign upgrade programs studied include the following:

- City of Sioux City, IA
- Insurance Corporation of British Columbia
- Mendocino County Department of Transportation, CA
- Putnam County Highway Commission, NY

Figure 1. North American study locations

In each case, the sign programs consisted of upgrading existing engineer-grade signs to high-intensity (ATSM Type III) and/or Diamond Grade™ (ASTM Type IX) reflective sheeting. The methods and rational where different in each case and provide a good illustration that there are many ways to implement a sign safety program. In each case, the results have been extremely positive, such that the programs have been continued and in most instances expanded. A summary of each case study follows.

SIOUX CITY, IOWA

The City of Sioux City, IA began a broad-based upgrade of its traffic signs throughout the community of 85,000 people in late 1995 and 1996 (Carlson 2004). Prior to the project, the city used exclusively engineer-grade (ASTM Type I) reflective sign sheeting on city routes. The program consists of a complete inventory change-out from engineer grade sheeting to Diamond Grade™ (ASTM Type IX) reflective sheeting for all sign series, except parking. The aggressive upgrade process began as a high-intensity upgrade program in late 1995, but Diamond Grade™ (ASTM Type IX) material replaced the high-intensity material in 1996. The city began replacing approximately 10% of the total sign inventory per year with 3Ms VIP™ (ASTM Type IX) material in 1996.

Figure 2 (Office of Traffic and Safety 2004) illustrates the before and after crash numbers and shows an actual reduction of more than 700 crashes from 1993 to 1998. The actual crashes were reduced by approximately 30% and continue through 2001. Figure 2 also shows the reduction of actual crashes as well as the night/day ratio reduction for the years in which data are available.

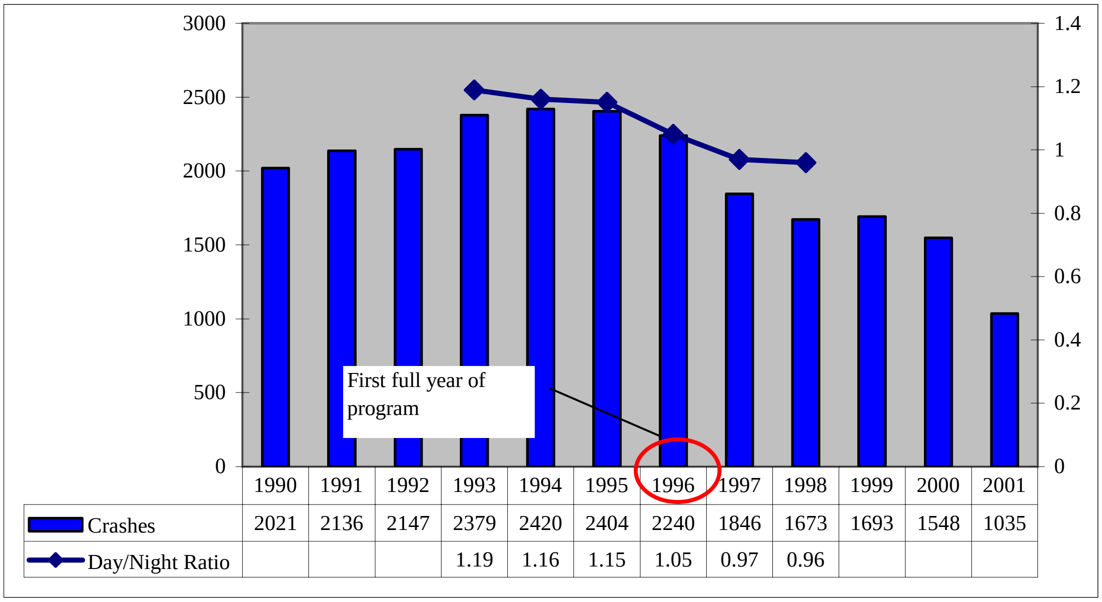

Figure 2. Sioux City crash experience

The program has been successful, according to R. Scott Carlson, city traffic supervisor: “Not only have total accidents gone down, but the night/day ratio is a real testament to higher visibility signs and durable/better retro-reflective pavement markings” (Carlson 2004).

A summary of the program in the Iowa Traffic Control and Safety Association newsletter compared the total number of crashes to the amount of traffic on the roadways. The newsletter states, “The crash rate on Sioux City streets has fallen from 6.53 accidents per million miles in 1995 to 4.03 in 1998 using traffic volumes from the Iowa Department of Transportation and crash records from Access-ALAS.” The article continues, “Based on these observations, it could be concluded that the enhanced visibility of upgraded signs and markings has been effective in contributing to reduced crashes in Sioux City, especially at night” (Iowa Traffic Control 2001).

The city estimated the cost of the program to be $ 144,925 from 1997 to 1999, and the city estimates a cost savings to the public of $ 4,920,900 from the reduction of crashes over the same period using the average crash cost of $2,350. Using the city’s calculations, the benefit to cost ratio for their program was $33.95 to $1.00 over the three-year period. In addition to the sign system improvements, the city also began an upgrade to durable, more reflective pavement markings at approximately the same time. The pavement marking program began on a few corridors, and each year additional roadways have been added. While it is difficult to attribute all the safety benefits to the sign upgrades alone, there is clearly a connection between the improvements made by the city and the improved safety results.

INSURANCE CORPORATION OF BRITISH COLUMBIA

In 1996, the Insurance Corporation of British Columbia (ICBC) in Vancouver, British Columbia, Canada, began a program to help pay for the costs associated with upgrading traffic control signs from engineer-grade sign sheeting to Diamond Grade™ (ASTM Type IX) sheeting. The program only included safety-related signs, such as regulatory and warning signs. Information and guidance signs are not included in the program. The sign program was applied throughout various locations in British Columbia (Pump 2004).

In addition to the signing upgrades, highly retro-reflective pavement markings were also installed. The combination of the improvements makes a direct comparison difficult based on crash data alone. However, in 1998 the ICBC commissioned a study on the success of the safety program. Since the applications were widespread and the improvement types were not consistent throughout the program, there has not been a detailed study of the before and after crash statistics of the various locations directly related to the sign upgrades.

The study completed by G.D. Hamilton Associates Consulting, Ltd. (Gibbs 1998), estimated the safety benefits of the improvements and found that the program was cost-effective, assuming a single collision was prevented due to the sign upgrades. Table 1 was recreated from the 1998 study.

Table 1. Safety benefit-cost ratio (highly reflective traffic signs)

Type of collision	**Collision claim cost**	**Incremental cost of sign improvement**		**Benefit-cost ratio**	
		1 sign	**4 signs**	**1 sign**	**4 signs**
PDO	$3,400	$31.25	$125.00	>10:1	>10:1
Injury	$35,000			>10:1	>10:1
Fatality	$226,000			>10:1	>10:1

Notes:
1. Average collision claim cost from ICBC
2. Based on incremental cost of replacing existing 75-cm engineering grade stop signs with 75-cm Diamond Grade stop signs
3. Based on the prevention of one collision due to installation of highly reflective signs

In addition, the Hamilton study found the following: “Implementation of highly reflective signs and pavement markings has the potential to reduce night-time collisions by increasing sign conspicuity, sign legibility, and consequently driver perception time. ‘Before and after’ studies performed in the UK of sites at which highly reflective signs were installed showed statistically significant reductions in injury collisions following installation of highly reflective signs, though these studies could not provide a reliable estimate of the percentage reduction in collisions that could be expected” (Gibbs 1998).

When the program began in 1996, the ICBC spent approximately $12,000 on the program. Based on the Hamilton study and positive program results, the ICBC spent $750,000 on the safety program in 2003. The overall safety program also includes intersection and roadway improvements of all types. While specific crash data is not available on the sign-only improvements, the ICBC considers the program a resounding success.

In reviewing the entire road improvement program, a study completed by Sayed, deLeur, and Sawalha (2004) concluded through using claim prediction models that the overall ICBC road improvement investments, including sign upgrades and various other safety remedies, are cost-effective.

MENDOCINO COUNTY, CALIFORNIA

In the 1990s, the Mendocino County Department of Transportation began a program to improve roadway signing and markings on arterial and collector roadways (Ford and Calvert 2002). The program consisted of completing a review of selected roadways; identifying deficiencies such as obsolete or inconsistent signs, poor sign locations, or unmarked hazards; recommending corrective actions; and reviewing the results.

The program included upgrading the signing on county-maintained roads. In the county, 2,400 of the 11,000 total signs were modified through removal, new installations, and/or modifications to the signs. When new signs where installed, high-intensity reflective sheeting was used, whereas before 1993 all signs were installed using engineering-grade reflective material. Crashes were reviewed over three-year cycles before and after the improvements. In addition, two control groups were evaluated to better measure the actual improvements compared to the control group roadways.

Table 2 (Ford and Calvert 2002) illustrates the crash reductions on the roadways improved as part of the safety program, compared to two control groups (roadways not improved as part of the safety program). Improvements to the program roadways included removing unnecessary signs, replacing outdated signs, modifying existing signs, or installing new signs where needed based on the initial safety review.

Table 2. Mendocino County sign upgrade crash experience

Study group	**Change in crashes over six-year study period**
Roadways with improved signing	-42%
Control Group1: no improvements	26%
Control Group 2: no improvements	-3%

The cost to complete the work was approximately $79,300. Based on average crash costs from the State of California of $34,100 and the actual reduction in crashes, it is estimated that the program saved between $12.5 million and $23.7 million with a benefit to cost ratio between 1:159 and 1:299.

Since the original results were so positive, the program was expanded in 1996 to include local streets with some crash history and continues today. Since 2000, some of the signs are being converted to more reflective Diamond Grade™ (ASTM Type IX).

PUTNAM COUNTY, NEW YORK

The Highway Commission of Putnam County, New York, located just north of the New York City metro area, began a traffic sign replacement program in 1993 that replaced more than 2,000 signs from engineer-grade sheeting to a combination of high-intensity sheeting (ASTM Type III) and Diamond Grade™ sheeting. The high-intensity (ASTM Type III) upgrades were used for regulatory and warning signs for roads with recommended speeds of 30 mph and above. Diamond Grade™ (ASTM Type IX) sheeting was installed on arrows and chevrons and warning signs with recommended speeds of 25 mph or less. This action was done based on the premise of driving being a visual action, and typical county roads do not have overhead illumination. A before-and-after study was completed from 1992–1995 (Rogers 1999). Signing improvements were the only significant improvements completed during the study period, according to county representatives. The county saw a 26% reduction in crashes, a 23% reduction in injuries, and a 50% reduction in nighttime crashes. Table 3 illustrates the crash experience (Rogers 1999).

Table 3. Putnam County crash reductions

Year	**1992**	**1995**	**% change**
Fair Street	42	21	-50%
Croton Falls	39	39	0%
Stoneleigh Ave	31	23	-35%
Total	112 Crashes	83 Crashes	-25%

The Croton Falls location did not see a real number decrease in crashes. However, nighttime crashes were reduced by 53% and wet pavement was a contributing factor in 21 of the crashes. Of greater significance is that the type of crashes improved. The improvements were completed on warning and chevron type signs. The crashes that may be related to these types of conditions showed even more dramatic reductions, as shown in Table 4 (Rogers 1999).

Table 4. Contributing factors for crashes 1992 vs. 1995

Contributing factors	1992	1995	Reduction
Failure to yield right-of-way	16	2	88%
Unsafe speed	23	2	91%
Slippery pavement	22	4	82%
Following too closely	8	1	88%
Improper lane usage	4	2	50%
Driver inattention	1	2	-100%
Other	24	2	92%

The improvements took place on locations with crash history. The county invested approximately $160,000 (1999 dollars), which included the cost of all equipment, sheeting, aluminum blanks, and installation. Based on an average cost per crash of $6,400, the county saved $185,000 in its first year of the program. Rogers (1999) said, "In 1995, there were 29 fewer crashes on these roads compared to 1992. The cost savings of these crashes can be estimated at $185,600 ($6,400 x 29.) Monetary savings in the full first year of the improved signage have already surpassed the initial safety investment."

In addition, the results of the program are summarized by the county as the following: "The benefits to the county appear to be phenomenal. By taking an active role in roadway safety, the Putnam County Highway Department offers its customer – the vehicular and pedestrian traffic on county roads – a much safer environment. This improvement will continue to yield positive results as the county population grows and ADTs continue to rise" (Rogers 1999).

Putnam County has continued the program and is currently replacing the signs originally installed with high-intensity (ASTM Type III) sheeting with Diamond Grade™ (ASTM Type IX) signs.

PROGRAM COMPARISON

Since each program was initiated, implemented, and evaluated differently, it is difficult to draw absolute similarities between the different approaches. The fact that the programs were not controlled laboratory studies makes it additionally difficult to eliminate all of the variables that could have impacted the success of each program. However, enough is known to draw comparisons and to look at the different methods used to calculate the benefit to costs ratios to provide a good idea of a range of results that could be anticipated if similar programs are undertaken by other agencies.

The City of Sioux City estimates that it spent $144,925 over three years and had a total savings to the public of $4.92 million. Mendocino County estimated a total cost of $79,300 over six years with a savings to the public of $12.5 million–$23.7 million. Putnam County completed its program in one year with an expense of $160,000 and a savings of $185,000. The ICBC did not track specific costs and benefits, as discussed previously. Table 5 shows the four programs and how they varied in their approach.

Table 5. Program comparison based on local data

Location	Years	Savings*	Costs	B/C ratio
Sioux City	3	$4.92M	$144,925	33.95:1
Putnam County	1	$185,600	$160,000	1.16:1
Mendocino County	6	$12.5M–$23.7M	$79,000	159:1 299:1
ICBC	3	NA	$1.5 M	>10:1

* Savings based on local data provided by each agency

To compensate for the variations in years and cost savings, each program was adjusted to spread the costs and savings over a ten-year period (the approximate life of the signs). The crash savings calculated by each community based on their local data were also adjusted to a similar cost-per-crash basis of $6,400 per crash. Table 6 shows the adjusted costs, savings, and benefit to cost ratios that result.

Table 6. Program comparison using similar cost basis

Location	Years	Savings*	Savings adjusted**	Costs	B/C ratio	B/C ratio adjusted for 10 years**
Sioux City	3	$4.92M	$11.6M	$144,925	33.95:1	267:1
Putnam County	1	$185,600	$185,600	$160,000	1.16:1	11.6:1
Mendocino County	6	$12.5M–$23.7M	$2.4M–$4.5M	$79,000	159:1 299:1	50.6:1 to 94.9:1
ICBC	3	NA	NA	$1.5 M	> 10:1	NA

* Savings based on local data provided by each agency

** Crashes adjusted to a 1997 NSC Estimate of $ 6,400; signs adjusted for ten-year life

When comparing the various programs, it is difficult to eliminate all of the variables. However, each program, when adjusted to constant costs and time periods, shows a benefit to cost ratio of between 11.6 to 1 and 267 to one. The Mendocino County study, which had the only control data to compare, when adjusted showed a benefit to cost ratio of about 50 to 1. Regardless of the application differences between programs, all of the results were positive.

SUMMARY AND CONCLUSIONS

As mentioned, each community is very different and the programs undertaken were also very different. Such factors as location, roadway types, sign series implemented, and available data can vary widely. However, each of the programs has similarities that establish trends and results. Each program was completed at approximately the same time period (1990s) and, perhaps most importantly, each program was deemed successful enough to continue or expand in all cases. Representatives from each area were contacted and all felt the program had achieved positive results that were measurable in various ways.

The programs in Sioux City, IA, Mendocino County, CA, and Putnam County, NY, all showed crash reductions that can be attributed to the sign upgrade program. The ICBC research showed that if a single crash was prevented by upgrading a sign, the program would have a benefit to cost ratio of greater than 10 to 1. Therefore, evidence is clear that completing a sign upgrade from engineer-grade to high-intensity (ASTM Type III) and/or Diamond Grade™ (ASTM Type IX) reflective sheeting will improve safety and have a positive cost to benefit ratio.

As this review shows, there are various ways to implement a sign upgrade program, and each method will have safety benefits to the roadway users. Sign series can be selected, target roadways can be implemented first, spot locations can be identified, or a system-wide upgrade can be completed. In each method, the result of upgrading the signing system using high-performance reflective sheeting is a reduction in crashes. As shown in the Hamilton Associates study (Gibbs 1998), given the relative expense of sign upgrades, even one crash prevented can have a positive benefit to cost ratio of more than 10 to 1.

Identifying what an expected benefit might be is more difficult, due to the amount of variables in each circumstance, limited available data, and the dynamics surrounding each program. However, when comparing the different programs over a similar time frame and using similar crash costs/savings, an expected range can be identified between 11.6 to 1 and 267 to 1. The Mendocino County study was the most controlled application and resulted in an adjusted benefit to cost ratio of between 50 to 1 and 95 to 1.

The initial goal of identifying one expected benefit to cost ratio was not achieved, since there were too many variables to provide a true comparison. Despite the differences in each case, regardless of the implementation process, the results of improving signs from ASTM Type I to Type III and/or Type IX were positive in terms of the actual reduction in crashes and the dollar benefits to the driving public when compared to the costs.

ACKNOWLEDGMENTS

The author would like to acknowledge the following persons for their input and information, which became the basis for this paper: R. Scott Carlson, City of Sioux City, IA; John Pump, Insurance Corporation of British Columbia, Vancouver, B.C.; Stephen Ford, Mendocino County, CA; Mike Druckreier, Putnam County, NY.

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Interstate 235 Reconstruction Update (Invited Presentation)

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ABSTRACT

The reconstruction of I-235 through the Des Moines metropolitan area began in 2002 and is scheduled to run through 2007. This project presents a wide range of challenges that must be addressed in order to complete the project. The greatest challenge, however, was developing a plan on how to rebuild the freeway while maintaining access for the more than 100,000 vehicles that use the roadway each day. Marty Sankey, I-235 Project Manager for the Iowa DOT, will discuss the planning, design, and traffic management plans that have gone into the reconstruction effort. He will present an overview of the project and the progress made to date. Mr. Sankey will also discuss some of the interesting issues that have arisen and how these have been addressed during the construction.

Note: Contact the presenter above for more information on this topic.

Key words: interstate—planning—reconstruction—traffic management

Increasing School Transportation Efficiency Using GIS and the Web

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ABSTRACT

The parents, teachers, administration, and students of Sioux City's schools deserve prompt and reliable service. The safety of the students depends on the Transportation Department's ability to organize all of the aspects involved in transporting students in varying situations. The goal of the Sioux City Transportation Department is to provide the safe, efficient, and reliable transport of the students to and from their school. With the integration of ArcView with the SmartR extension and a database with a web interface, this is possible. The Sioux City Schools in cooperation with the Dakota Valley Schools have developed a methodology to describe, collect, and analyze the data items needed to be tracked for a complete transportation management system. The benefits of using a combination of GIS and web interface will be presented.

Note: Contact the presenters above for more information on this topic.

Key words: ArcView—Lydia—SmartR—transportation

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Fargo-Moorhead Quiet Zone Demonstration Project

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ABSTRACT

The Federal Railroad Administration's (FRA) Final Rule on quiet zones was issued in April 2005. It will allow many cities an opportunity to end the routine sounding of locomotive horns while maintaining public safety. This case study provides local officials with information about technical aspects of the FRA's Final Rule and describes practical implementation actions, based on the results of a national quiet zone demonstration project located in the Fargo, ND-Moorhead, MN, metropolitan area.

The aims of the Fargo-Moorhead Demonstration Quiet Zone project were the following: (1) improve pubic safety along the tracks and (2) foster downtown redevelopment by eliminating train horns. To complete this pilot project, a citizen task force appointed by each city's mayor led a six-year effort that required extensive coordination with the FRA, Burlington Northern Santa Fe (BNSF) Railway, two state departments of transportation, the metropolitan planning organization (MPO), and downtown interest groups. A detailed planning and design process was completed after substantial agency and public input. Additionally, state legislative action was required and the FRA rule-making process was monitored. Further negotiations on multiple issues were necessary to identify safety measures acceptable to various stakeholders. Finally, the application to gain FRA/BNSF approval was completed, and grantsmanship efforts were undertaken to secure the funding necessary to implement the project.

In September 2003, the FRA approved the quiet zone for the Fargo-Moorhead area as a demonstration project. Implementation is currently underway.

Key words: Fargo-Moorhead—quiet zones—railroad—safety

PROBLEM STATEMENT

The need for a Fargo-Moorhead metropolitan quiet zone was generated by two complementary local desires: (1) to provide a positive environment for downtown redevelopment by mitigating the adverse effect of locomotive horns, and (2) alleviate serious pedestrian and vehicle safety problems caused by rail conflicts.

Regarding downtown rail impacts, the Federal Railroad Administration (FRA) data, later adjusted by Burlington Northern Santa Fe (BNSF) Railway (August 2002), indicated rail movements for the northern trackage (Prosper subdivision) averaging 17 trains per day. Rail movements to the southern mainline trackage (KO subdivision) were 50 trains per day, down from 70 trains per day in 1997. Maximum train speeds averaged 20–25 mph on the Prosper line and up to 40 mph on the KO line. Approximately 45% of these metro train movements occurred between 10:00 pm and 7:00 am. Based on field surveys, train horns created noise levels ranging from 57 to 106 decibels (dBA). During public meetings held for the downtown redevelopment planning process, numerous business leaders expressed the need to reduce rail noise as a fundamental action if central business district revitalization were to be successful.

Another important factor supporting the need for a quiet zone was at-grade crossing collision data. FRA collision data reflected a 30-year period from 1975 to January 2005. Both vehicle and pedestrian incidents and the level of severity (injuries, fatalities) were documented for each at-grade crossing in the proposed metro quiet zone. Overall during this period, there were 46 vehicle accidents, 9 pedestrian incidents, 9 serious personal injuries, and 16 fatalities, 10 of which were in the last five years. Clearly, business leaders argued, the use of train horns was not fully protecting public safety, and improved rail crossing measures that also supported downtown revitalization were needed.

RESEARCH OBJECTIVES

To address these significant needs, the mayors of both Fargo and Moorhead appointed a Metropolitan Railroad Issues Task Force (MRITF) and challenged it to attain two objectives: 1) improve public safety along the trackage downtown and 2) foster redevelopment by eliminating train horn sounding in a manner that would have legal standing with the railroad and state (federal cognizant agencies). This was a daunting mission that would required the creation of the longest, most complex quiet zone in the nation, and the only bi-state quiet zone in the United States.

METHODOLOGY

After much planning and evaluation, the MRITF identified the area for the proposed metropolitan bi-state quiet zone. It was to cover the downtown areas of Fargo and Moorhead, cross two states' boundaries, and encompass both BNSF Railway's KO and Prosper mainline tracks. The strategically located quiet zone would stretch almost two miles, which would provide a train horn cessation area of nearly five miles within a three-city area.

According to railroad and federal standards, a quiet zone is defined as "a segment of rail line that has one or more consecutive public/rail crossings where train horns are not routinely sounded because acceptable safety improvements have been installed." The MRITF designed its quiet zone to meet this standard.

The MRITF worked closely with BNSF Railway, the FRA, Federal Highway Administration (FHWA), Minnesota Department of Transportation (Mn/DOT), North Dakota Department of Transportation (NDDOT), and local leaders to prepare a mutually agreeable metropolitan quiet zone proposal. This proposal complied with both the BNSF-Whistle Ban Implementation Criteria Pre-FRA Rules Adoption

(December 13, 2002) and the FRA-Proposed Rule 49 CFR Parts 222 and 229-Use of Locomotive Horns at Highway-Rail Grade Crossing. The proposal used three of the five supplementary safety measures (SSMs) approved by Congress. The quiet zone also included alternative safety measures (ASMs) which were specially designed SSMs that required FRA review and approval. These SSMs and ASMs included three- and four-quadrant gates, medians, and road closures at 20 at-grade crossings in Fargo and Moorhead on both the Prosper and KO BNSF Railway mainline tracks. The program of safety measures were installed on 1 private and 19 public at-grade crossings.

Compliance with Applicable Quiet Zone Standards

The MRITF and the cities understood that to achieve a metropolitan quiet zone certain criteria must be met. The MRITF designed its quiet zone to meet both railroad and federal criteria.

In designing its metropolitan quiet zone, the MRITF worked closely with BNSF Railway representatives to satisfy BNSF Railway criteria. The cities or the MRITF completed all pre-requisites needed to secure BNSF Railway's support, prior to the submission of its quiet zone application to FRA. These specific actions or commitments included the following:

- The MRITF completed four on-site diagnostic item evaluations with BNSF Railway, State DOTs, the FRA, FHWA, city engineers, and task force personnel to select the most appropriate SSMs per site, field inspect the sites, and establish preliminary design and costs for SSMs along both the KO and Prosper trackage.
- The cities installed at all grade crossings the required minimum traffic control devices (flashing lights and automatic gates).
- The communities, with BNSF Railway's advice, mutually selected SSMs that were eligible under the law and FRA's interim rule.
- The communities committed to a continuing program of public awareness and police enforcement of grade crossing violations.
- The communities agreed to reimburse BNSF Railway for all costs associated within the quiet zone implementation.
- The communities secured written approval from NDDOT and Mn/DOT that each DOT would provide a small portion of their federal Section 130 funds to establish the warning devices in the quiet zone, as required by BNSF Railway.
- The communities understood that to obtain horn cessation FRA must approve their quiet zone application and officially notify BNSF that the use of horns at the crossings would cease within the zone.
- The communities further understood that each state DOT needed to approve the warning devices installed at each crossing in the quiet zone.

Furthermore, FRA's interim rule contained certain threshold requirements that also had to be addressed before the federal agency could approve the quiet zone application. The cities and the MRITF met these criteria as well. For example, these included the following:

- The Fargo-Moorhead quiet zone will extend at least one-half mile in length and all at-grade crossings within the zone will be served by approved SSMs or ASMs, even at a private crossing not required to be addressed.
- The mutually selected engineering-based SSMs were designed according to federal standards (MUTCD, 49 CFR 222), and the ASMs were designed with BNSF and FRA input.
- The application for a quiet zone was submitted jointly with the affected railroad, as preferred by the FRA.

Through these efforts and commitments, the approved quiet zone for the Fargo-Moorhead area met or exceeded all FRA and BNSF Railway safety criteria. These safeguards ensured that the quiet zone was effective and that public safety would not be compromised as railroad noise impacts were reduced.

Proposed Metropolitan Quiet Zone

The entire 20-crossing metropolitan quiet zone extends along the BNSF Railway KO line (approximately two miles) from 8th Street in Fargo through each downtown to 14th Street in Moorhead, and along BNSF Railway's Prosper subdivision (approximately two miles) from 7th Street in Fargo to 14th Street in Moorhead. Figure 1 presents the metropolitan quiet zone, locates each at-grade crossing within it, and notes the SSM/ASM to be installed at each crossing. Specific SSM layouts for each of these 20 crossings were then prepared to establish cost estimates. Figure 2 provides examples of the three types of SSMs utilized in the Fargo-Moorhead quiet zone. These SSMs reflected the diagnostic team's assessment and further negotiations and agreement between the cities and BNSF Railway.

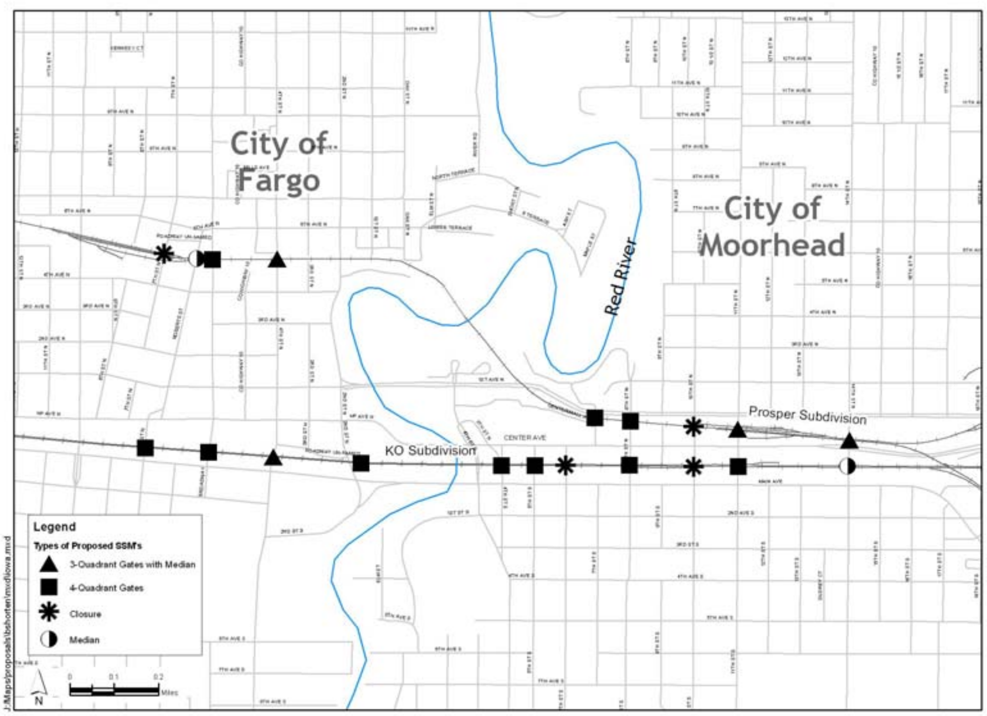

Figure 1. Fargo-Moorhead quiet zone and proposed SSM treatments

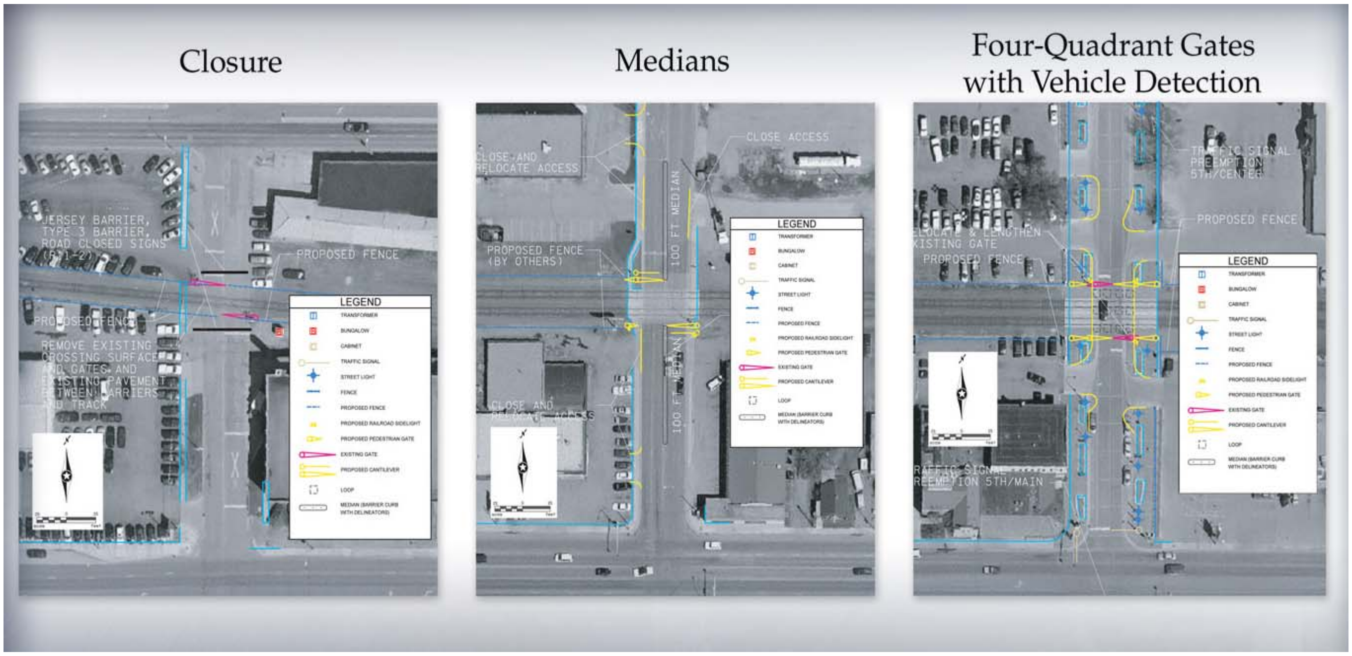

Figure 2. SSMs used in the Fargo-Moorhead quiet zone

RESULTS

After many years of collaboration, the MRITF unanimously approved at its May 2, 2002 meeting a motion to proceed with the metropolitan quiet zone application process. The Fargo-Moorhead Council of Governments then secured consultant assistance in August 2002, and additional public/private meetings were convened to prepare cooperatively the application for FRA review and approval. A draft application was circulated to all interested parties (cities, state and federal agencies, and BNSF Railway) for comment in November 2002. Additional work was then completed, which included responding to comments on the draft and conducting additional field work to prepare detailed SSM layouts. A final draft application was prepared in April 2003.

Prior to submitting the quiet zone application to the FRA, all key partners executed letters documenting their support for the application and indicating their commitment to implement cooperatively the proposed safety measures within the proposed quiet zone. The formal application for the Fargo-Moorhead metropolitan quiet zone was completed and approved by the MRITF and the cities of Moorhead and Fargo in May 2003. In September 2003, the national demonstration project was approved by the FRA.

Concurrently, the MRITF and the cities actively pursued grantsmanship efforts, with over $5 million of various federal funds secured to implement the $7 million project. The progress made in establishing the metropolitan quiet zone has prompted over $2.3 million in private redevelopment investments to be announced in the downtown area, with additional projects anticipated.

Over the past year, the design of the SSMs and ancillary traffic control (detection and preemption) was completed, as required by BNSF. Furthermore, after a year of negotiations, the contract between BNSF and the cities was approved in June 2005, which allows for the manufacture and installation of the SSMs on railroad property. The SSM installation is slated for summer 2006, with the quiet zone officially in operation shortly after that date.

The project serves as a national showcase demonstrating effective rail safety measures and the beneficial impacts of train horn cessation in the revitalization of a central business district.

LESSONS LEARNED

The successful completion of this major metropolitan collaborative effort required the following:

- Strong leadership
- Clear project objectives
- Perseverance and tenacity
- Substantial technical advice and traffic/rail operations knowledge
- Strong partnerships
- Cooperation, negotiation, and compromise
- Congressional support
- Aggressive grantsmanship efforts
- Intergovernmental coordination and landowner support

CONCLUSION

The Fargo-Moorhead Demonstration Quiet Zone project presented an excellent research opportunity for FRA to address methods and processes prior to preparing its Final Rule on quiet zones. The FRA's Final Rule was issued on April 22, 2005 and will take effect on June 24, 2005. This rule will allow communities to establish quiet zones with less effort, at lower costs, and with less influence from railroads. Furthermore, future quiet zone approvals will be based on risk-reduction statistical data, unlike the Fargo-Moorhead Demonstration Quiet Zone project, which was largely determined through multi-agency negotiations and qualitative assessment.

ACKNOWLEDGMENTS

The Fargo-Moorhead area quiet zone project was an inclusive process. Over the six-year planning effort, representatives from BNSF Railway, the FRA, FHWA, Mn/DOT, NDDOT, MRITF, the cities of Fargo and Moorhead, and the Fargo-Moorhead Council of Governments were invited to participate, and all were actively involved in the detailed analysis required to advance this complex project.

Roughness Progression on Superpave Pavements

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ABSTRACT

Pavement smoothness is a major factor affecting performance. It is believed that the service life of a pavement is directly related to its as-constructed smoothness. Since the introduction of the Superpave system in Kansas, bonus payment has increased significantly, indicating that these pavements are smoother initially. However, whether the roughness progression has slowed down is not known yet. This paper presents the analysis results of short-term roughness progression on Superpave sections in Kansas. Seventeen test sections built between 1999 and 2001 were selected for the study. These pavement sections were constructed over different subgrade and base types, and different PG binders were used in the asphalt layers. Annual roughness data was collected from the Kansas Department of Transportation's Pavement Management Information System database. International Roughness Index (IRI) was used as the roughness statistic. Statistical analysis results show that the type of project—reconstruction or rehabilitation—has a significant effect on short-term roughness. It was found that the short-term roughness of the Superpave sections with aggregate base was significantly lower than those with other base types. The analysis results also show that future roughness is a function of initial roughness. This indicates that a smooth Superpave pavement will remain smoother over time.

Key words: pavement—smoothness—superpave

INTRODUCTION

Pavement smoothness is probably the single most important factor of performance from the standpoint of traveling public. Road user surveys identify pavement smoothness as the most important measure of pavement quality (NQI 1996; Keever et al. 2001). Studies at the road test performed by the American Association of State Highway Officials (AASHO) showed that the subjective evaluation of pavement, based on mean panel ratings, was also primarily influenced by roughness (Hass et al. 1994). Smoothness plays a significant role in the construction, functionality, and performance of roadways (Wolters and Grogg 2002). Many state agencies have adopted specifications that require minimum levels of smoothness for newly built pavements, with some specifications incorporating significant incentive/disincentives as additional encouragement for attaining specified smoothness levels. There are many factors that contribute to the roughness (or lack of smoothness) of a pavement surface. The most common cause of roughness is pavement distress. Common hot-mix asphalt pavement distresses that contribute to the roughness are fatigue cracking, deteriorated transverse cracks, corrugations, and shoving. Over time, swelling soils or frost heave can also contribute to the pavement roughness. Roughness can also be "built in" during construction because of several factors, such as, excessive mixture segregation, variability in the base and subgrade, poor grade control, inconsistency in paving operations, construction joints, and random construction deviations (Wolters and Grogg 2002).

PROBLEM STATEMENT

In 1985, the Kansas Department of Transportation (KDOT) selected a 25 ft California-type profilograph and 0.2 in blanking band for evaluation of profilograms for determining as-constructed smoothness of Portland cement concrete pavements (Hossain and Parcells 1994). By 1990, KDOT was successful in controlling concrete pavement smoothness. This success led to the development of profilograph-based specifications for asphalt concrete (AC) pavements in 1990. Profilograph results for ensuring smoothness of bituminous pavement with higher than four-inch paving depth was implemented through Special Provision 90P-39 (Hossain and Parcells 1994). In 2000, 38 percent of the sections were in bonus range of 0 to 10 inch/mile limit, which was better than any previous year. During 1990 to 1999, 98 to 99 percent of asphalt pavement sections were in the bonus or full pay range of smoothness (Parcells 2001).

Many in the asphalt pavement industry believe that initial pavement smoothness is directly related to the pavement service life. Janoff (1985) studied the relationship between initial roughness and roughness after 10 years of service. The results indicated that approximately 110 percent of initial roughness was present after 10 years of service. Raymond (2000) examined the effect of initial smoothness on long-term roughness progression on asphalt overlays placed over existing asphalt pavements using roughness data from SPS-5, GPS-6, and Canadian Long Term Pavement Performance (C-LTPP) sites. For the C-LTPP sites, 68 percent of the initial roughness remained after eight years of service. These values were 57, 85, and 84 percent for the Long Term Pavement Performance (LTPP), SPS-5, GPS-6, and combination of these three sites, respectively. Perera and Kohn (2001) examined the smoothness of the LTTP SPS-1 (strategic study of structural factors for flexible pavements) test sections in Kansas constructed in 1993. They found that the average as-built IRI of 12 test sections in Kansas was 51 in/mile. However, with time significant smoothness loss occurred for most of these sections. The roughness of some sections increased by 100 in/mile over 5-year time period.

Studies on roughness progression of Superpave pavements are almost nonexistent. After the introduction of Superpave pavements in Kansas, smoothness bonus payment has increased significantly, indicating that Superpave pavements are smoother initially. But the rate of roughness progression is yet to be determined.

OBJECTIVE

The objective of this study was to evaluate the roughness progression of Superpave pavements in Kansas.

TEST SECTIONS

Seventeen sections, built between 1998 and 2002, were selected in this study. These projects are all major modification projects—they were reconstructed or rehabilitated with intensive surface preparation. There were eleven reconstruction and six rehabilitation projects. Tables 1 and 2 show the locations and layer thickness data of these projects, respectively. The project lengths are variable and so are the layer thicknesses. Some of these projects had aggregate base and some had asphalt concrete base. Eleven projects were built over lime-treated subgrade, two over compacted soil, and the rest were over cold-in-place-recycled (CIPR) asphalt base. Most sections are two-lane undivided highways with 8 to10 ft wide shoulders. The K-254 sections are 4-lane divided highways.

Table 1. Features of study sections

KDOT project no.	Route	County	Project length (mile)	Work performed	Construction year
169-1-K-4419-02	US-169	Allen	8.41	Reconstruction	1999
169-2-K-4420-02	US-169	Anderson	4.24	Reconstruction	1999
57-2-K-4421-02	K-57	Anderson	2.19	Reconstruction	1999
254-08-K-5060-02	K-254 (NB)	Butler	4.65	Reconstruction	1998
	K-254 (SB)		4.65	Rehabilitation	1998
70-27-K-5982-01	I-70	Ellsworth	16.9	Reconstruction	2000
50-38-K-5743-01	US-50	Hamilton	12.40	Reconstruction	2001
50-47-K-5744-01	US-47	Kearney	14.96	Rehabilitation	2000
83-55-K-5388-01	US-83	Logan	14.92	Reconstruction	1999
61-59-K-5386-01	K-61	McPherson	2.22	Reconstruction	1999
81B-59-5386-02	US-81B	McPherson	2.50	Reconstruction	1999
27-65-K-5382-01	K-27	Morton	14.39	Rehabilitation	1999
169-67-K-5387-02	US-169	Neosho	6.81	Reconstruction	1999
283-68-K-5391-01	US-283	Ness	16.54	Rehabilitation	1999
281-76-K-5390-01	US-169	Pratt	6.43	Rehabilitation	1998
254-87-K-5060-02	K-254 (NB)	Sedgwick	7.31	Reconstruction	1998
	K-254 (SB)		7.31	Rehabilitation	1998
183-82-K-5751-01	US-183	Rooks	2.78	Rehabilitation	2000
36-101-K-5383-01	US-36	Washington	9.18	Rehabilitation	1999

Table 2. Layer type and thickness of the study sections

Project no.	Subgrade		Base		Binder course		Surface course	
	Type	Thick-ness (in)	Type	Thick-ness (in)	Binder	Thick-ness (in)	Binder	Thick-ness (in)
169-1-K-4419-02	LT	6	AC	6	SM-2C (PG64-28)	4	SM-1T (PG64-28)	1
169-2-K-4420-02	LT	6	AC	6	SM-2C (PG64-28)	4	SM-1T (PG64-28)	1
57-2-K-4421-02	LT	6			SM-2C (PG64-28)	5.5	SM-1T (PG64-28)	1
254-08-K-5060-02	LT	6	AB+UDB	7 + 6	SM-2C (PG58-28)	6.5	SM-1T (PG70-28)	1
254-08-K-5060-02	LT	6	CIPR	4	SM-2C (PG58-28)	8	SM-1T (PG70-28)	1
70-27-K-5982-01	COM	18	AC	4	SR-2C (PG58-34)	2	SM-1T (PG64-28)	1
50-38-K-5743-01	FA	6	AC	6	SM-19B (PG70-28)	2.5	SM-9.5T (PG70-28)	1.5
50-47-K-5744-01	COM	18	AC	10	SM-19B (PG70-28)	2.5	SM-9.5T (PG70-28)	1.5
83-55-K-5388-01	COM	18	CIPR	4	SM-2C (PG58-28)	9	SM-1T (PG58-28)	1
61-59-K-5386-01	LT	6	AB+UDB	11 + 6.5	SR-2C (PG58-28)	5.5	SM-1T (PG64-28)	1
81B-59-5386-02	LT	6	AB+UDB	11 + 6.5	SR-2C (PG58-34)	5.5	SM-1T (PG64-28)	1
27-65-K-5382-01	COM	18	CIPR	3	SM-2C (PG58-28)	6.5	SM-1T (PG70-28)	1.5
169-67-K-5387-02	LT	6	AC	8	SM-2C (PG58-28)	4	SM-1T (PG64-28)	1
283-68-K-5391-01	COM	18	CIPR	4.5	SR-2C (PG58-34)	6.5	SM-2A (PG58-28)	1.5
281-76-K-5390-01	COM	18	CIPR	4	SM-2C (PG58-28)	5	SM-1T (PG64-28)	1
254-87-K-5060-02	LT	6	AB+UDB	7 + 6	SM-2C (PG58-28)	4	SM-1T (PG70-28)	1
254-87-K-5060-02	LT	6	CIPR	4	SM-2C (PG58-28)	7	SM-1T (PG70-28)	1
183-82-K-5751-01	COM	18	CIPR	4	SR-2C (PG-64-28)	4	SM-1T (PG64-28)	1
36-101-K-5383-01	LT	6	AB	13	SR-2C (PG58-28)	-	SM-1T (PG58-28)	1

LT: Lime-treated
COM: Compaction type - 95% of standard density and moisture range: optimum ± 5%
FA: Fly-ash
CIPR: Cold-in-place-recycled asphalt
AC: Asphalt concrete
AB: Aggregate base
UDB: Unbound-drainable-base

DATA COLLECTION

Data collected for this study can be divided into two categories: layer property data and profile or roughness data.

Layer Property Data

This category includes properties of subgrade soil, as well as characteristics of the Superpave mixtures for the surface and binder layers.

Subgrade Data

Subgrade data was collected from the design files. Table 3 shows subgrade soil properties of the study sections. These properties include optimum moisture content, dry density, plasticity index, and percent of soil passing No. 200 sieve. Subgrade soils on most sections were modified with lime. On other sections, subgarde soils were either modified with fly ash or just compacted. According to the Unified Soil Classification system, soil type for most sections is CL or CL-ML. More than 81 percent subgrade materials passed through No. 200 sieve. Plasticity index values varied from 8 to 35 percent. The range of optimum moisture content was 13 to 23 percent. Dry density of the subgrade soil exceeded 90 lb/ft^3 for all sections.

Table 3. Subgrade soil properties

Project No.	Unified Soil Classification	Dry Density (lb/ft^3)	Optimum Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Subgrade Materials Passing US No. 200 Sieve (%)
169-1-K-4419-02	ML-CL	99	21	45	24	91
169-2-K-4420-02	ML-CL	97	18	36	10	93
57-2-K-4421-02	ML-CL	97	18	38	10	93
254-08-K-5060-02	CH	92	23	55	31	99
70-27-K-5982-01	CL	100	19	49	25	88
50-38-K-5743-01	ML-CL	100	21	36	13	96
50-47-K-5744-01	CL-CL	103	21	42	18	89
83-55-K-5388-01	ML-CL	101	19	39	17	99
61-59-K-5386-01	CL	97	18	35	11	98
81B-59-5386-02	CL	103	22	38	18	95
27-65-K-5382-01	CL	102	20	35	13	91
169-67-K-5387-02	CL	99	22	44	19	89
283-68-K-5391-01	CL	103	19	45	24	85
281-76-K-5390-01	SC	115	13	23	8	85
254-87-K-5060-02	CL	99	19	38	17	96
183-82-K-5751-01	CL	N/A	N/A	36	16	81
36-101-K-5383-01	MH	N/A	N/A	60	35	88

Superpave Mixture Data

Table 4 shows the mixture properties of the binder and surface courses for the study sections. Asphalt content of the mixtures varied from 4.8 to 6.0 percent. Air voids for all projects met KDOT specification of 4±2 percent. However, the VMA values are different due to varying nominal maximum aggregate size of the mixtures. The VMA values ranged from 12 to 16.4 percent. VFA's were very close to 70 percent required for most projects. Fine aggregate angularity and sand equivalent values were similar for all mixtures.

Table 4. Superpave mixture properties

Section	Asphalt content (%)		Air voids (%)		VMA (%)		VFA (%)		Aggregate passing #200 sieve (%)		Fine aggregate angularity		Sand equivalent		Gmm	
	Surf*	Base	Surf	Base	Surf	Base	Surf	Base	Surf	Base	Surf	Base	Surf	Base	Surf	Base
US-169 (1)	4.6	4.6	3.4	3.8	13.8	13.8	75.4	72.5	4.7	5.1	44	44	80	79	2.466	2.448
US-169 (2)	5	4.8	3.8	3.9	13.6	13.4	72.1	70.9	4.8	5.5	44	44	80	78	2.449	2.392
K-57	5.6	5	4.0	4	14.7	13.6	72.8	70.6	4.5	4.9	44	44	78	78	2.433	2.457
K-254 (1)	6	5.2	4.7	4.2	15.3	13.7	69.3	69.3	3.5	4.2	44	43	83	75	2.409	2.434
I-70	6.3	5.3	4.3	4.4	15.0	13.6	70.6	67.6	4.8	4.9	46	44	79	78		2.374
US-50 (1)	5.1	4.9	3.9	3.5	15.1	13.2	74.2	73.5	4.1	3.6	47	44	88	86	2.432	2.442
US-50 (2)	5	4.8	4.2	4.3	15.1	12.8	72.2	66.4	4.2	3.4	46	44	78	86	2.426	2.401
US-83	6.1	4.7	4	3.7	16.2	13.9	75	73.4	4.2	4	42	48	92	80	2.489	2.437
K-61	5.9	5	5.9	4.2	15.8	13.3	62.7	68.4	4.1	4.7	42	43	77	88	2.388	2.403
US-81B	6.1	5	5.2	4	15.6	13.1	66.7	69.5	3.8	5.3	41	44	65	89	2.397	2.415
K-27	5.9	5.8	5	3.3	15.3	14.2	68.7	76.7	4.8	4.9	48	46	67	69	2.535	2.397
US-169 (3)	6.2	3.7	3.9	4.3	15.6	13.5	75	68.1	3.6	3.6	45	47	93	95	2.41	2.429
US-283	5.4	4.4	4.6	4.3	15.8	13.7	70.9	68.6	3.4	3.3	43	42	92	90	2.442	2.481
US-281	5.4	4.9	4.6	4.4	15.8	13.9	70.9	68.3	4.6	3	43	43	87	88	2.376	2.404
K-254 (2)	5.6	5.3	4.5	4.3	14.9	14.1	69.8	69.5	4.3	4.5	44	44	79	78	2.394	2.405
US-183	5.48	5.1	4.45	3.7	15.08	13.4	70.5	72.2	5.8	4.1	43	43	99	84	2.471	2.464
US-136	5.29	5.4	4.81	4.2	15.72	13.3	69.4	68.4	4.6	4.9	42	42	76	76	2.367	2.364

* Surface layer

Roughness Data

Annual roughness data in terms of International Roughness Index (IRI) were collected from the KDOT Network Optimization System (NOS) survey results stored in the Pavement Management Information System (PMIS) database (KDOT 2003). For each section, three to five years of roughness data was available from the PMIS database. Profile data in the NOS survey is collected with a South-Dakota-type high-speed inertial profiler equipped with laser sensors. Profile measurements were done once on both left and right wheel paths.

As-constructed IRI value represents the average IRI of the left and right wheel path measurements. Most sections in this study were built with low initial roughness. The range of the as-constructed IRI was 32 to 76 in/mi with an average of 44 in/mi. All sections fell under Roughness Level I of KDOT NOS. US-254 section in Butler County, a reconstruction project, had the lowest IRI value. On the other hand, US-281 section in Republic County, which was built over compacted natural subgrade, had the highest IRI value.

Figure 1 presents the typical pattern of roughness progression. A definite pattern of roughness progression is evident—roughness increased with time.

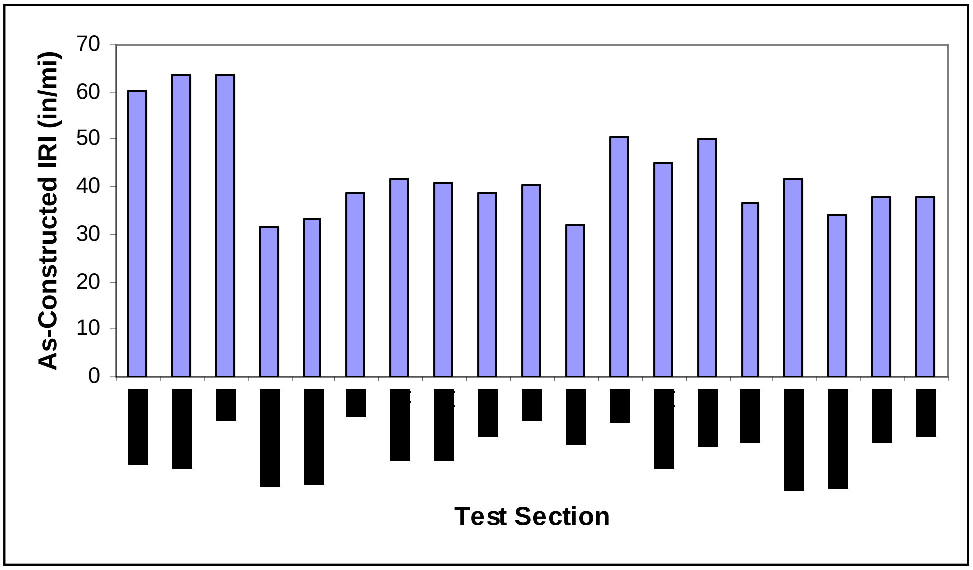

Figure 1. As-constructed IRI values for the study sections

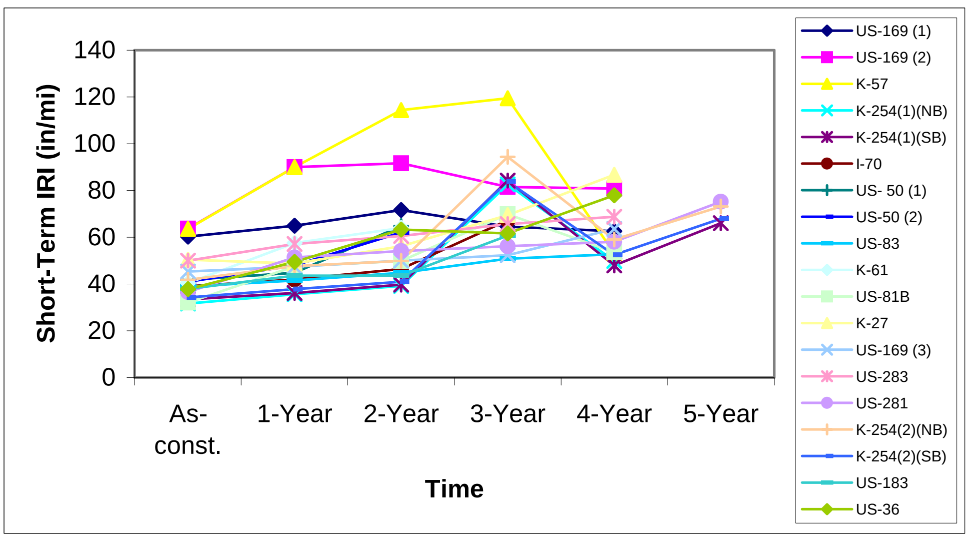

Figure 2. Roughness progression on the study sections

STATISTICAL ANALYSIS

Analysis of Variance (ANOVA)

Analysis of Variance (ANOVA) was performed to examine the effect of different factors on short-term roughness of Superpave pavement sections using the SAS software (SAS 1979). ANOVA tests the difference between two or more groups of population. The process compares the variability that is observed between two conditions to the variability observed within each condition. When the variability that can be predicted (between the two groups) is much greater than the variability that cannot be predicted (within each group), it can be concluded that those groups of population means are significantly different from each other. The response variable for this analysis was IRI. There were several treatment variables: (a) work type (reconstruction or rehabilitation); (b) profile age (as-constructed, 1, 2, 3, 4, and 5 years); (c) base type (aggregate, AB; asphalt, AC; and CIPR); (d) base thickness (A = thickness < 4 in; B= 4 in ≤ thickness≤ 6 in; and C = thickness >6 in); (e) subgrade type (Lime-treated and others); and (d) PG binder type (PG 58-28, PG 64-28, and PG 70-28).

The statistical model for the experiment is given by the following equation:

$$IRI_{ijklmn} = WORK_i + AGE_j + SG_k + BASE_l + BCTHIK_m + PG_n + Interactions + \varepsilon_{ijklmn} \quad (1)$$

Where IRI_{ijklmn} is the International Roughness Index (in/mile);
$WORK_i$ is the ith work type effect;
AGE_j is the jth profile age effect;
SG_k is the kth subgrade type effect;
$BASE_l$ is the lth base type effect;
$BCTHIK_m$ is the mth base course thickness effect; and
PG_n is the nth PG binder type effect.

All conclusions were made at 95% confidence level. The means of the response variable (IRI) at different levels of a factor were compared by the least square means (LSMean) approach (Milliken and Johnson 1984). This technique weighs the estimates of each treatment or treatment combination effect equally, but not each observation. The LSMean model deals with the average of individual treatment measurements and treatment combination; it gives unequal weight to each observation.

ANOVA Results

ANOVA results show that the project type (reconstruction or rehabilitation), profile age, base type, and base thickness have significant effects on the mean IRI values at 95% confidence level. Table 5 shows the pair wise comparison of LSMean IRI values for different factors. On average, the reconstruction projects showed higher IRI values than the rehabilitation projects. Deep milling and/or cold-in-placing recycling appear to be beneficial in achieving smoother Superpave pavements.

The age of the pavement has a significant effect on the mean IRI values. The change in mean IRI is most significant during the first year. There is no significant difference in mean IRI values between the first and the second years. It is to be noted that for a number of projects, the mean IRI value decreased after four years. This happened due to some preventive maintenance activities, such as slurry sealing, etc., on those projects.

The lime-treated subgrade results in smoother pavements; although, the difference in mean IRI values for the lime-treated subgrade and other subgrade types (compacted natural and fly ash treated) is not statistically significant.

As mentioned earlier, the effect of base type on Superpave pavement smoothness is significant. The projects with aggregate base showed much lower IRI than those on asphalt bases. The projects with CIPR base also showed the same trend. Projects with thinner base appeared to outperform those with thicker base in terms of smoothness.

Table 5. Comparisons of IRI values for different factors

Levels of Factor	LSMean of IRI (in/mi)	Pair-wise Comparison
Factor: Work Type*		
Reconstruction	60	> Rehabilitation
Rehabilitation	54.1	
Factor: Time*		
As-constructed	40.6	< All other time periods
1-year	50.1	> As-constructed; < 3, 4, 5-year
2-year	52.8	> As-constructed; < 3, 5-year
3-year	67.0	> As-constructed, 1, 2-year
4-year	60.9	> As-constructed, 1-year
5-year	71.0	> As-constructed, 1, 2-year
Factor: Subgrade		
Lime-treated	54.0	
Others	60.1	
Factor: Base Type*		
Aggregate (AB)	51.0	< AC
Asphalt (AC)	68.9	> AB; > CIPR
CIPR	51.3	> AC
Factor: Base Thickness*		
A (< 4 in)	44.7	< B; < C
B (4 –6 in.)	64.8	>A
C (> 6 in.)	61.6	> A
Factor: PG Grade		
PG 70-28	59.5	
PG 64-28	53.5	
PG 58-28	58.1	

* Significant at 95% confidence level

CONCLUSIONS

Analysis of as-constructed and short-term roughness of the Superpave sections in Kansas is presented. Some of the key findings of this study are as follows:

- Superpave sections built over asphalt bases have significantly higher IRI values than those over aggregate or CIPR bases.
- As-constructed roughness significantly affects the short-term roughness of a pavement section. The smoother the Superpave pavement is built, the smoother it will remain over time.
- Rehabilitation projects tend to be smoother than reconstruction projects.
- Overall, the lime-treated subgrade results in smoother pavements.

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Temperature and Curling Measurements on Concrete Pavement

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ABSTRACT

Curling generally results from the temperature differential across the concrete slab thickness. Curling induces stresses in the pavement slab that may contribute to early-age concrete cracking. This study deals with the field measurement of temperature and curling on a newly built jointed plain concrete pavement. The pavement section consisted of a 12-inch concrete slab, 4-inch bound drainable base, and 6-inch lime-treated subgrade. Temperature data was collected at five different depth locations across the thickness of the concrete slab with the digital data loggers embedded in the slabs. Curling was measured on five different days in the summer and fall with a simple setup. The results show that both upward and downward curling increase as the temperature differential increases. The magnitude of curling deflection resulting from a particular positive temperature differential is slightly higher than that resulting from the same negative temperature differential value. The in situ curling can be simulated with a properly built finite element model. Since temperature differential has a significant influence on curling, the effect of curling can be mitigated at an early age of pavement concrete with proper measures, such as enhanced curing.

Key words: concrete—curling—pavement

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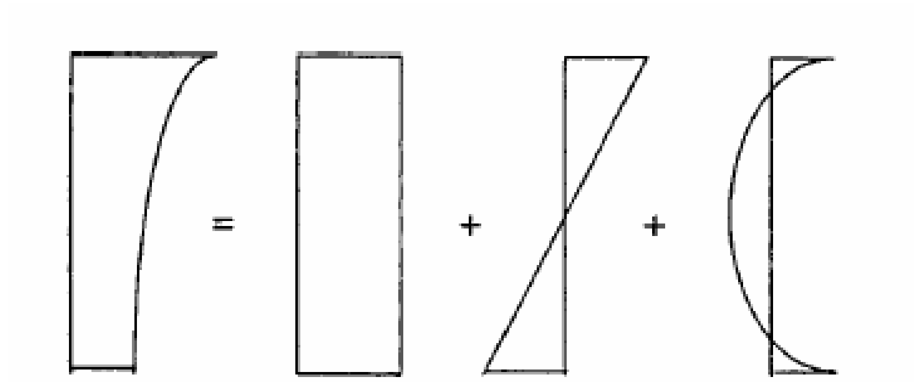
INTRODUCTION

Temperature is an important environmental factor that influences the performance of concrete pavements. Curling, which results from the temperature gradient between the concrete pavement top and bottom surfaces, induces stresses in the pavement, since the pavement is restrained by its weight. The thermally induced stress caused by such interaction may result in early pavement cracking (Tang et al. 1993). Curling also results in a loss of support along the slab edges or at the slab interior. The effect of the loss of support results in higher stresses as the subbase becomes stiffer. This may become critical, particularly within a few hours after slab placement, since concrete at the early stage of hydration may not have sufficient strength to prevent cracking. Temperature increase caused by hydration does not immediately produce thermal stresses because of the process of stress relaxation or creep in the concrete (Emborg 1989). Thermal stresses arise when the temperature drops after its peak value and the concrete has set. The temperature gradient that causes the slab to curl is affected by ambient temperature and solar radiation. Even if cracks do not develop, the temperature gradient can cause curling and permanent set of the concrete slabs in a non-planar fashion, a phenomenon known as as-built curling. The temperature differential between the slab top and bottom surfaces of a concrete pavement is also related to the fatigue damage (Masad et al. 1996). Fatigue damage caused by truck traffic can be 10 times higher for a 1°F/in.-temperature differential compared to that for a zero temperature gradient.

CONCRETE PAVEMENT TEMPERATURE AND CURLING

The concrete pavement temperature's effect on curling has been studied since the 1920s. Westergaard (1926; 1927), in his analysis of concrete pavement curling stresses, assumed the temperature distribution through the thickness of the slab to be linear. In the early 1930s, nonlinear temperature gradients were obtained from the results of the Arlington road tests (Teller and Sutherland 1936). The nonlinearity in the temperature distribution has been proven by experimental data presented by several investigators (Richardson and Armaghani 1987; Armaghani et al. 1987; Thompson et al. 1987; Choubane and Tia 1995). Typically, the nonlinear profile is featured with a relatively rapid change of temperature within the top quarter of the slab thickness, followed by a more gradual change toward the bottom face (Yoder and Witczak 1975). Researchers have also represented temperature profile by a quadratic equation or by a third-order polynomial (Choubane and Tia 1992; Zhang et al. 2003). Following the classical theory of plates, the nonlinear temperature distribution can be divided into three parts based on their respective effects on plate deformation: a) a part with uniform temperature that causes the pavement slab to expand or contract uniformly across its cross section, b) a linear part that causes bending of the pavement slab, and c) a nonlinear part that remains after the linear and uniform temperature parts have been subtracted from the total temperature distribution, as shown in Figure 1 (Choubane and Tia 1992). Some other regression-based and theory-based equations have also been developed to estimate effective temperature differential between pavement top and bottom (Kuo 1998; HIPERPAV 2003).

The implications of nonlinear temperature distribution across the concrete pavement slab have been studied by Choubane and Tia (1992) and Zhang et al. (2003). Choubane and Tia showed that the assumption of a linear temperature profile could lead to errors of 30% or more in the computed peak warping stresses (Choubane and Tia 1992). In their study, Zhang et al. (2003) reported that assuming a linear temperature distribution overestimated tensile stresses during certain periods of the day, while underestimated them during other periods. Differences between the peak tensile stresses of the linear and nonlinear analyses reached as high as 75%.



Nonlinear temperature distribution

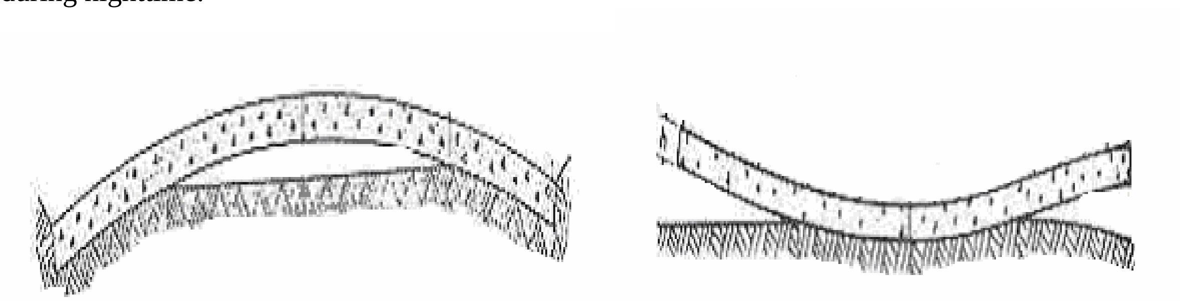
Component causing axial expansion or contraction

Linear component causing bending

Nonlinear component

Figure 1. Temperature distribution across the concrete slab

As mentioned earlier, the temperature gradient may result in as-built curling. Studies have shown that pavement slabs are not necessarily flat at a zero temperature gradient, i.e. these slabs are built with significant curling already in them. The factors that cause built-in curling in jointed plain concrete pavement (JPCP) slabs include temperature gradient at the time of concrete hardening and differential shrinkage (Beckemeyer 2002). The differential shrinkage causes pavement slabs to curl upward, as shown in Figure 2. Because pavements are typically constructed during the daytime, the temperature gradients during construction also tend to cause upward curling. Yu et al. (1998) have shown that on average, the magnitude of built-in curling is about 1°F/in. for the pavements in wet-freeze climates. The combination of built-in curling and actual temperature gradients can cause significant upward curling of the slabs during nighttime.



(a) Downward curling **(b) Upward curling**

Figure 2. Curling of concrete slab due to temperature gradient

TEMPERATURE AND CURLING MEASUREMENTS

Test Section

The test section for temperature and curling measurements was a JPCP section on Interstate 70, constructed in the summer of 2003. The section has 16.4-ft joint spacing with 1.5-inch–diameter dowels.

The pavement cross section consists of a 12-inch concrete slab, 4-inch portland cement-treated subbase (known as bound drainable base, BDB) and 6-inch lime-treated subgrade. The concrete mixture was composed of 40% coarse and 60% fine aggregates with a water-cement ratio of 0.45. The entrained air was 5.8%. Average 28-day core compressive strength and 3-day modulus of rupture were 5,220 psi and 575 psi, respectively. The section was cured with a curing compound.

Temperature Measurement

Temperature data was collected by a digital data logger iButton® (iButton 2003). iButton® is a computer chip enclosed in a 0.63-inch stainless steel can. Because of this unique and durable stainless steel can, up-to-date information can be obtained readily. The steel button can be mounted anywhere because it is rugged enough to withstand harsh environments, indoors or outdoors. Information from the iButton® can be retrieved with a computer at any time intervals. They are also very cheap and easy to install.

Figure 3 shows the layout of iButton® placement for the study. The assembly was installed near the right wheel path, which is about three feet away from the edge of the driving lane. Temperature data was collected at five different depths along the slab thickness: the top surface; three inches, six inches, and nine inches below the top surface; and the bottom surface. Data was collected at 10-minute intervals. The advantage of using five buttons is that they capture the actual temperature distribution across the slab thickness.

Figure 3. Digital data logger placement

Figure 4 presents a typical hourly pavement temperature distribution curve. It is apparent from the figure that the temperature distribution along the thickness of the slab is nonlinear. The nonlinearities of temperature distribution along pavement thickness for the bottom half of the slab are not as pronounced as they are for the top half. For the bottom half, the distribution is almost linear. For the top half, the distribution for the hours of positive temperature differential (i.e., the temperature of the top surface is higher than that of the bottom) is steeper than that for the hours of negative temperature differentials (i.e., the temperature of the bottom is higher than that of the top). The hourly temperature variation at the bottom of the slab is not much: the difference between maximum and minimum temperature is about 7°F for this particular case. However, this difference is more pronounced for the top surface (about 32°F).

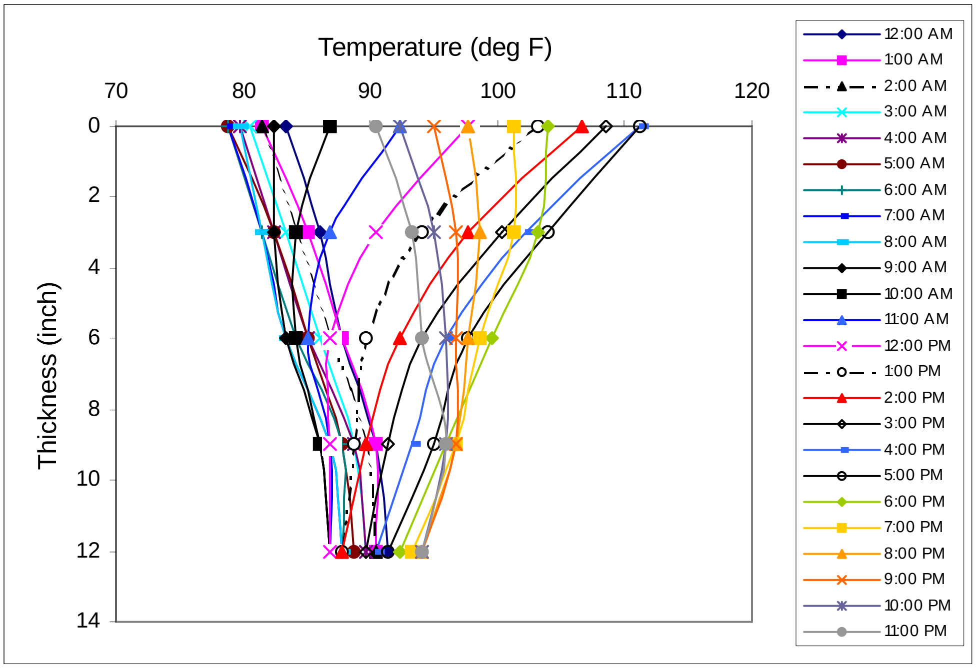


Figure 4. Typical temperature variation across the slab thickness

Curling Measurement

Curling on the section was measured by a simple setup, shown in Figure 5. It consists of an extensometer, which is mounted at the center of a lightweight aluminum frame. The length of the frame is approximately 16.4 ft, which represents the length of the concrete slab. The frame is positioned on steel pins attached to the bottom of the frame. These pins ensure correct and repeatable positioning and serve to form a reference level. The pavement surface moves vertically with time because of the temperature differential between the pavement top and bottom surfaces. The extensometer, which is in contact with the top surface of the pavement, moves with the vertical movement of the concrete slab, thus measuring both upward and downward movements. This measurement represents the curling or mid-slab deflection of the pavement slab with respect to the reference plane established by the pin.

Curling was measured at different locations of the Maple Hill section: the left and right wheel paths, and the center of the slab. These locations were selected randomly and were located on both driving and passing lanes. Data was collected on five different days: August 7, September 2, September 25, October 19, and November 6 of 2003. The first set of data was taken approximately one week after construction. Other measurements were done at approximately three-week intervals. In Kansas, during the summer when the first measurement was done, the ambient temperature was very high. On the other hand, during November when the last set of measurements was made, the weather was cooler, and the ambient temperature, as well as the pavement temperature, was relatively low compared to that of the summer. For a particular day, hourly curling measurements were taken throughout the day. No nighttime measurements were done.

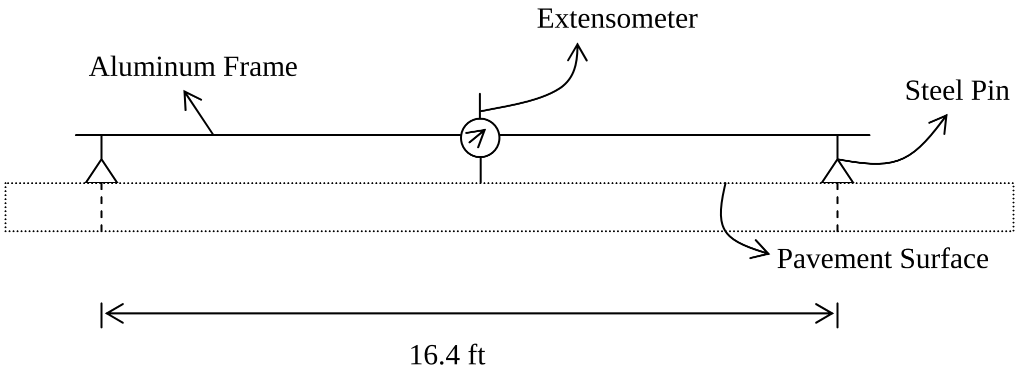
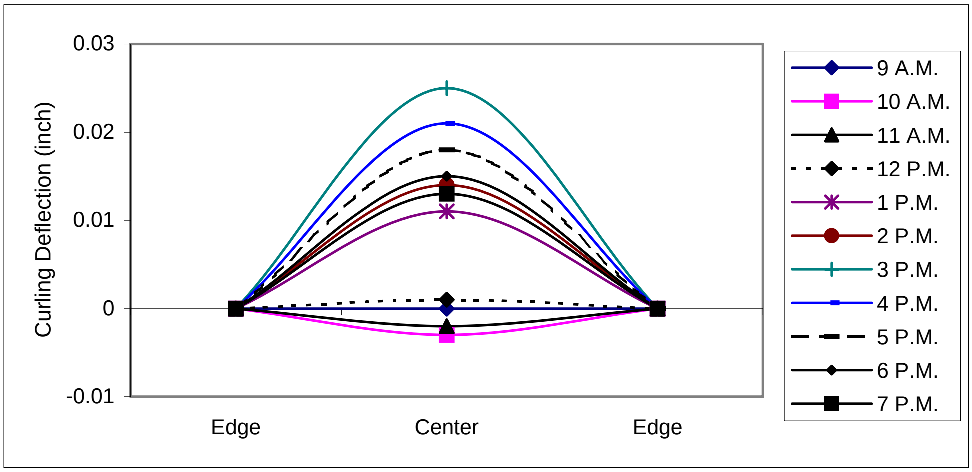


Figure 5. Curling measurement setup

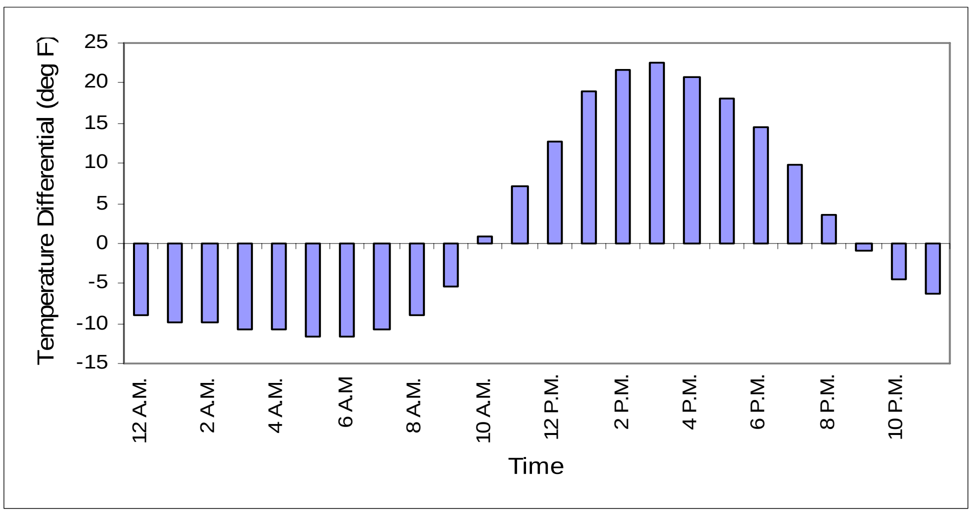
DATA ANALYSIS

Figures 6 (a) and (b) show the measured curling and temperature differentials between the pavement top and bottom on August 7, 2003 for a selected location. On that day, the first measurement was made at 8:00 am and hourly measurements were taken throughout the day. It is to be noted that the curling or mid-slab deflections shown in this figure do not represent the actual curling; rather it is relative to the first measurement. Hence, the actual curling of the pavement may be higher or lower depending on the time of measurement. The figure shows that in the morning, the pavement section was curled upward. At around noon, the opposite scenario (downward curling) was observed. Figure 6 (b) indicates that in 13 out of 24 hours this section experienced negative temperature differentials. The temperature at the top exceeded the temperature of the bottom at around 10 am, and the opposite scenario occurred at 9:00 pm. For the first few hours, upward curling was observed and at around noon; when the pavement first experienced a positive temperature gradient, downward curling was noticed. The maximum downward curling was observed at around 3:00 pm, when the temperature differential was the highest. After this time, downward curling started to decrease. Although nighttime measurements were not taken, it can be assumed that at some point in the evening (at around 9:00 pm in this case), when the bottom temperature exceeded the top temperature, the pavement experienced upward curling again. If there is no difference in moisture content between the pavement top and bottom surfaces, it can be said that the maximum upward curling occurred sometime between 4:00 and 7:00 am. Similar trends were observed at other locations.

Measured curling on other days and temperature differentials between the pavement top and bottom are shown in Table 1. The results show that although the maximum temperature differential on September 2 was higher than that on August 7, measured maximum upward curling was not much higher (0.023 in. compared to 0.025 in.). It happened probably because of the fact that the ambient temperature on September 2 was much lower than that on August 7. The average ambient temperatures on August 7 and September 2 were about 80°F and 69°F, respectively. The last set of measurement was carried out in early November. The maximum upward curling was much lower than previous measurements (about 50% of the first measurement). This is obvious because the maximum positive temperature differential is much lower (about 13°F) compared to other measurements. For the last two measurements, both pavement temperatures and temperature differentials were much lower compared to the other three measurements. During these two measurements, for only about 30% of the time, the pavement experienced a positive temperature gradient, compared to over 45% of the time for other measurements. Since temperature differential has a significant influence on curling, the effect of curling can be mitigated at a early age of the pavement concrete with proper measures, such as enhanced curing.



(a) Measured curling



(b) Temperature differential

Figure 6. Measured curling and temperature differential on August 7, 2003

Table 1. Curling deflection and temperature differential values

Day / Time	August 7		September 2		September 25		October 15		November 6	
	Curling (mils)	Temp. Diff. (°F)	Curling (mils)	Temp. Diff. (°F)	Curling (mils)	Temp. Diff. (°F)	Curling (mils)	Temp. Diff. (°F)	Curling (mils)	Temp. Diff. (°F)
8 am	-	-9.0	-2.008	-9.0	-0.984	-11.7	-	-9.9	-	-8.1
9 am	-0.984	-5.4	-5.000	-5.4	-0.984	-7.2	-2.992	-8.1	-2.008	-8.1
10 am	-2.992	0.9	-0.402	-0.9	-2.008	-2.7	-2.008	-1.8	-2.992	-4.5
11 am	-2.008	7.2	-2.008	2.7	-2.008	2.7	-2.008	4.5	-0.945	0.0
12 pm	0.984	12.6	2.008	9.9	2.008	8.1	2.992	9.9	-0.984	4.5
1 pm	10.984	18.9	7.992	16.2	9.016	11.7	9.016	12.6	2.992	9.0
2 pm	14.016	21.6	12.992	20.7	15.000	14.4	12.992	14.4	7.008	11.7
3 pm	25.000	22.5	22.992	23.4	17.992	15.3	17.008	15.3	14.016	12.6
4 pm	20.984	20.7	20.000	24.3	20.000	14.4	15.984	13.5	12.992	10.8
5 pm	17.992	18.0	15.984	21.6	15.000	11.7	15.984	10.8	10.984	7.2
6 pm	15.000	14.4	12.992	17.1	14.016	7.2	12.008	7.2	9.016	1.8
7 pm	12.992	9.9	12.992	11.7	10.984	2.7	-	3.6	-	-2.7

(negative sign indicates upward curling)

FINITE ELEMENT MODELING

Geometry and Element

The finite element (FE) model consists of three layers, as shown in Figure 7. The top layer is the 12-in.–thick concrete slab. The bottom two layers are a 4-in. BDB layer and a 6-in. lime-treated subgrade. The model consists of two lanes (driving and passing) and two shoulders (inside and outside). Each lane is 12 ft wide, whereas the widths of the inside and outside shoulders are 6 ft and 10 ft, respectively. All lanes and shoulders are separated by longitudinal joints with width of 3/8 inches and a depth of a quarter of the slab thickness. Transverse joints in the model are located at 16.4-ft intervals, and the joint dimensions are the same as those of the longitudinal joints. Cracks that developed under the transverse joints were also modeled. Dowel bars were used as load transfer devices. Dowel bars are located at the mid-depth of the slab with a bar diameter of 1.5 inches and length of 18 inches. Dowel bars were placed at 12-in. intervals. Because of the symmetry in the longitudinal (driving) direction, half of two slabs on either side of a transverse joint were used as the model geometry. A 3D 20-node brick element, known as SOLID186 in the ANSYS (ANSYS 2003) library, was used in this study. The element is defined by 20 nodes with three degrees of freedom per node: translations in the nodal x, y, and z directions. The interaction between the concrete and the steel dowel bars is complex, and this interaction was modeled as a contact problem. Rigid-to-flexible type of contact is also available in the ANSYS library. Target element, TARGE170 and contact element, CONTA174 were used in this study to model target and contact surfaces, respectively.

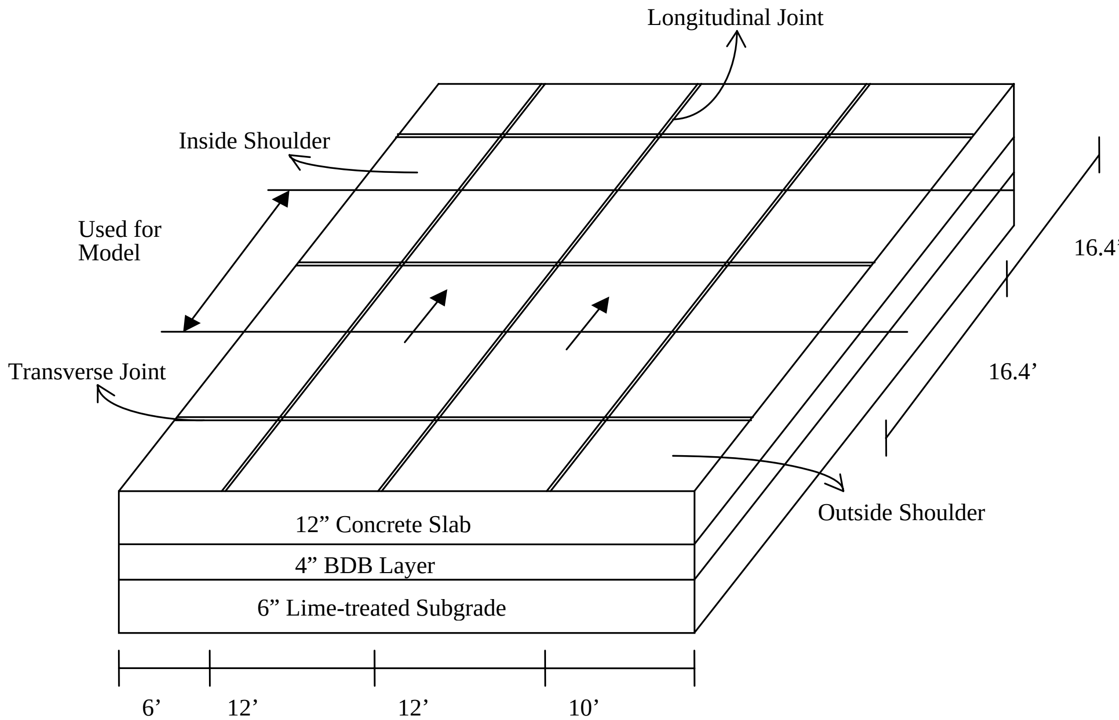


Figure 7. Geometry of the final element model

Material Properties

All layers were modeled as linear elastic. Material properties needed for this FE model include elastic properties, such as the modulus of elasticity and Poisson’s ratio of different layers, density of concrete, etc. Since this study deals with curling caused by temperature, another important material property that was needed is the coefficient of thermal expansion. Modulus of elasticity for the slab, BDB layer, and subgrade layers used in the study were 4.2×10^6 psi, 9.5×10^5 psi, and 40,000 psi, respectively. Poisson ratios for the concrete, BDB, and lime-treated subgrade layers were assumed to be 0.15, 0.15, and 0.20, respectively. The modulus of elasticity and the Poisson ratio for the dowel bars were assumed as 29×10^6 psi and 0.25, respectively. The typical values of 5×10^{-6} in./in./ °F, and 6.5×10^{-6} in./in./°F were used as the coefficients of thermal expansion values of concrete and steel, respectively. Temperature data, collected by iButtons, was applied at the top and bottom of the concrete slab. The thermal loading applied represent a temperature differential of -10°F, -5°F, 5°F, 10°F, 15°F, and 20°F.

Mesh Generation and Loading

Meshing is an important part of an FE model. Finer meshes produce better results. However, several factors, such as size and complexity of the geometry, use of contact elements, and product restriction of the ANSYS version used in the study, restricted the creation of a very fine mesh. In general, the mesh is coarse, as shown in Figure 8. However, because of discontinuities created by joints in the pavement, areas near the joints were refined to obtain better results. The total number of elements generated for each model was approximately 30,000. As this study deals with curling, temperature was used as the main load. Temperature data was applied at the top and bottom of the concrete slabs. Hence, it was assumed that the temperature distribution across the slab is linear. The bottom of the subgrade layer was assumed to be fixed in all directions. The edges of the base and subgrade layers were fixed in the z-direction

(direction of traffic). The pavement edge was allowed to move in all directions. Both translations and rotations of the dowel bars were restrained in all directions at one side of the joint.

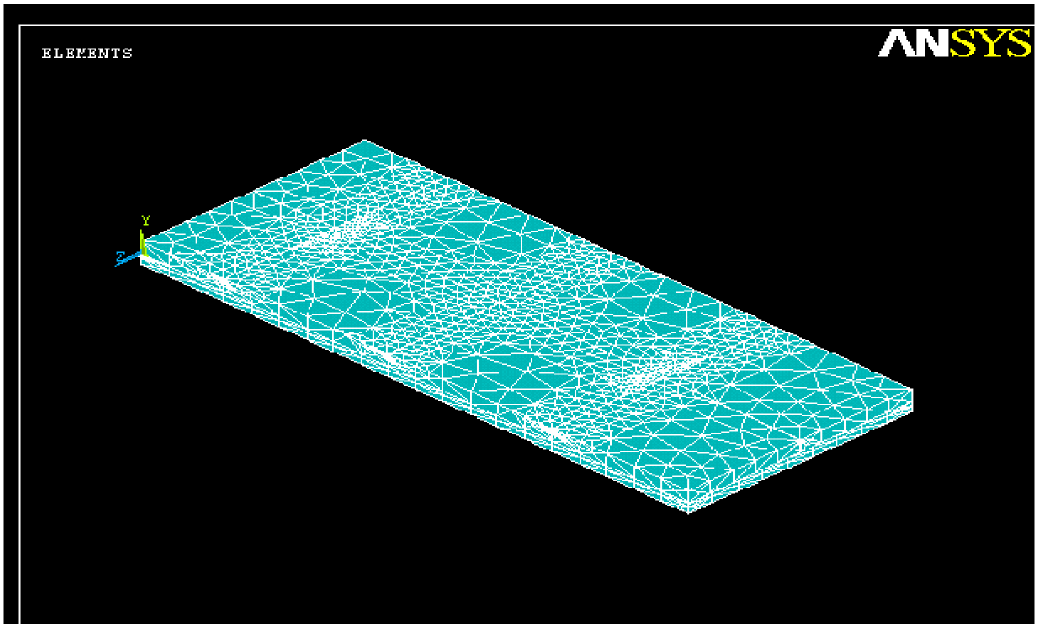


Figure 8. FE Simulation results for measurement section

Figure 9 shows the curled profile of the section for these temperature differentials. As expected, both upward and downward curling deflections increase with an increase in temperature differential. A maximum deflection of 0.012 inches was obtained when the temperature differential was maximum (20°F). However, curling from a temperature differential of 20°F is not twice that amount due to a temperature differential of 10°F, but about 75% higher. Again, the magnitude of curling for the same positive and negative temperature differential was not the same. A positive temperature gradient resulted in a higher magnitude of curling. The magnitudes of upward curling were about 74% and 80% of those of downward curling for temperature differentials of 5°F and 10°F, respectively.

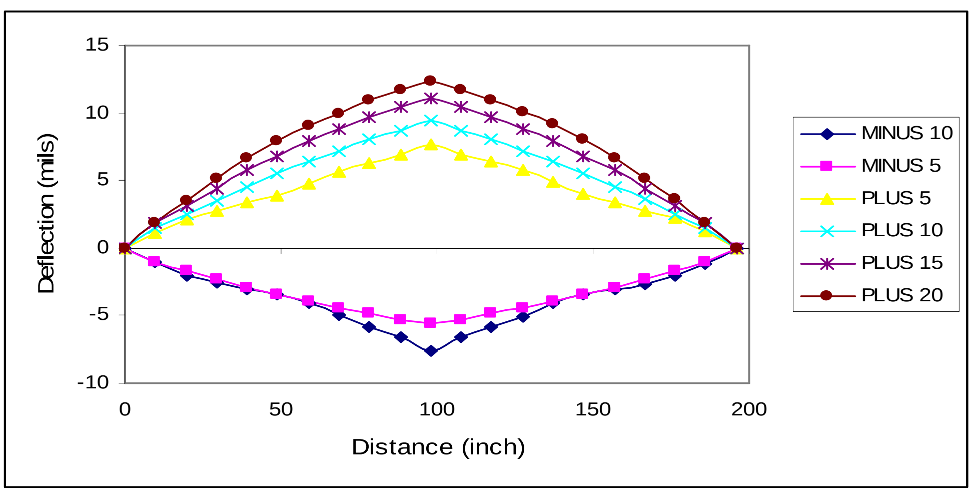


Figure 9. Simulated deflection profiles for different temperature differentials

COMPARISON OF RESULTS

Figure 10 shows the comparison of curling deflection data obtained from the field measurements and the FE simulation. As mentioned earlier, during curling measurement for any given day, it was assumed that the slab was flat during the first measurement. Subsequent measurements were done hourly. Hence, these measurements showed the curling deflection of the section with respect to the first measurement. The relative temperature differential shown in Figure 9 is the difference of temperature between the pavement's top and bottom surfaces for a particular measurement and the first measurement. For example, if the temperatures of the top and bottom surfaces during the first measurement are 84°F and 89°F, respectively, and those at 3 pm are 114°F and 91°F, respectively, then the relative temperature of the top and bottom surfaces are 30°F and 2°F, with a temperature differential of 28°F. Finite element analysis was performed using properties of the curling measurement sections and relative temperatures simulating the actual temperature condition during the curling measurement. Results from both methods show a similar trend, although the actual values are different. Deflections obtained by the FE simulation were lower than those measured in the field. Better agreements were observed for lower temperature differentials. However, the difference increased with an increase in temperature differential.

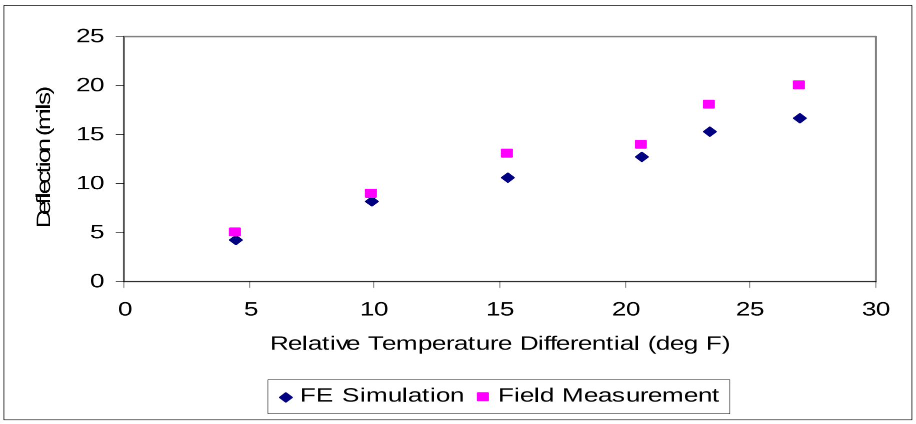


Figure 10. Comparison of results

It is to be noted that beside temperature, a pavement experiences changes in moisture with time. Ambient temperature also has some effect on the response of a pavement. There are also some construction irregularities that result in a pavement profile that is not perfectly plain. These features were not considered during FE model formulation. All or some of these factors might contribute to the differences between the FE simulation and field measurement results.

CONCLUSION

This study presents the results of curling and temperature measurements on a concrete pavement test section in Kansas. The curling was also simulated with a finite element model of the section. The simulated curling deflections were compared with those measured in the field. Based on the study, the following conclusions can be made:

- Higher temperature differentials increase both upward and downward curling. The magnitude of curling deflection resulting from a particular positive temperature differential is slightly higher than that resulting from the same negative temperature differential value. Since temperature differential has a significant influence on curling, the effect of curling can be mitigated at an early age of the pavement concrete with measures such as enhanced curing.
- The curling deflections measured in the field were in close agreement with the deflections obtained by the FE simulation, in most cases.

ACKNOWLEDGMENTS

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Regional ITS Architecture Development in Rural and Small Urban Areas

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ABSTRACT

Intelligent transportation systems (ITS) have been used across the United States to improve safety, enhance operations, and boost customer service. Traffic signal timings automatically adjust to regulate traffic flow, electronic signs advise motorists of hazards and alternate routes, and computer and telephone systems keep drivers abreast of travel conditions along their intended routes. Sometimes, however, those systems are not so seamless. As traffic moves from one jurisdiction to another, for example from rural highways to city routes or across state lines, information flow can be interrupted or inconsistent. Drivers may become confused and traffic may slow to a crawl.

To avoid those situations, the Federal Highway Administration (FHWA) is requiring transportation planning regions nationwide to develop a Regional ITS Architecture, a plan and vision for ITS implementation and use. Regions that do not comply by the spring of 2005 will no longer receive federal highway funding for ITS projects. The regional architecture will serve as a roadmap guiding future ITS planning, detail system requirements, coordinate agency roles, and integrate functions across jurisdictional lines. Therefore, the architecture development brings together nontraditional stakeholders while it emphasizes interagency and interjurisdictional coordination and operations.

This paper will discuss North Dakota's approach to meeting FHWA requirements and developing regional architectures that facilitate ITS in the state. It will report on four regional ITS architecture developments carried out by the Advanced Traffic Analysis Center at North Dakota State University. The paper discusses possible approaches that may be useful to other regions developing, using, and maintaining ITS architectures. It illustrates successful stakeholder participation strategies using a targeted small group format. The use of Turbo Architecture to generate architecture information and support stakeholder discussions is also explained. The paper also provides examples of using the architecture for a project involving two state transportation departments as part of the North/West Passage Pooled Fund Study.

Note: This research was still in progress at the time of publication; contact the lead author above for more information.

Key words: intelligent transportation systems—North/West Passage Pooled Fund Study—regional ITS architecture

Supporting Transportation Planning Models in Small Metropolitan Planning Organizations through Partnerships and Innovative Methods

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ABSTRACT

Metropolitan planning organizations (MPOs) must develop and maintain transportation planning models to support a multitude of system decisions and transportation and land use policies. The main components of transportation planning models include travel demand estimation, calibration, and network performance assessment. Evaluating network performance is a straightforward process that compares estimated service levels to those established by each community. However, the model must be calibrated before future traffic levels are examined. Traffic counts on key routes usually receive great attention during calibration, and they may trigger institutional conflicts. Estimating travel demand involves predicting the number of trips by activity.

Although the basic microeconomic principles used to estimate trips are straightforward, applying them to capture trip making behavior accurately gets to be a tricky endeavor. The availability and quality of local data may be the most critical obstacle that MPOs must face when estimating travel demand. This problem becomes even more acute in small MPOs, which lack the resources to undertake significant primary travel data collection. The lack of data also impacts the ability to calibrate models to reflect an area's unique characteristics and to allow reliable future forecasts. Furthermore, in many instances, the smaller MPOs lack the staff size and expertise necessary to develop, run, and maintain transportation planning models. Many smaller MPOs have difficulty recruiting or retaining skilled transportation modelers.

This paper presents an innovative approach that uses a partnership between academia and local, state, and federal transportation agencies to meet transportation modeling demands in North Dakota. The program consolidates all model development, enhancements, maintenance, at the Advanced Traffic Analysis Center (ATAC) at North Dakota State University to maintain all travel demand models in North Dakota. Furthermore, ATAC is the sole entity designated with the role of running models in support of the needs of various MPOs, state DOTs, and consultants. This paper discusses the approach to establishing the program, lessons learned, and challenges.

Key words: metropolitan planning organizations—transportation planning models

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INTRODUCTION

Metropolitan planning organizations (MPOs) have to develop, maintain, and update transportation plans to guide transportation and land use (and land development) decisions. Proper transportation planning is key for achieving a transportation system that supports the vision and goals of communities, i.e., safety, mobility, economic opportunity, clean air and a clean environment, and quality of life in general. It is no surprise that across the country in large metropolitan areas and rural towns alike, growth and development (and their implications on traffic and congestion) are receiving increased attention. One of the challenges for today's transportation officials include providing (and maintaining) a transportation system that supports the existing activity system and, more importantly, future growth. Another major challenge is to provide support with very limited transportation budgets, increased public scrutiny, and strict environmental demands.

Transportation planning is therefore governed by extensive and detailed requirements at the federal, state, and local levels (U.S. Congress). At the heart of these requirements is how transportation models are developed to support transportation plans. Transportation planning models generally incorporate several main components to account for trip making behavior by individuals given their socioeconomic characteristics and the attributes of available transportation services (mode-specific infrastructure and services). Given the potentially large number of travelers within a metropolitan area, individual decisions are often aggregated to the zonal level for similar land-use and socioeconomic characteristics. This aggregation, however, must be handled with great care and preferably supported by local travel data. The transportation planning process has been standardized over the years and is frequently referred to as the four-step process. The four steps are trip generation, trip distribution, modal split, and trip assignment. Each of these main components produces essential output that is used by subsequent steps, and therefore impacts the overall accuracy of the model.

Although each of the model steps on its own uses basic microeconomic principles, combining these steps into a modeling system that effectively addresses analysis scenarios is a complex process. The relationship between transportation and land use is dynamic and can best be characterized as multifaceted. Furthermore, predicting future growth and the associated demands on the transportation system is not an easy exercise and requires substantial data and different types of modeling systems (i.e., statistical, socioeconomic, political, financial, and transportation networks). Naturally, transportation modeling software was developed to address these needs. However, these software models have also gotten more complex with the availability of computing abilities and now frequently include GIS to facilitate data analysis. As a result, the level of resources often implies how detailed and sophisticated these models will be in a particular application.

Adequate and accurate data are probably the most essential (and the most resource-intensive) component to support modeling. Access to and retaining qualified staff for developing and maintaining transportation models are increasingly becoming critical issues as well for both public transportation agencies and private sector transportation firms. The developments in the complexities of the modeling systems and the decisions/issues they have to support will only add to the shortage of qualified individuals who can operate and support them. Microscopic simulation tools are increasingly used to model traffic operations with extensive details that allow a user to analyze various modifications to transportation networks and traffic behavior. These tools are now being used in combination with traditional planning models. Interfaces between planning and operations are being emphasized in new generation models, such as the U.S. DOT's TRANSIMS (2001), capable of simulating second-by-second movements of every person and every vehicle through the transportation network.

The issues of resources and data availability are more critical for smaller MPOs. These MPOs lack the funding and the staff resources to meet the data requirements and maintain state-of-the-art models. It is very common for these MPOs to rely on private consultants to develop, run, and maintain their models. However, such an arrangement could create a less-than-perfect situation where agency personnel have limited access to the model. Furthermore, the consultants who are often based at a different location may lack the knowledge of the modeled area, which becomes crucial for scenario analysis and model calibrations.

Due to these factors, a transportation planning modeling support program was envisioned to assist in modeling needs in North Dakota. The remainder of this paper will discuss this program, describe the participating partners and institutional relationships, illustrate the types of projects included, and highlight its success stories from over three years of operation.

SMALL MPO NEEDS

This section briefly describes some of the common characteristics of smaller MPOs, which significantly impact their ability to develop and maintain transportation planning models. For the purpose of this discussion, small MPOs are those metropolitan areas with a population between 50,000 and 200,000. Although some smaller urban areas with populations under 50,000 may still carry out transportation planning modeling, they do not have to meet the strict federal planning requirements for officially designated MPOs.

Some of the distinguishing characteristics (and issues) of smaller MPOs most relevant to transportation planning models include the following:

1. Resources
2. Limited data availability
3. Growth potential
4. Institutional complexity (interjurisdictional and interagency issues)

Although these issues were largely observed in the three MPOs included in the support program in North Dakota, they may very well be representative of other similar-sized MPOs nationwide. In fact, similar issues were identified in 2000 by the Transportation Research Board to be of high priority in the new millennium (Schutz 2000). Other potential issues identified in that effort included new technologies (intelligent transportation systems), modeling techniques that include multimodal issues, and communications and information overload (Schutz 2000).

Resources

The lack of resources is often cited as the most critical challenge facing government agencies in general and transportation agencies in particular. However, for smaller MPOs, inadequate resources could have detrimental effects on their ability to carry out transportation plans and fund supporting modeling activities. In most cases, the lack of funding leads to an inability among smaller MPOs to hire or retain full-time staff to support transportation modeling activities. Therefore, cost minimizing strategies often override the need to meet minimum planning and modeling requirements. Below is a discussion of some of the specific resource issues and their impacts on transportation planning modeling in smaller MPOs.

Limited Funding

The smaller MPOs (with populations under 200,000) usually receive less federal funding and have less control over how their funds are used. In some cases, the state transportation agency (DOT) plays a major role in influencing MPO programs and may also provide modeling support to smaller MPOs. However, in states that are predominantly rural, developing and maintaining transportation planning models may not be a high DOT priority.

In terms of absolute dollars, the current law (U.S. Congress) provides a disproportionate amount of funding for transportation planning activities, compared to traditional infrastructure and capacity enhancements. Table 1 shows funding amounts under TEA 21 for major programs, as well as metropolitan planning for selected states. The table also shows the percentage of the population living in urban areas among the selected states, based on 2000 Census data (U.S. Census Bureau 2000). The portion of federal funding allocated to metropolitan planning is mostly less than 1% of total funding awarded to the states.

Table 1. Selected 2002 funding levels (in millions) and population in urban areas

	TEA 21 Funding Program							
State	**Interstate maintenance**	**NHS**	**STP**	**CMAQ**	**Metro planning**	**Total**	**% for metro planning**	**% population in urban areas**
CA	346.5	443.3	539.8	292.5	30.0	2,822	1.1%	94.4
CO	63.4	80.6	85.9	19.9	2.8	396.6	0.71%	84.5
IA	53.8	76.8	81.5	6.9	1.1	368.2	0.30%	61.1
ND	23.3	64.6	35.7	6.9	1.0	201.7	0.49%	55.9
SD	27.7	54.2	38.9	6.9	1.0	223.7	0.44%	51.9
UT	53.1	38.4	46.3	8.8	1.6	239.9	0.65%	88.2
WY	39.8	69.5	28.1	6.9	1.0	214.6	0.46%	65.1

Funding Data and Software Updates

Modest MPO planning budgets also impact data collection and software updates, which affect not only model development, but also model calibration. Travel surveys provide a region with valuable travel behavior information, essential for developing models that can effectively explain local and regional travel behavior. These studies are expensive and time consuming. Therefore, travel survey data are often outdated or nonexistent in smaller metropolitan areas.

Software companies are continuously marketing new planning products or software upgrades. Most of these products cost thousands of dollars to purchase. It is extremely difficult, because of cost constraints, for an MPO to update their planning software continuously when new products are introduced. They must decide on the best and most economical software for their own individual purpose.

Few Modelers

Developing and maintaining transportation planning models require extensive knowledge. Universities serve as a valuable resource to expose future transportation college graduates to transportation planning and modeling. However, at the undergraduate level, many transportation-related programs, like Civil Engineering, do not incorporate specialized transportation planning classes into the degree curriculum.

Therefore, graduates may not have the fundamental technical background needed to support transportation planning functions.

The lack of young professionals exposed to planning is one reason why smaller MPOs have a difficult time recruiting knowledgeable staff. If an MPO successfully employs and trains a young talented modeler, the MPO will often lose them to a higher paying job at another organization. Again, the MPO will be faced with the reoccurring problem of finding experienced modeling staff.

Growth Potential

Because of their relative size and the availability of vacant land, the smaller MPOs continue to experience significant growth. The quality of life in these MPOs, especially low crime rates, good air quality, and little traffic congestion; continue to attract new residents. Additionally, aggressive economic development efforts often contribute to an increasing population as new jobs are created. With this growth, the relative impact on the transportation system could be quite significant. For example, a single housing development that changes land use from farmland to single-family dwellings could greatly impact the level of service on surrounding roads. Similarly, specialized stores, such as a Super Wal-Mart, Home Depot, and others, could become regional shopping destinations, attracting traffic from surrounding areas. Again, the impact on the transportation network is profound.

Limited Local Data

Data availability issues are common to MPOs of any size. However, these issues are more critical in smaller MPOs largely due to limited resources and the relative inexperience in developing, calibrating, and maintaining transportation planning models in these MPOs. Several types of data are required to support effective modeling, including socioeconomic data, travel behavior, and network traffic flows.

Since transportation planning models directly depend on estimating the level of demand on the transportation system, data to support accurate trip generation are most important. Most MPOs have good land use data for the traffic production step; however, the availability of data for estimating attractions is mixed. For intermodal (freight) traffic generation, there is a severe shortage of useable data. That shortage, combined with modeling shortcomings, means that freight traffic flows are ignored completely in metropolitan transportation planning models in smaller MPOs.

Travel behavior data support three main modeling components: trip generation, trip distribution, and mode choice. The main source for these data has traditionally been travel surveys (also referred to as O-D surveys). These types of data are either unavailable or outdated in the smaller MPOS and hence have little value to support transportation planning models. Budget limitations are perhaps the main cause for smaller MPOs' inability to undertake major travel survey data collection.

Network traffic data are essential for understanding the traffic assignment process and calibrating the transportation planning models to base conditions. Although many smaller MPOs collect traffic flow data periodically, these collection plans often are done in absence of any consideration as to how the data would be used to support model development or calibration. Therefore, there is less data coverage (both location and time period), which means less comparable data for model calibration.

Multi-agency/multi-jurisdictional issues

The smaller MPOs have to deal with other (and perhaps more powerful) agencies in balancing their transportation plans. In North Dakota, each of the three MPOs has one major city with the majority of the population (and hence the funding), giving it more control in future plans. This issue may have significant impacts on transportation planning modeling, i.e., favoring the wishes of the more powerful agency. Similarly, state DOTs may have a more powerful role and may dictate the modeling approach, model calibration, and even choice of software.

PROGRAM DESCRIPTION

This section describes the details of a transportation planning modeling support program established at North Dakota State University (NDSU) in 2001. The program was developed in response to growing modeling needs among the program partners and their inability to retain full-time modeling staff. The emphasis of the program is on modeling enhancements that would be useful regardless of the modeling software used and would be applicable for North Dakota MPOs and other small- to medium-sized urban areas. The program builds on a strong partnership between the Advanced Traffic Analysis Center (ATAC) at NDSU and various state and local transportation agencies. ATAC has staff and students with significant experience and familiarity with analysis tools and links to transportation research organizations and groups. The program consolidated planning model improvement activities in North Dakota at ATAC. This has afforded individual MPOs, NDDOT, and private consultants greater opportunities to benefit from modeling expertise that would otherwise be unavailable.

Program Partners

The program was developed at ATAC in partnership with the three MPOs of Bismarck-Mandan, Fargo-Moorhead, Grand Forks-East Grand Forks; NDDOT; and the Federal Highway Administration Division in North Dakota. The role of NDDOT and the FHWA was to facilitate the partnership and also to ensure that MPO transportation planning models meet federal and state requirements. It is also common that for some of the major corridor analyses, federal, state, and local agencies are involved in interpreting the results and finalizing future design plans. Therefore, effective participation from all of these agencies is paramount for the success of the transportation planning modeling support program.

Institutional Arrangements

The program is guided by a steering committee consisting of MPO directors, an urban engineer for NDDOT, a planning engineer from the FHWA's division, and two ATAC staff. The committee is responsible for maintaining the required partnership agreement for the program and allocating programmatic funding. Committee meetings are held twice a year to discuss progress and identify potential work activities. The respective advisory boards for each MPO have endorsed the partnership agreements and are updated on program activities as needed. Both technical and policy MPO advisory boards have been supportive of the program due in large part to the value they perceive from its services.

The program is governed by a master agreement ratified by program partners and NDSU. The master agreement specifies, among other legal provisions, the annual contribution amounts, the types of activities allowable under the agreement, the performance period, and the mechanism for conducting work by special request. Model improvement, model runs, corridor analyses, and other special studies are covered under addenda to the master agreement. These addenda can usually be processed within one or two business days, thus enhancing ATAC's ability to respond to the MPO's analysis needs as they arise.

The program funding may be categorized into two main sources: core funding and activity-based funding. Core funding for the program consists of annual contributions in the amount of $5,000 made by each of the program partners. These contributions are used to provide ATAC with basic funding for general program activities that benefit all partners. Initiatives that affect more than one MPO, such as acquiring additional modeling tools, technical training for staff, and general operating expenses for the program, are covered under the core funding. Activity-based funding consists of model improvements, corridor analysis, or special studies requested by any of the program participants. Activity-based funding has been at least double the amount of the core funding per year.

Private consultants participate in the program in a variety of ways, but mostly through a contract with one of the program partners. The types of services conducted to support consultants' work falls under the master agreement and are often covered by provisions between the contracting MPO and the consultant. Consultants also participate in training activities offered under the program. They are also valuable participants in model improvement forums.

To illustrate how MPOs and their consultants interact with ATAC modelers, it may be worthwhile to describe a typical arrangement for a corridor study. Generally, the MPO consults with ATAC staff during the preparation of the Request for Proposal (RFP) in order to identify potential modeling needs and possible resources for the study. The RFP specifically states that the selected consultant would identify modeling needs (i.e., model runs to reflect analysis scenarios) and work with the MPO and ATAC on coordinating the analysis. ATAC is paid through the MPO, which may retain a portion of the project award to cover the modeling cost. All work requests are submitted through the MPO using the standard addendum form. All the changes to the model are accounted for and documented by ATAC staff, and hence there is little chance of compromising the model. Furthermore, there are significant cost savings that result from this specialization because of the familiarity of ATAC's modelers with the analysis and their ability to use enhanced software tools to meet the requirements.

Program Activities

This section discusses the types of activities conducted under a North Dakota transportation planning modeling support program. The discussion is intended to highlight major types of projects rather than list all possible studies or activities.

Model Overhaul

This category of program activities includes major model enhancements, such as changing the traffic analysis zones' structure, changing the model traffic generation formulas, adding new trip types, etc. These types of enhancements are typically done during major transportation planning model updates, which also entails developing a long-range transportation plan. Additionally, implementing new software to support transportation planning models also falls under this category.

Under the current program, a major model update for the Fargo-Moorhead area was completed, and one is underway for the Grand Forks-East Grand Forks area. During the Fargo-Moorhead update, the number of traffic analysis zones was almost doubled from a 1995 version of the model. Additionally, transportation network components were revised to reflect recent changes. The model's approach to measuring network performance was also enhanced by refining intersection cost (delay) values based on traffic operational analysis in the area.

Model Calibration

This type of work involves calibrating existing models to a particular year or land-use and traffic conditions. The calibration is mostly triggered by major changes in conditions or analysis to support major projects. The calibration does not involve major model changes (i.e., model estimation formulas remain intact).

Sub-Area/Corridor Analysis

There is often a need to apply macro-level travel demand models to a micro-level or corridor level analysis. Since demand models are developed regionally, model refinements may need to be made to run these analyses. Refinements may consist of adding traffic analysis zones, adding roadway segments, adjusting network loading, or adjusting predictive socioeconomic data. Because of model sensitivity, any future model adjustments need to be examined carefully to verify that results are reasonable compared to the calibrated model. Corridor analyses provide valuable demand and network performance information that aids in determining future intersection configuration, roadway geometry, and future roadway right-of-way boundaries.

Most of the recent work in this area has focused on determining projected turn volumes. For this type of analysis, agencies provide current turn volumes at all corridor intersection approaches. These current counts are compared to the calibrated model volumes to produce a future year adjustment coefficient. This coefficient is applied to the future turn volume according to a method described in National Cooperative Highway Research Program Report (NCHRP) 255 (Pederson and Samdahl 1982). For each predictive intersection turn volume, target volumes are established based on the error between the current counted and the calibrated model volume and the future year predictive turn volume. This method assumes that the future year model will also predict that particular movement flow. Since applying a target value affects the volume balance at each intersection, ATAC developed an application that uses a matrix estimation program to balance the intersection volumes. This program minimizes the deviation of the assigned volume to the target value while maintaining balanced intersection flow through the corridor.

PROGRAM BENEFITS/ACCOMPLISHMENTS

This section highlights some of the key benefits and accomplishments of North Dakota's transportation planning modeling support program. This information is largely based on the program activities during the last three years. It should be noted that the recognition of these benefits by program partners has resulted in renewing the program for another three years. It is expected that the program will continue to grow in the future by tackling additional issues.

Model Improvements

One of the most direct benefits resulting from the program was to allow partner agencies to have a concerted effort for upgrading and updating existing travel demand models. Due largely to resource limitations and the dependency on outsourcing for developing models, improvements were not easily attainable. The program steering committee frequently discusses potential model improvements at its semi-annual meetings. Two of the MPOs have officially set up transportation model improvement committees to identify and guide future improvements.

One of the first MPOs to receive a major model update was the Fargo-Moorhead Metropolitan Council of Governments. The update included doubling the number of analysis zones, revising the network structure, refining intersection costs, improving model calibration, and implementing additional software features that facilitate the analysis. One of the important supporting activities of this model update was to take advantage of GIS capabilities in preparing the data and communicating the results to both technical staff and elected officials. Primary data were collected in order to support network performance measurement (i.e., travel time) and enhance the model's ability to capture additional trips (school trips). It should be noted that due to the effective partnership built in the program, these benefits were not restricted to the Fargo-Moorhead Metropolitan Council of Governments. Other project partners were aware of these improvements and how they could utilize similar strategies to improve their own models. Also, the data collected in the Fargo-Moorhead area, such as school trips, would apply in the other two MPOs due to their similar characteristics.

Modeling Consistency

One of the main advantages to the North Dakota support program is providing a forum for all three MPOs and the NDDOT planning staff to discuss common modeling issues. Also, by having a single source for technical expertise at ATAC, the process of reviewing and calibrating models is greatly enhanced. Most importantly, this partnership provides for private consulting firms to participate, depending on the type of projects, and still benefit from this consistency. Consultants request model runs with various land use, transportation network, or other model input scenarios from ATAC under the service agreement with the MPOs. ATAC is responsible for making the changes, running the model, and presenting the results to the MPO and consultant. This mechanism ensures the integrity of the travel demand model, since all the changes are carried out by the ATAC staff, while meeting the modeling needs of the project partners and their consultants.

Modeling Software

Many transportation agencies struggle with the issue of transportation-related software, whether it is for traffic operation analysis or transportation planning. Software purchases can consume significant portions of the limited budgets of small MPOs. The recent trend of more powerful computer processing continues to fuel larger, more complex, and more expensive models. Additionally, it is difficult to interpret the results produced by the various transportation planning software tools. Other issues related to modeling software are maintenance and updates. Therefore, ATAC has been designated with the role of maintaining current modeling software used in North Dakota and evaluating needed upgrades. ATAC's modelers frequently evaluate new versions and enhancements to the modeling software in order to identify justified changes. In turn, the MPOs benefit by having a single source for current information on state-of-the-art modeling software.

Finally, modeling software-related training for agency staff can often be costly and time consuming. The North Dakota program addresses training needs by providing opportunities for informal and one-on-one training to partner agency staff. Having in-house training greatly cuts back on travel costs, training fees, and time away from the job for the staff. Furthermore, receiving training from ATAC ensures consistency and provides opportunities for using examples from actual MPO models in the training.

Stability and Continuity

Recruiting and retaining qualified modelers present great challenges to all transportation agencies, but more so for smaller MPOs. The limited budgets of the smaller MPOs often affect their ability to offer competitive compensation to potential modelers. As a result, smaller MPOs generally lack modeling staff with the experience necessary to operate, maintain, and update the model. They rely on consulting firms, either through a retainer arrangement or on a case-by-case basis, for meeting their modeling needs. Having to work with different firms and deal with changing modelers within these firms are not conducive to programmatic model enhancement and updates. Unless there is a commitment on the MPO's part (and the associated resources are available), the consulting firm has little incentive to perform model improvements and enhancements.

Under the support program, ATAC maintains properly trained staff and students to ensure the stability of the program. Also, due to the large reserve of graduate students familiar with transportation modeling, it is possible to increase staffing levels to meet project deadlines. ATAC has to illustrate to the program steering committee that adequate resources are committed to the program. In turn, they assist in securing additional resources needed to carry out the program activities.

Responsiveness to Modeling Needs

It is common for land use changes or development decisions to trigger the need for modeling analyses to examine their impacts on very short notice. This means that changes in conditions must be represented in the model and calibrated prior to producing the results. Given the level of detail involved in corridor studies, it is often necessary to create a sub-area with a higher level of detail than the metropolitan model. After the sub-area analysis is complete, interpreting the results and communicating them to the decision-makers can be a political and time consuming process.

These types of requests have been streamlined in the North Dakota program. First, there are procedures in place to identify potential analysis needs ahead of time as much as possible (programmed needs). Second, a prioritization system for handling agency requests was established by the program's steering committee to recognize the timing constraints for certain requests. Third, a process for conducting sub-area analyses was developed with the program partners to ensure expedited approval of the results. Technical and decision-making staff from the MPOs, cities, and NDDOT participated in a training workshop on the methods and software tools typically used in sub-area analysis. Finally, ATAC has acquired software enhancements to facilitate and expedite sub-area analysis.

Economics

In addition to providing responsive and effective modeling expertise, the North Dakota program offers significant cost savings to the program partners. This is largely due to the effective partnerships established by the program. The three MPOs and NDDOT provide base funding for the program through an annual participation fee. The base funding covers some of the core activities, such as model enhancements, training, and software updates. Agency-specific work activities are handled on a cost-per-use basis. However, these costs are minimized in the program due to the familiarity of the modelers with analysis procedures, therefore eliminating start-up time. It should be noted that the cost savings are also passed on to the private consulting firms conducting corridor analysis.

Supporting private consultants

One of the initial challenges faced during the program establishment was to sort out the relationship with and impacts on private consulting firms. At issue was possible competition between an academic institution and private industry. However, the program steering committee quickly realized how this program can help private consulting firms that are strong in corridor studies but may lack modeling experience. As a rule, ATAC does not compete on corridor studies, but rather provides modeling support to the contracting agency and its consultant.

Further, the types of model enhancements afforded by a research environment are not typically available in the private sector. The knowledge and techniques developed through the program are therefore helpful to both private consultants, as well as other small MPOs.

Recruiting and Training Students

By establishing the transportation planning modeling support program at NDSU, both graduate and undergraduate students are involved in the program activities. Typical student involvement ranges from data processing to researching model enhancements. As more students are exposed to transportation planning models, their future career decisions (and opportunities) are greatly enhanced. This is especially true for undergraduate students in civil engineering who often lack any particular specialization. Participating agencies and private consulting firms can potentially recruit new graduates who are familiar with transportation planning models.

Additional Support Areas

The success of the program in providing transportation planning modeling service has triggered additional support areas for the program partners. The program strengthened the relationship between MPOs and NDDOT with the university and created effective mechanisms for sharing ideas and leveraging resources. Specifically, there has been an increase in support activities in the areas of intelligent transportation systems and traffic operations

Program Challenges

Although the North Dakota support program has been successful in meeting the desired objectives for the MPOs, NDDOT, and NDSU, this success did not come without difficulties at times. The most significant challenges were experienced during the early stages of establishing the program. It should be mentioned, however, that these challenges were not insurmountable due to a critical need among the program partners, strong partnerships with the MPOs and NDDOT, and a clear and measurable benefit to program participants. Below are some key challenges and how they were addressed in the North Dakota program.

1. *Developing a clear business plan*. This may be the most critical factor in selling the program to transportation agencies' technical staff and policy makers. The business plan must outline goals and objectives, potential institutional arrangements, funding, accountability, and possible conflicts.
2. *Securing stable funding*. Transportation agencies generally shy away from multi-year funding agreements. However, stable funding was crucial for the university in order to invest in setting up the program and developing the required expertise and resources. As a compromise, the program was initially set up for two years with the possibility of canceling the agreement. After the initial two-year period expired in 2003, the program was renewed for three more years and is expected to be renewed for a similar period after 2006.
3. *Clarifying program relationships with private sector consultants*. This issue had to be faced early as the program was formulated. There was concern among consulting companies that provide transportation planning and traffic analysis services in North Dakota that this program would compete with their business. However, once the program's activities and benefits were outlined, these concerns were greatly reduced. Further, the program steering committee provided a leadership role to stand behind the program and assure consultants that their business would not be affected. The key to selling the program was to emphasize ATAC's role in model enhancements that ultimately benefit consultants who need model results to support their work.

4. *Reconciling differences between MPO's and academic environments*. MPOs must meet strict planning requirements and follow a structured process for approving their plans. This creates pressures on the MPOs to meet deadlines and get approval from several layers of advisory bodies. In contrast, academic research staff work in a less structured environment, where there may be high emphasis on long-term model improvements vs. the immediate needs to meet requirements. This issue is best addressed by continuous dialogue among project partners in order to balance desired improvements with the real-world planning needs of the MPOs and the DOT.

CONCLUSIONS

Smaller MPOs face significant challenges in maintaining effective and up-to-date transportation planning models. Constrained budgets, modest staffing levels, and limited expertise create a difficult environment for developing, supporting, and enhancing planning models. In this paper we described an innovative program that meets the metropolitan modeling demands in three MPOs and NDDOT through a strong partnership with a university. The program facilitates resource pooling, knowledge sharing, and ensuring consistency across the state. It also provides mechanisms to support various analysis needs involving private consulting firms. Over the last three years, this program has demonstrated great success by yielding substantial model improvements, providing training opportunities, improving the timeliness of analyses, and involving graduate and undergraduate students.

It is important to remember that the success of this program did not come without some difficulty. One of the initial drawbacks involved consultants viewing the arrangement as competition. Another difficulty focused on bridging the thinking gap between MPOs, the DOT, and the university. However, both of these concerns were alleviated by developing a clear business plan in concert with potential program partners. Perhaps this may be the best recommendation to other states interested in developing similar programs.

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The Inherent Challenges of Securing Transportation Infrastructure: Examination of the National Capital Region

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ABSTRACT

Surface transportation systems are a critical element of virtually every aspect of life in the United States. They support national defense, move people and goods, employ millions of people, generate revenue and consume resources and services generated by other sectors of the economy. The terrorist attacks of September 11th had a significant impact on the United States, not only in terms of loss of life and property, negative economic impacts, but also with the unsettling realization that an inherently free society renders critical infrastructure in the United States susceptible to future attacks.

This paper examines the inherent challenges that transportation service providers and public safety agencies face in securing transportation infrastructure in the National Capital Region (NCR). In addition, this paper examines various planning, development, and operational activities that have been executed in the NCR to foster more secure transportation systems. This paper concentrates on highway networks, public transportation systems (including rail and buses), railways, and airport ground access networks.

Transportation infrastructure security is approached in this paper from the perspective that ensuring transportation infrastructures is not merely a process ensuring that an event such as a malevolent attack does not occur. Rather, the process also entails developing and implementing strategies and measures that specifically focus on responding to and recovering from the loss of an asset.

Key words: National Capital Region—security challenges—transportation security

BACKGROUND

Surface transportation systems are a critical element of virtually every aspect of life in the United States. They support national defense, move people and goods, employ millions of people, generate revenue, and consume resources and services generated by other sectors of the economy. In 2003, the transportation sector of the U.S. economy contributed $1.150 trillion to an $11 trillion gross domestic product (GDP) (BTS 2001). This is not surprising, considering that the United States is one of the most mobile nations in the world, accommodating over 3 trillion miles of passenger travel annually on highways and public transit systems (FHWA 2005). In 2003, there were also an estimated 196 million licensed drivers and over 134 million registered vehicles (private and commercial) in the United States (FHWA 2003). Further, public transit users made more than 9 billion unlinked trips using the more than 6,000 transit properties in 2003 (APTA 2005).

The terrorist attacks of September 11, 2001 had a significant impact on the United States, not only in terms of loss of life and property and negative economic impacts, but also with the unsettling realization that an inherently free society renders critical infrastructure in the United States susceptible to future attacks. Asa Hutchinson, Undersecretary for the Border and Transportation Security Department of Homeland Security (DHS), concludes that "whenever we are in a free society there will always be vulnerabilities that you can point to. That is an absolute rule of American Freedom" (Hutchinson 2003).

This paper examines the inherent challenges that transportation service providers and public safety agencies face in securing transportation infrastructure in the National Capital Region (NCR).[1] In addition, this research examines various planning, development, and operational activities that have been executed in the NCR to foster more secure transportation systems. This paper concentrates on highway networks, public transportation systems (including rail and buses), railways, airport ground access networks, and postal and shipping systems. This paper is based on work conducted as part of the National Capital Region Critical Infrastructure Vulnerability Assessment Project, executed through the National Center for Technology and Law at the George Mason University School of Law, under grants from the Urban Area Security Initiative and the Department of Justice Community-Oriented Policing Program.

National Capital Region Transportation Systems

The NCR covers more than 3,000 square miles in the Commonwealth of Virginia, State of Maryland, and the District of Columbia and is currently home to some 4.2 million people and 2.7 million jobs. Supporting the movement of people and goods in the NCR is a vast transportation network that includes 14,100 lane miles of highways, more than 200 miles of carpool lanes, 103 miles of metrorail, and 162 additional miles of commuter rail. The system also includes an extensive bus network or local and commuter services, as well as three major airports: Ronald Reagan Washington National, Dulles International, and Baltimore/Washington International, and more than 10 general aviation airports (NCRTPB). Figure 1 illustrates the complex highway network in the NCR (from a Rand-McNally map).

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[1] The NCR is defined in the United States Code [40 USC 71 (b)] as the District of Columbia; Montgomery and Prince Georges Counties in Maryland; Arlington, Fairfax, Loudoun, and Prince William Counties in Virginia; and all cities existing in Maryland or Virginia within the geographic area designated by the outer boundaries of the combined counties listed. For consistency with the Metropolitan Washington Council of Governments (WashCOG) this research expands the definition of the NCR to include Frederick County in Maryland.

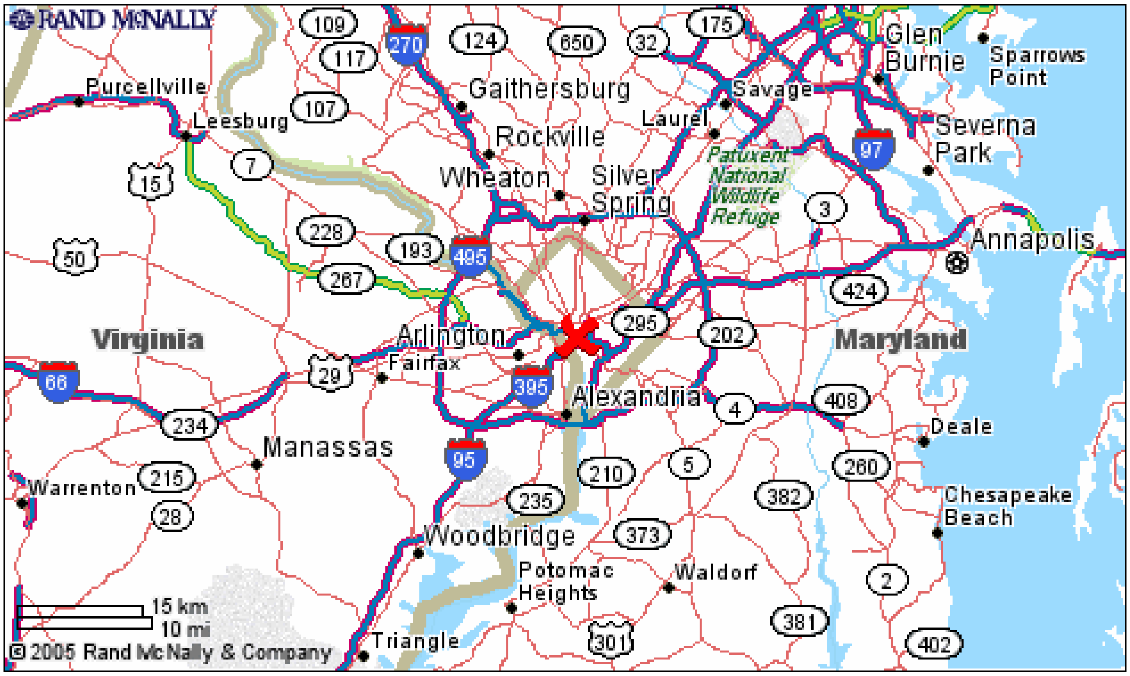

Figure 1. National Capital Region highway network

NCR Transportation Systems

The definition of the transportation systems that are a focus of this paper are in part based on *Uniting and Strengthening America by Providing Appropriate Tools Required to Intercept and Obstruct Terrorism Act (USA PATRIOT Act) of 2001* and the *National Infrastructure Protection Plan, The Physical Protection of Critical Infrastructures and Key Assets, February 2003.* For the purposes of this paper, each mode within the transportation sector is composed of fixed infrastructure, such as facilities, roadways, information systems, and voice and data communications systems; vehicles and conveyance systems; and human capital at the state, local, and federal levels of government and civilian employees responsible for the development, operation, and management of transportation systems. Specific modes of transportation considered in this paper include the following:

- *Highway system.* This includes the physical facilities themselves, including roadways, bridges and tunnels, inter-modal terminals, maintenance facilities, vehicles (private and commercial motor carriers) operating on the system, and the control and information infrastructure that monitors and manages the flow of goods, vehicles, and people on the highway system (SAIC 2002; *Physical Protection* 2003).
- *Transit system*. This includes bus, rail, or other conveyance, either publicly or privately owned, that provide the public general or special service (but not including school buses or charter or sightseeing services) on a regular and continuing basis. System components include all of the vehicles, equipment, right-of-way, routes, support equipment and facilities, and buildings and real estate belonging to or operated by the public transportation authority (US DOT 2003).
- *Rail system*. Rail system infrastructure management elements include track maintenance throughout the NCR and the vehicles that operate on and adjacent to these facilities.
- *Airport ground access systems*. All of the vehicles, equipment, right-of-way, routes, support equipment and facilities, and buildings and real estate that provide ground access to Washington Reagan National Airport, Dulles International Airport, and Baltimore-Washington International

Airport. This paper does not address terminal and airfield operations and their related security challenges.

Dimensions of Transportation Infrastructure Security

Transportation infrastructures present three unique dimensions to the challenging task of securing transportation infrastructure. First, transportation infrastructure must be secured and protected from malevolent attacks that destroy, damage, or otherwise reduce the level of service provided by the given system as they, as mentioned, support national defense, move people and goods, employ millions of people, generate revenue, and consume resources and services generated by other sectors of the economy. It is also important to note that in order to ensure that transportation infrastructure is afforded the highest level of security possible other events, such as natural disasters and major incidents that are not man-made, must also be considered. Second, transportation systems themselves can be a vehicle or method of delivery for a malevolent attack. Although many examples can be sited to illustrate this, perhaps no event illustrates this challenge better than the terrorist attacks of September 11, 2001, when commercial airliners were used to carry out attacks. Third, the interdependencies among multiple infrastructure sectors necessitate that transportation security activities consider many things outside the transportation domain. As an example, the emergency service sector (police, fire, and emergency medical services) is dependent on the transportation sector to transport emergency resources and patients. Without a functioning transportation infrastructure, these functions may be impossible.

FEMA Disaster Lifecycle

Given the varying dimensions of transportation infrastructure security challenges, this paper is approached from the perspective that ensuring transportation infrastructures is not merely a process ensuring that an event such as a malevolent attack does not occur. Rather the process also entails developing and implementing strategies and measures that specifically focus on responding to and recovering from the loss of an asset. This is necessary, as it is understood that this collection of infrastructures are vulnerable and quite susceptible to failures due to their accessibility; the expansive and open natures of these infrastructures contribute to their vulnerability. Accordingly, to provide structure to the discussion and analysis, this paper presents issues in the context of what is commonly referred to as the Federal Emergency Management Administration (FEMA) disaster life-cycle. The FEMA disaster life-cycle is a series of activities that a region engages in to prepare for and respond to emergencies and disasters, help people and public and private institutions recover from emergencies and disasters and the effects of the emergency or disaster, reduce the risk of loss, and help prevent disasters and emergencies from occurring (FEMA 2003). Although variations of the disaster life-cycle have been used by others, for the purposes of this paper, the process includes the following:

- Response. This involves the mobilization and positioning of emergency equipment and personnel to get the public out of danger; bring the impacts of the disaster under control; provide basic necessities, including food, water, shelter, and medical services to those impacted by the disaster; and bring transportation systems and services back on line as soon as possible.
- Recovery. This involves repairing and/or replacing transportation infrastructure and vehicles in order to return them to their pre-disaster levels of service.
- Risk reduction and prevention. This involves reducing the probability of an attack on transportation infrastructure and mitigating the potential impacts should an attack occur.
- Planning and preparedness. This ensures that if a disaster occurs, transportation and public safety agencies, among others, are ready to respond in the safest and most efficient manner while ensuring the safety of the general public.

These functions collectively represent the processes and activities necessary to secure critical infrastructure. Figure 2 illustrates the disaster lifecycle as it relates to general tasks that would likely be undertaken in securing transportation infrastructure.

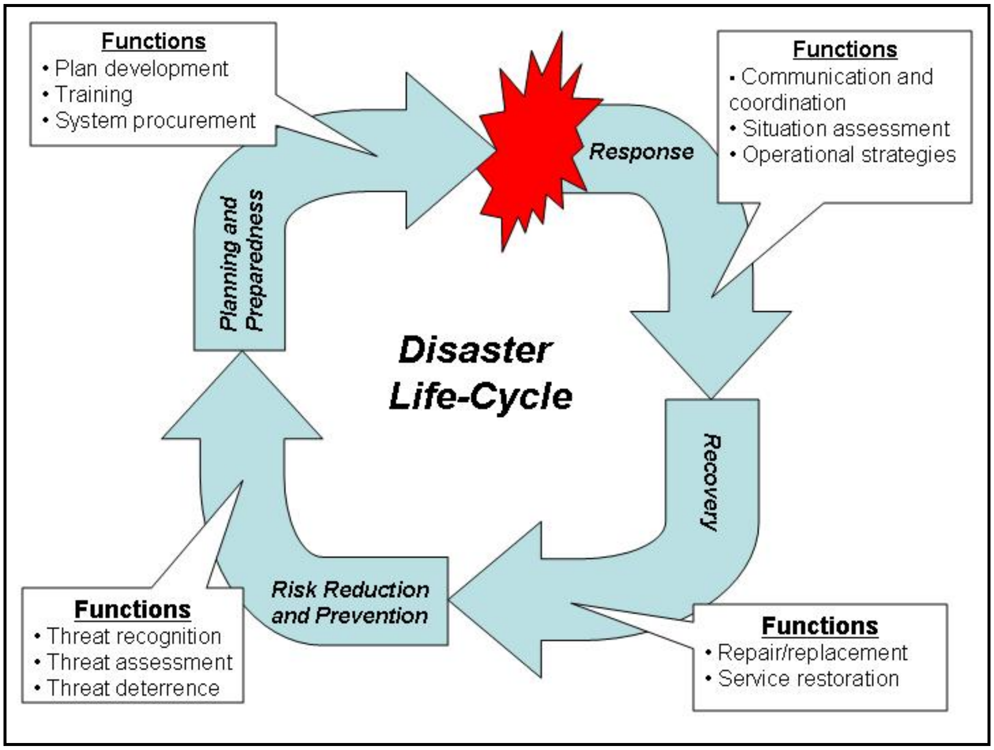

Figure 2. FEMA disaster life-cycle

INHERENT CHALLENGES FROM A MODAL PERSPECTIVE

Highway

Quite possibly the most significant factors underlying the challenges associated with securing highway infrastructure, including bridges and tunnels, is that by design they are easily accessible to almost anyone and they transverse relatively long distances. Consequently, it is many times not feasible, or possible, to secure the entire highway network, whether through strategies such as surveillance, law enforcement patrols, or infrastructure hardening. Closely related to the inefficiencies of providing surveillance or monitoring of the entire highway network is the threat that commercial vehicles can possibly be targets of hijacking and consequently used as a means for carrying out malevolent attacks.

It is understood that, from an operational perspective, highway systems in the NCR are critical in moving people and goods, not only within the region, but also through it to other destinations nationally and internationally. Highway networks in the NCR are not necessarily susceptible to a single point of failure. For example, if a bridge crossing the Potomac River were to be lost, there are alternate means to move people and goods across the river. Loss of an asset such as a bridge crossing the Potomac River for any length of time, however, would have significant impacts on regional mobility. Loss of such an asset for an extended period of time would ultimately impact the economic vitality of the region. These impacts partially result from the fact that the NCR already suffers from the some of the worst congestion in the United States, even when transportation systems are operating under normal conditions.

Also from an operational perspective, and closely related to the issue of limited capacity, are the challenges associated with evacuating the region in the event of an emergency using the existing highway system. As was demonstrated in the NCR following the terrorist attacks of September 11, 2001, rapidly moving masses of people out of the central city in a limited amount of time greatly compounds the mobility challenges of normal peak-hour flows. This challenge could be made exponentially worse if a highway asset such as a bridge or tunnel were lost during an attack. Ultimately, the inability to move people out of an urban core can compromise personal safety.

Planning for measures to prevent, respond to, and recover from transportation emergencies also presents a variety of challenges. Planning must be approached from the perspective that the systems work collectively to provide regional mobility. This can be especially challenging when multiple jurisdictions and service providers are involved. Consequently, the NCR provides an environment that too often exacerbates the inherent turf issues associated with regional planning activities. This is critical, in that if highway networks are to provide the safest and most efficient movement of people and goods, both during emergencies and during normal operations, the network must be planned and operated from a regional perspective.

Transit

History has also taught that transit facilities are attractive targets of malevolent attacks throughout the world. Between 1997 and 2000, more than 200 terrorist attacks were carried out worldwide on transit systems. These attacks involved, among other things, hijacked vehicles, chemical attacks, bombs, poison gas, guns, and grenades (Gersten and Jenkins 2001).

These threats are of particular concern because transit systems in the NCR provide similar opportunities for significant death and destruction, as they generate large volumes and densities of travelers with quite predictable patterns. In 2002, the Washington Metropolitan Transit Authority (WMATA) accommodated a combined total of 328.7 million bus and rail transit trips in the NCR. Factors that make the Metrorail specifically an attractive target of malevolent attacks include the following:

- There is relatively little redundancy in the system of tracks, which makes maneuvering trains difficult.
- A back-up operations center does not exist.
- Local transit service providers depend on WMATA to provide regional connectivity.
- To maintain operational efficiencies, it is impossible to screen passengers in a manner comparable to that of commercial air travel.

With respect to WMATA's Metrorail system, biological or chemical agents could be spread to many, if not all, of the system's 86 stations before being detected. Consequently, large portions of the system would be rendered unusable for long periods of time while cleanup activities are being conducted, dramatically extending recovery times. This could be catastrophic from a regional mobility perspective, since more than 40% of the work trips to the urban core of the NCR are made via public transit (WMATA).

Rail

The NCR has rail lines that pass directly through the center of the region. Consequently, there is significant concern with respect to the transportation of large amounts of hazardous materials. The primary concern is that significant numbers of injuries and loss of life could result should hazardous materials be released as a result of an accident or malevolent attack. A loss of a bridge or a tunnel that supports rail operations could also have long-term adverse impacts on the economy, through, among other things, disruptions in supply chains and infrastructure repair and replacement costs.

With this issue in mind, on February 1, 2005 the District of Columbia City Council approved emergency legislation restricting the shipment of hazardous materials within two miles of the U.S. Capitol and federal buildings. The legislation requires rail and trucking companies to receive a special permit before they can transport large quantities of hazardous chemicals through the District of Columbia. C-S-X is suing the city and asking a federal judge to strike down its ban on hazardous train shipments. Although the issue has yet to be resolved, it has the potential to impact the transportation of hazardous materials in other regions. Of particular concern with respect to this issue are the potential impacts that such restrictions could have on interstate trade.

Airport

Ground access to Dulles International Airport and Ronald Reagan Washington National Airport has been significantly impacted since the attacks of September 11th. Measures implemented to enhance security, such as prohibiting motorists to park on the terminal drives, has impacted the flow of traffic. Both airports have been forced to implement strategies, such as barrier walls, that have also disturbed the operation of the airport approaches. Accordingly, the challenges presented by peak traffic volumes are exacerbated when the given measure is put in place. Furthermore, given the unrestricted access to the airports, very little can be done to prevent a malevolent attack on a terminal, parking structure, or roadway network under current operating conditions, regardless of measures that have been put in place.

OTHER TRANSPORTATION SECURITY CONSIDERATIONS

Transportation Stakeholders and Interdependencies

The diverse and complex composition of transportation stakeholders in the NCR in itself presents a variety of challenges that can affect transportation security. The inherent multi-modal nature of the services provided and the sizable number of jurisdictions in the NCR creates an environment that necessitates a large number of stakeholders who are all in some way responsible for the planning, development, operations, management, and security of the various systems. Bringing multiple stakeholders together to achieve a common goal can be challenging, especially when transportation agencies and service providers must consider the needs and interests of the entire region first, and their individual needs and interests second. Adding to the complexity is that operating agencies at the federal, state, and local levels, as well as in the private sector, all play integral roles in the delivery of services.

As mentioned, it also important to understand the interdependencies of other infrastructure sectors. Figure 3 illustrates some of the dependencies that other sectors have on the transportation sector, and vice-versa. As an example of interdependencies, the safe and efficient operation of some transportation systems (highway and transit) is not possible without the services of other critical infrastructures. As an example, if traffic signals are not equipped with uninterruptible powers supplies (UPS), traffic signals are rendered inoperable when power to the signals is lost. Loss of power can also render some or all rail operations inoperable. Each of these sectoral dependencies, among others, illustrated in Figure 3 are underpinned by a complex set of institutional and operational arrangements that have been developed to maximize the efficiency and reliability of each of the sectors.

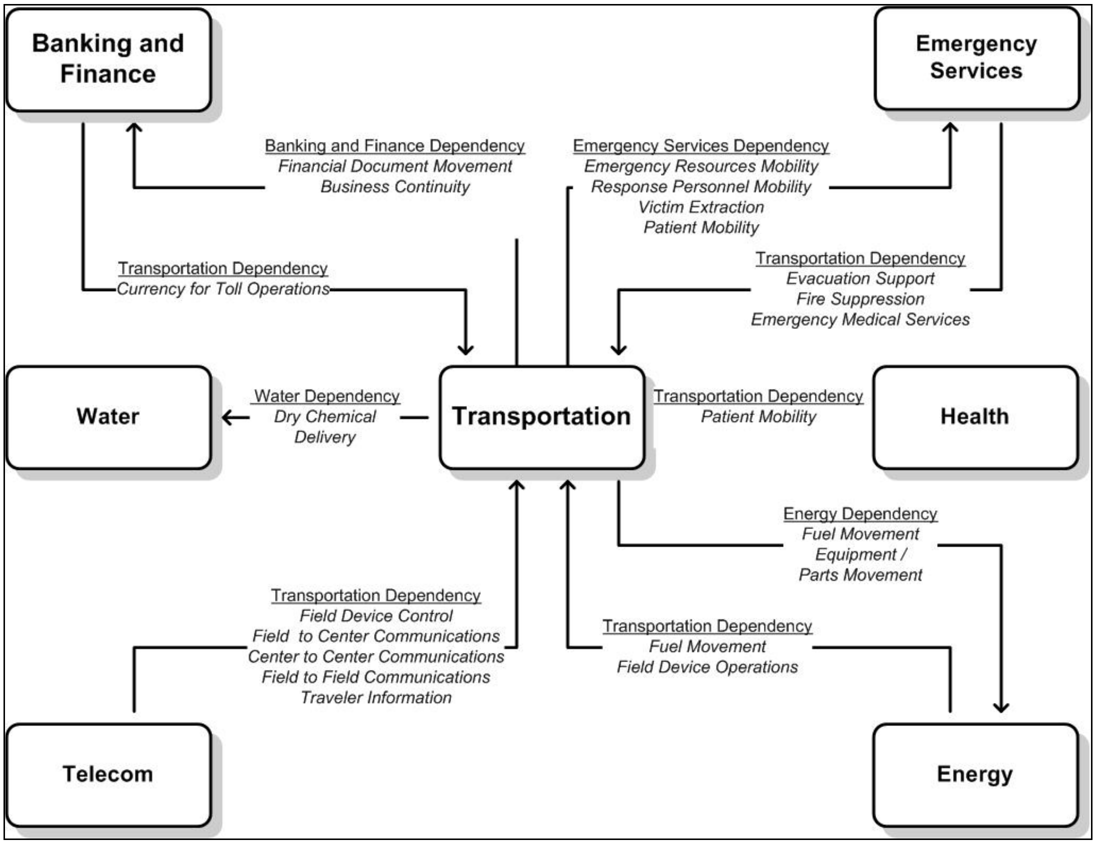

Figure 3. Transportation sector centric dependencies

Lack of Voice Communications

Generally, from an operational perspective, at the local level there is also a chronic lack of voice and data communications systems interoperability among responding agencies from various disciplines and jurisdictions. A study sponsored by the National Institute of Justice, National Task Force on Interoperability concluded that a variety of factors are impediments to interoperability of voice and data communications systems, including the following:

- Communications equipment is aging and incompatible. Adjacent jurisdictions often use different types of radio systems and different frequencies.
- Funding is limited and fragmented. Resources for replacement of voice and data communications systems are limited at every level of government.
- Planning is limited and fragmented. Planning activities are typically underfunded and are rarely conducted in the context of the region as a whole.
- There is a lack of coordination and cooperation. Agencies are often reluctant, if not unwilling, to give up management and control of their own voice and data communications systems at any level of planning, development, implementation, or operation.

Additionally, attacks on transportation infrastructure involving weapons of mass destruction pose even more complicated response challenges. Responding to such attacks requires a combination of trained personnel, specialized equipment, well-defined procedures, and supplies. The challenge lies not only in having access to these resources, but having access to a sufficient number of these resources. For example, an attack on a WMATA Metrorail that could literally affect thousands of people at one time would likely be overwhelming for all of the involved response agencies.

OVECOMING THE CHALLENGES IN THE NATIONAL CAPITAL REGION

The inherent challenges of securing transportation infrastructure facing federal, state, and local agencies are unlike any that the United States has ever faced. The threats have no boundaries, either jurisdictionally or in terms of disciplines they affect. To secure transportation infrastructure adequately, stakeholders in the NCR have begun to embrace new measures to enhance security. Described below are descriptions of such initiatives that have been executed in the NCR.

Department of Homeland Security, Office of National Capitol Region Coordination

Recognizing that the federal government was not organized, staffed, nor adequately prepared to meet the changing demands of the 21st century (including preventing terrorist attacks and securing transportation infrastructure), President George W. Bush proposed draft legislation to Congress to create the Department of Homeland Security (DHS) on June 18, 2002. The passage of the legislation and the subsequent creation of the DHS represented the largest reorganization of the federal government since the outbreak of the Cold War, when the National Security Act of 1947 unified the Armed Forces under a single department and created the National Security Council and the Central Intelligence Agency.

With respect to influence on the NCR, the Office of National Capitol Region Coordination (ONCRC), located within the DHS, has been charged with overseeing and coordinating federal programs and domestic preparedness initiatives for state, local, and regional authorities in the NCR. ONCRC has been an instrumental player in the many initiatives that have been carried out in the NCR by the Metropolitan Washington Council of Governments (WashCOG) and its member agencies, including the development of the Regional Emergency Coordination Plan and its ancillary components that are described below.

Regional Emergency Coordination Plan

Initial evaluations of WashCOG’s security planning following the attacks of September 11th indicated that the components of the regional transportation system preformed fairly well individually (Wegmann). Collectively, however, at the regional level coordinating among the various transportation service providers fell short. Consequently, the Transportation Planning Board (TPB) was charged with developing a coordinated emergency response plan.

In response to this challenge, the TPB developed the Regional Emergency Coordination Plan (RECP) to provide a vehicle for collaborating in planning, communication, information sharing, and coordination activities before, during, or after a regional emergency for the 17 WashCOG member governments, the State of Maryland, the Commonwealth of Virginia, the federal government, public agencies, private sector and volunteer organizations, and local schools and universities.

Regional Integrated Communication and Coordination

The Regional Integrated Communication and Coordination Systems (RICCS) provides a system for WashCOG members, the State of Maryland, the Commonwealth of Virginia, the federal government, public agencies, private sector and volunteer organizations, and schools and universities to collaborate in planning, communication, information sharing, and coordination activities before, during, and after a regional incident or regional emergency. RICCS utilizes a variety of new and existing communication devices to foster collaboration that will be critical in the event of an emergency that has regional impacts.

Regional Emergency Evacuation Transportation Coordination

The Regional Emergency Evacuation Transportation Coordination (REETC) is intended to address the transportation aspects of moving people around or out of the region and moving required resources into the area in anticipation of and following a regional incident or emergency that requires evacuation. Therefore, REETC addresses coordination of demand management, identifying situations and strategies where the majority of people do not evacuate the area, but shelter in place to ensure that transportation system capacity is available for those who truly need it.

Capital Wireless Integrated Network

The Capital Wireless Integrated Network (CapWIN) project is a partnership between transportation and public safety agencies in the State of Maryland, the Commonwealth of Virginia, and the District of Columbia. The focus of this initiative is to develop an integrated transportation and criminal justice information wireless network. CapWIN will integrate transportation and public safety data and voice communication systems in the two states and the District of Columbia and will be the first regional transportation and integrated wireless network for public safety in the United States. CapWIN will enable critical and time-sensitive data to reach responders in other jurisdictions throughout the region.

CapCOM

In late 2004 TPB requested $5 million from the United States Congress to develop CapCOM, which may best be characterized as an operations center that will be implemented to foster regional operations coordination for surface transportation systems. This new initiative is envisioned to function similarly to TransCom in the New York/New Jersey/Connecticut tri-state area and provide a governance structure for regional transportation operations during normal and emergency operations. TransCom was widely regarded as essential in evacuating Manhattan during the attacks on the World Trade Center.

The primary motivations for initiating this effort stem from the absence of one single entity that is in charge of coordinating regional operations in real-time and managing incidents that have a regional impact. Managing such incidents has become a priority for transportation and public safety agencies in the region. The two primary functions of CapCom will be regional operations support and regional planning and preparedness. CapCOM development efforts in 2005 have focused so far on developing the supporting governance structures.

PRELIMINARY FINDINGS

Transportation systems in the NCR are critical in supporting the movement of people and goods. A loss of any number of transportation assets or a prolonged reduction in the level of service provided by the systems would likely result in serious economic impacts to the region. Consequently, it is critical that transportation systems are provided with the highest level of security possible to protect them from natural disasters, major incidents, and malevolent attacks.

To secure transportation infrastructure in the NCR, it will be necessary for the regional partners to overcome the inherent challenges associated with this phase of the process. Furthermore, it is becoming increasingly important to continue the shift of planning, operations and management, and systems procurement activities from a system that has a localized focus to one that considers such activities in the context of the entire region. The NCR has made significant progress with respect to this challenge as demonstrated in the development of the Regional Emergency Coordination Plan, the Regional Information Communication and Coordination Systems, CapWIN, and CapCOM.

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Surface Transportation Weather Issues for the 21st Century

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ABSTRACT

Transportation agencies in the United States are facing financial and staffing difficulties while the motoring public is asking for improved, year-round mobility and increased safety. At the same time, some agencies are facing extreme environmental problems caused by highway maintenance operations, especially from chemicals used in winter maintenance operations. Since the political outlook for increased taxes to fund highway maintenance is bleak, the only option available for public agencies to solve these problems is to use more efficient and effective methods to accomplish maintenance. Cutting edge improvements for surface transportation weather, now on the threshold of implementation, offer unlimited opportunities for safety, mobility, and operational improvements.

This paper will summarize technological advances and report on the progress in three major areas of surface transportation weather and show how they are anticipated to improve the safety and mobility of the nation's roadways and the productivity of operating agencies. The following topics will be discussed:

- Improved Road Weather Information Systems (RWIS)
- Aurora Consortium Project 7.9, Temperature Sensor Accuracy
- NCHRP Project 6-15, Testing and Calibration Methods for RWIS Standards
- FHWA and AASHTO SICOP Project, Development of RWIS Environmental Sensor Station Siting Guidelines
- Establishment of a national road weather information observation system
- FHWA Project Clarus—the Nationwide Surface Transportation Weather Observing and Forecasting System
- Recommendations by the National Academy of Sciences in their 2004 report, "Where the Weather Meets the Road: A Research Agenda for Improving Road Weather Services"
- Implementation of a winter maintenance decision support system (MDSS)
- An FHWA MDSS Project that integrates state-of-the-art weather forecasting, data fusion, and optimizing techniques with computerized winter road maintenance rules-of-practice logic. The result is guidance aimed at maintenance managers that provides specific forecast of surface conditions and treatment recommendations customized for individual plow routes.
- Field evaluations showing that MDSS optimizes the use of snow and ice control chemicals and thus reduces their environmental impact and increases their effectiveness.

Key words: decision support—road weather conditions—weather observation

INTRODUCTION

Safe, reliable, all-weather, every-season mobility continues to be the demand of the motoring public. The increasing expectations of the motoring public are a major challenge to transportation agencies, now facing financial and staffing difficulties. Meeting these demands poses unique problems during inclement weather. Rain, fog, snow, and ice significantly affect the safety and capacity of a roadway. However, quality road weather observations and forecasts specific to the roadway environment have the potential to help users, including drivers, maintenance personnel, fleet dispatchers, enforcement officers, traffic managers, and emergency personnel, make better decisions, thus increasing travel efficiency and safety during adverse weather conditions. Cutting edge improvements for surface transportation weather, now on the threshold of implementation, integrated with enhanced decision making techniques, offer unlimited opportunities for safety, mobility, and operational improvements.

SURFACE TRANSPORTATION WEATHER OBSERVATIONS

Surface weather is different from upper atmospheric weather because of ground interaction. This results in the information provided by a traditional forecast often being inadequate for effective surface transportation-related decision making. The reason is air, humidity, precipitation, pavement orientation, surface and subsurface temperatures, and traffic all come together at the pavement surface to create an environment that can be much different than that occurring a few feet above the ground. The foundational investigative work for road surface weather took place in 1988 in the Strategic Highway Research Program (SHRP) and was published in 1993 (Boselly et al. 1993). The road weather information systems (RWIS) were installed generally using manufacturer’s recommendations and local maintenance supervisor knowledge. Often these RWIS sites were installed in problem areas (high-accident, coldest spots, frost-prone areas, etc.) and did not represent regional road weather conditions. Little was known about testing and calibration of these systems or the accuracy of the sensors.

Figure 1 shows a typical RWIS environmental sensor station (ESS). RWIS consists of the hardware, software, and communications interfaces necessary to collect and transfer road weather observations from the roadway to a display device at a user’s location. While the original purpose of RWIS was to address winter weather conditions, applications have been developed to detect and monitor a variety of road weather conditions impacting road operations and maintenance, thus providing opportunities to serve a wider array of users.

Improved road weather information systems (RWIS)

RWIS Siting Guidelines

The Federal Highway Administration (FHWA), recognizing the need for a national uniform identification of RWIS, published “Road Weather Information System Environmental Sensor Station Siting Guidelines” in April 2005 to provide guidance for siting an RWIS ESS and its associated environmental and pavement sensors (FHWA 2005). The guidelines provide siting criteria that satisfy as many road weather monitoring, detection, and prediction requirements as possible.

Most state DOTs do not have a detailed inventory of their RWIS ESS system, so a national uniform inventory will need to be undertaken. The FHWA guidelines provide the desired elements of this inventory and emphasize the importance of documenting each site’s metadata. Metadata are defined as data about data. Metadata should consist of information about the site location; elevation; topography; soil; surrounding vegetation; relationship to the roadway surface; exposure of the site to wind, sun, moisture sources, and artificial temperature; accuracy of sensors; measurement range of sensors; sampling time and intervals; and an indicator of the quality of the exposure of the sensors. (Note that these indicators have not yet been developed). Once a national inventory is completed, the RWIS ESS data will be useable by the wider transportation and meteorological communities.

RWIS Temperature Sensor Accuracy

The Aurora Program, a consortium of agencies focused on collaborative research, evaluation, and deployment of advanced technologies for detailed road weather monitoring and forecasting, recognizing the need for accurate sensor data, completed a study entitled “Pavement Temperature Sensor Accuracy.” The project determined the accuracy and variation in readings of various pavement temperature sensors, both in-pavement and mobile, by first developing a method to determine true pavement temperature for comparison purposes by the following processes:

- Compared sensor readings in a controlled laboratory environment
- Compared sensor readings in an operational environment
- Compared sensor readings under various temperature and weather conditions
- Compared the effects of commonly used road deicing and anti-icing chemicals on sensor readings
- Compared the effects of traffic on sensor readings

Summary results show that overall the temperature sensors reported surface temperatures within 0.8° C of the actual pavement surface temperature. Also, the application of sodium chloride to the sensors had an insignificant impact on sensor temperature performance. Solar impact was difficult to reproduce in the laboratory environment, so it was listed as suggested research into the effects of radiational cooling and solar heating for a better understanding of RWIS sensors. The final report outlining how well various sensors worked and their conditions is posted on the Aurora website at http://www.aurora-program.org.

Testing and Calibration Methods for RWIS

The National Cooperative Highway Research Program (NCHRP), recognizing that most agencies either rely on vendor-developed testing and calibration methods or simply accept sensor data without verification or regular and timely calibration, is developing guidance for the practical testing of RWIS ESS to ensure that sensors are providing an accurate representation of actual site conditions. A contract was awarded in June 2003 to develop concise guidelines of best practices for RWIS ESS testing and calibration methods in field deployments. The final report, complete with guidelines that can be distributed as a stand-alone document for use by RWIS end-user organizations, will be available in the fall of 2005.

ESTABLISHMENT OF A NATIONAL ROAD WEATHER INFORMATION OBSERVATION SYSTEM

National Research Agenda for Improving Road Weather Services

In the past few years, weather-related transportation issues have become a priority for the national research agenda. The FHWA funded the Board on Atmospheric Sciences and Climate (BASC) of the National Research Council to examine the research that needs to be done and the technology transfer that should be accomplished to improve the production and delivery of weather and road weather (road weather is the micro-climate at the road's surface) information for the nation's roadways. The results of that study were published in an April 2004 report entitled *Where the Weather Meets the Road: A Research Agenda for Improving Road Weather Services* (NRC 2004). The report concluded that forecasting and handling road weather is a highly interdisciplinary problem, spanning micrometeorology, numerical weather prediction, vehicle technology, meteorological and pavement instrumentation, roadway construction and maintenance, human factors, and technology transfer. The report is available at the National Academies website: http://www7.nationalacademies.org/basc/publications.html.

At about the same time in 2003, the American Meteorological Society (AMS) decided to hold a forum to address various issues connected with the effective use of road weather information. The AMS Atmospheric Policy Program organized a "Weather and Highways Forum" and invited nearly 100 public and private transportation managers and users, representatives of weather information providers, academia, and policy makers knowledgeable about the nation's highway system, to participate in a two-day forum in Washington, DC. More information is available at the AMS website at http://www.ametsoc.org/atmospolicy/2003transportationforum.html.

Both the BASC report and the AMS policy forum put forth a suite of research activities and other efforts to foster the implementation of an improved road weather program. The AMS "Weather and Highways" report (AMS 2004) sets forth a concise six-point program as follows:

1. Congress should authorize and provide long-term funding for the appropriate federal agencies to develop a national road weather research, development, and applications program to improve the application of weather information for highway safety and operations.
2. The federal and state departments of transportation should closely coordinate with public, private, and academic sector road weather stakeholders to improve the safety and efficiency of the nation's highway system during adverse weather.
3. The US DOT/FHWA and the National Oceanic and Atmospheric Administration (NOAA), working with state DOTs, should establish a national road weather and road condition data collection, processing, and dissemination infrastructure to improve the safety and efficiency of the roadway system.
4. NOAA/NWS, commercial weather providers, and weather information users should work cooperatively to improve the observation system, develop and improve forecasts, and enhance the delivery of information and services on road weather.
5. Federal and state DOTs should train the road management community to integrate weather into the decision process more effectively. In addition, the atmospheric science community, particularly academia, should develop course curricula focusing on road weather science and engineering.
6. The US DOT/FHWA should provide incentives for vehicle manufacturers and highway engineers to raise public and private sector demand for in-vehicle road weather information.

Establishment of a National Road Weather Observation System

Following recommendations 3 and 4 listed above in the BASC report and the AMS policy forum, FHWA developed a project called the Clarus Initiative, which establishes a vision for leveraging local and regional road and rail weather observations to serve a greater community and enhance 21st century transportation operations. This vision will be accomplished through the design, demonstration, and deployment of a national surface transportation weather data collection and management system that complements the existing National Weather Observation System with RWIS ESS and other surface observations. Surface transportation-based weather observations integrated with the existing National Weather Observation System data will provide broader support for surface transportation-specific models to predict impacts on surface transportation operations, safety, and mobility.

At the writing of this paper, the Clarus Initiative was finishing writing the Concept of Operations, which provides a high-level definition of how the system will work. A great deal of time has been spent on establishing and understanding of the needs of various stakeholders and how the Clarus System can be structured to meet user requirements. Functional scenarios are being developed for the many market segments that will be served by the system. Stakeholders and the consultant are involved in preparing a narrative text with an illustrated Use-Case Diagram and a Sequence Diagram addressing the typical concepts anticipated to exist in the application of Clarus System data.

The current Clarus scheduling lists System Design as completed in fall 2006, Multi-state Regional Demonstration in FY 2007 and 2008, and Field Design and Model Deployment in 2009.

IMPLEMENTATION OF A WINTER MAINTENANCE DECISION SUPPORT SYSTEM

Background

In 2000, the Office of Federal Coordinator for Meteorology and the FHWA Road Weather Management Program, in an effort to capture surface transportation weather requirements and unmet user need, cosponsored symposiums on weather information for surface transportation. The author of this paper participated in those symposiums. The symposiums brought together users and providers in an effort to capture surface transportation weather requirements and identify unmet user needs. An unmistakable message emerged that users were not satisfied with the current surface transportation weather capabilities and that much more could be done to address their needs. Although weather forecasts were plentiful and a few private service providers issued road-specific forecasts, there was a lack of linkage between the information available and the decisions being made by winter maintenance managers. Thus, the FHWA Road Weather Management Program decided to address some of the unmet weather needs by launching the Maintenance Decision Support System (MDSS) project.

During the past four years, MDSS has evolved into a functional prototype system. A prototype MDSS was deployed at several maintenance garages in central Iowa in winter 2002–2003. During the winter of 2003–2004 a second, more comprehensive field demonstration was conducted. As one would expect, the performance of the prototype MDSS was much improved during the second winter of use, but still not sufficiently mature to convince the private sector that the system was ready to market or convince the public sector to require MDSS as part of their forecasting specifications. A third field demonstration was held in Iowa during the winter of 2004–2005 and a first demonstration was held in the more difficult terrain and climate of Colorado.

Project Resources and Organization

The MDSS research project is funded and administered by the FHWA Road Weather Management Program. The project builds upon the work accomplished in the SHRP and the Test and Evaluation Project TE-28 that FHWA, the American Association of State Highway and Transportation Officials (AASHTO), and the Transportation Research Board (TRB) completed in the mid-1990s. A consortium of five national laboratories in coordination with state DOTs, academia, and the private sector have been participating in the development and field evaluations of the project. These laboratories include the following:

- Cold Regions Research and Engineering Laboratory (CRREL)
- National Center for Atmospheric Research (NCAR)
- Massachusetts Institute of Technology/Lincoln Laboratory (MIT/LL)
- NOAA Forecast Systems Laboratory (FSL)
- NOAA National Severe Storms Laboratory (NSSL)

Both CRREL and MIT/LL had participated in the SHRP and TE-28 projects, so they brought some familiarity and background to the project. CRREL brought experience in creating models for predicting road surface temperature while MIT/LL concentrated on translating the road maintenance rules of practice into computer algorithms. FSL provided high resolution weather models while NSSL contributed algorithms for determining precipitation type in weather models. NCAR was the lead laboratory providing the core data processing capability, the graphical user display, and the engineering to integrate all the parts into a working MDSS prototype.

Project Overview

The MDSS project integrates surface weather forecasting, data fusion, and optimization techniques with the computerized anti-icing techniques of the TE-28 project rules-of-practice logic. The result is a specific forecast of surface conditions and treatment recommendations customized for individual highway sections.

The MDSS project goals include the following:

- Show state DOTs that new technologies can be integrated to provide maintenance managers decision making support in improving safety and mobility on roadways while making more efficient use of chemicals, equipment, and staff.
- Convince private sector surface weather providers that there is a market for these new technologies.

FHWA-defined success for this MDSS project would be reached when private sector companies integrated MDSS components into their product lines and state DOTs were writing MDSS requirements into their purchase specifications.

Field Evaluations

Six state DOTs competed to win the opportunity to evaluate the prototype MDSS. Factors used in selecting the winning candidate were the availability of high-speed communications and computers at the maintenance garages, progressive winter maintenance programs, and a willingness of the DOT personnel to participate in training and field verification activities. The Iowa DOT met those criteria and did not have complex terrain.

The first demonstration period began February 3, 2003 and concluded April 7, 2003. A total of 15 plow routes in 3 maintenance garages around Des Moines and Ames, Iowa were selected. During that time, five light snow events (three inches or less accumulation), three heavy snow events (more than three inches), and one mixed rain/snow/ice event occurred. The demonstration was a success because the prototype was deployed and utilized and the following list of lessons learned was compiled:

- The prototype was unable to capture light precipitation events, which caught crews off guard.
- The rules-of-practice module needed additional development to handle a wider variety of weather and road condition scenarios and treatment responses.
- The availability and quality of observed real-time precipitation rate and accumulation data were very poor for snow and ice.
- The prototypes needed algorithms in the road condition treatment model (RCTM) to account for the impact of vehicle speed and volume on chemicals and the complex problem of blowing and drifting snow on the roadway.

A second demonstration was held in central Iowa from December 29, 2003 to March 24, 2004. Although the MDSS graphical user interface (GUI) was highly rated by the Iowa DOT for its ease of use and logical layout, further enhancements were made for the second demonstration. These enhancements included adding digital values to state and route view graphics, a real-time display of RWIS data, a historical window to review guidance from previous forecast cycles, and an event summary of weather and road variables for each forecast period and for each plow route. The GUI is illustrated on the MDSS web site http://www.rap.ucar.edu/projects/rdwx_mdss.

The RCTM and rules-of-practice module were significantly enhanced to recognize the overall storm situation, handle changing weather situations, and provide a blowing snow alert. Other enhancements included modifying the road temperature model to accept actual road temperature and subsurface temperatures as input, rather than relying on model-derived values.

The road weather forecast system (RWFS) was configured to utilize and integrate ten different forecast modules, and probabilistic forecast information was added. Since winter maintenance supervisors are in the business of risk management, the probabilistic prediction products is helpful. Results were that the forecasts had significantly better skill in the first six hours because of their forward error correcting capability, but still had poor-quality precipitation measurements. Also, the ability of models to predict insolation varied greatly between models, particularly in partly cloudy conditions. Since insolation measurements are critical for road temperature prediction, it was recommended that insolation measurements be added to surface observing stations and be provided in real time to weather service providers. Further enhancements to the prototype are explained in Mahoney et al. (2005).

The third year of demonstration, for the winter of 2004–2005, was held in both Iowa and Colorado. The Iowa implementation went well, since most of the problems had been worked out in the previous two years. Several of the recommended treatments were utilized without modifications and proved to be optimal solutions. Others needed only minor tuning. Any that were in serious error were so because of errors in the forecasts.

The demonstration in Colorado had difficulties. The major effort for MDSS was concentrated on the 470 toll road. As in previous demonstrations, the RWFS was configured to utilize and integrate ten different forecast modules. Unfortunately in the complex terrain, there were large spreads between the models, and it was difficult to tune a best model. Work is needed to provide a better coupling between the land surface models to the atmospheric models.

Detailed reports on the third-year demonstration will be developed during the summer of 2005. Results may be available for reporting at the 2005 Mid-Continent Transportation Research Symposium.

Estimating MDSS Success

As stated earlier, success as defined by the FHWA will be reached when private sector companies integrate MDSS components into their product lines and state DOTs purchase these new services. Guidance to state DOTs in preparing and evaluating specifications for weather forecasting services that include capabilities similar to those found in the FHWA prototype MDSS can be found in NCAR (2004). This guidance includes the following:

- Candidate functional requirements for MDSS products and services
- Information that DOT personnel can use to evaluate prospective DMSS service
- Questions that could be asked when interviewing prospective vendors

Requests for proposals for weather forecasting services are currently being developed by the Iowa DOT for a summer 2005 letting. MDSS guidance is currently listed in the Mandatory Requirements section. MDSS capability requirements are divided into three-year steps to allow time for the development of the system over the three-year contract period.

The Iowa DOT presented its success with evaluating and implementing MDSS during the past three winters at the 2005 Midwest Snow and Ice Workshop, held in Kansas City, Kansas, on April 19 and 20, 2005. Eleven Midwestern states attended this workshop. All eleven states expressed interest in learning more about MDSS and how it could be evaluated in their states. The MDSS presentation is scheduled for September 8, 2005 at the 10th Eastern Winter Road Maintenance Symposium and Equipment Expo being held in Hartford, Connecticut. The opinion of the author is that some of these Midwestern states and other states beyond the Midwest will soon follow the Iowa DOT's lead in implementing MDSS and eventually requiring MDSS guidance in their future weather forecasting services.

CONCLUSION

The first part of this paper addressed identifying and correcting the inadequacies of the current RWIS ESS sites and sensing equipment and the efforts to establish a national road weather observation system. The referenced studies by the FHWA, Aurora, and NCHRP will improve the sensor quality and representative characteristics of each site. The Clarus Initiative will provide quality assurance of the data stream. The Clarus System operational attributes call for continuous quality control of data with feedback to state DOTs and other users.

The reminder of this paper addressed the MDSS project with its integrated state-of-the-art weather forecasting and data fusion and treatment optimization techniques. Field evaluations show MDSS will provide maintenance managers with a specific forecast of road surface conditions and optimized treatment recommendations customized for each plow route, thus maximizing the efficiency and effectiveness of snow and ice control operations.

Stakeholders in these projects have expressed their confidence that quality data, transferred in one common protocol with full metadata, as envisioned in the Clarus Initiative, will result in more effective and proactive operational decision making and better performance of meteorological support services. Quality data provided to travelers will build their confidence and help them develop and promote a proactive approach in their decision making process.

Although much as been accomplished in developing "Surface Transportation Weather for the 21st Century", considerable research, evaluation and testing remains to be done. A common vision and belief that prevails at every stakeholder meeting is that these unlimited opportunities for safety, mobility, and operational improvements are achievable.

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Guidelines for the Removal of Traffic Control Devices in Rural Areas

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ABSTRACT

The overuse of traffic control devices desensitizes drivers and leads to disrespect, especially on low-volume secondary roads with limited enforcement. The maintenance of traffic signs is also a tort liability issue, exacerbated by unnecessary signs. The Federal Highway Administration's (FHWA) Manual on Uniform Traffic Control Devices (MUTCD) and the Institute of Transportation Engineers' Traffic Control Devices Handbook provide guidance for the implementation of stop signs based on expected compliance with right-of-way rules, provision of through traffic flow, context (proximity to other controlled intersections), speed, sight distance, and crash history. The direction to stop is left to engineering judgment and is usually dependent on traffic volume or functional class/continuity of the system. Although currently being considered by the National Committee on Traffic Control Devices, traffic volume itself is not given as a criterion for implementation in the MUTCD. Stop signs have been installed at many locations for various reasons that no longer (or perhaps never) met engineering needs. However, no guidance exists for the removal of stop signs at two-way stop-controlled intersections. The scope of this research is low volume gravel intersections in rural agricultural areas of Iowa, where each of the 99 counties may have as many as 300 or more stop sign pairs. This research investigates the relationship between type of control and independent variables (including traffic volume) to determine the type of warrants that may be used for the removal of stop signs. Overall safety performance is examined as a function of a county overuse factor, developed specifically for this study and based on terrain as a proxy for sight distance and various volume warrant levels. Counties with high levels of rural gravel stop controlled intersection crashes may wish to examine the potential for removing stop control in unnecessary locations to promote compliance with remaining control devices.

Key words: rural roadways—stop signs— stop sign removal—unpaved roads

INTRODUCTION

Many rural Iowa two-way stop signs are installed based on policy, general procedure, or an engineering study of geometric and operational factors. Still others may have been placed in response to citizen complaints or studies that were conducted when traffic volumes may have been greater or sight distances smaller than they are today. Sign removal or a change to less restrictive control (i.e., yield signs) may present an opportunity for improved operations and safety through increased respect for traffic control devices.

Problem Statement

Unnecessary stop signs are expensive to maintain, are a potential liability, and can reduce respect for necessary signs. The expense of maintenance and generally sparse enforcement in rural areas is particularly challenging to local officials. The effect of the overuse of stop control on traffic control disrespect has not been quantified. Furthermore, there are no criteria for the removal of two-way stop signs or the change to less restrictive control.

Current practices and the potential for traffic control reduction at two-way rural gravel-to-gravel road intersections in Iowa are investigated. The safety performance of intersections with and without stop signs is compared. Results are intended to serve as a first step in the development of warrants and procedures for elimination of unnecessary stop control.

Objectives

This study pursued several objectives framed by the following questions:

1. Adjusted for volumes alone, do stop-controlled rural gravel intersections have lower crash rates than uncontrolled intersections?
2. Can a factor to represent the overuse of stop signs be developed to quantify the effect of stop sign disrespect?
3. As data collection for sight distance for thousands of intersections is cost prohibitive, is it possible to develop a surrogate for the relative number of intersections that would be expected to have sight distance limitations, based on county landcover and topography?
4. Can a combination of volume, overuse, and terrain be used to improve models of the effectiveness of stop control at low-volume intersections?
5. Do certain driver groups (older or younger) have particular problems at rural stop-controlled or uncontrolled intersections?
6. Are certain types of crashes more or less prevalent at rural stop-controlled and uncontrolled intersections?
7. Can region-specific guidance for the removal/reduction of control at two-way stop-controlled intersections be developed?

BACKGROUND AND LITERATURE REVIEW

Installation Warrants

Two key references are used for warrants regarding the installation of stop signs: the Federal Highway Administration’s (FHWA) Manual on Uniform Traffic Control Devices (MUTCD) and the Institute of Transportation Engineers’ (ITE) Traffic Control Devices Handbook. Neither of these documents provides warrants for the removal of stop signs. The MUTCD calls for multi-way stop signs at intersections with high speeds, sight distance issues, or a crash problem (indicated by five or more reported crashes in a 12-month period that can be corrected by a multi-way stop installation), but does not include a daily entering vehicle (DEV) volume warrant. Instead, it has a warrant based upon the average volume for both the major and minor roadway over an eight hour period during the day. This warrant states that a stop sign should be considered at a location where the vehicular volume entering the intersection from the major street approaches (total of both approaches) averages at least 300 vehicles per hour for any 8 hours of an average day, and the combined vehicular, pedestrian, and bicycle volume entering the intersection from the minor street approaches (total of both approaches) averages at least 200 units per hour for the same 8 hours, with an average delay to minor-street vehicular traffic of at least 30 seconds per vehicle during the highest hour. The FHWA conducted a study in 1981 to attempt to establish definitive criteria for the application of two-way stop or yield control at low-volume intersections. After completing an analysis of variance, the researchers observed a significant increase in crash experience when the volume on the major roadway reached 2,000 vehicles per day. The Uniform Traffic Control Devices Committee is considering proposed language for the third revision of the MUTCD for stop or yield signs used at the intersection of two minor streets, or local roads having more than three approaches, where the combined vehicular, pedestrian, and bicycle volume entering the intersection from all approaches averages more than 2,000 units per day. The proposed language says that stop signs should be considered if engineering judgment indicates that a stop is always required when vehicular traffic volumes on the through street or highway exceed 6,000 vehicles per day. The ITE handbook discourages the use of multi-way stop signs unless the volumes on the major and minor roadways are approximately equal. The ITE handbook also states that the MUTCD warrants should be the main criteria used to determine the locations for stop signs.

In several studies, Stokes (2000; 2004) advocates stop sign warrants based on available sight distance and crash history. The studies contend that the majority of crashes associated with rural stop-controlled intersections are not caused by stop sign violations, but rather the driver's inability to judge distance adequately. Stokes concludes that effective solutions to the failure to yield problem should focus on the intersection as a whole rather than simply improving the sight distance or other characteristics of one leg of the intersection. He suggests the use of speed zones and advanced warning signs in places where drivers on the side road may have difficulty judging the speeds of approaching vehicles. Table 1 presents Stokes' recommendation for control based on available sight distance, crash history (three years worth of data), and the major roadway volume. A note at the bottom of the chart indicates that the values should be used in conjunction with MUTCD criteria when assessing the need for stop or yield control. It should be noted that while many intersections are stop controlled, volumes at Iowa rural gravel intersections are typically an order of magnitude lower than the values recommended by Stokes.

Table 1: Stokes' method for determining control type

<table>
<tr><th rowspan="2">Sight Distance</th><th rowspan="2">Accident History
(Last 3 Years)</th><th colspan="2">Major Roadway Volume</th></tr>
<tr><th>≤ 2000 vpd</th><th>> 2000 vpd</th></tr>
<tr><td rowspan="4">Adequate</td><td>0</td><td>No Control</td><td rowspan="2">YIELD</td></tr>
<tr><td>≤ 2</td><td>YIELD</td></tr>
<tr><td>3</td><td colspan="2">STOP*</td></tr>
<tr><td>4+</td><td colspan="2" rowspan="2">STOP</td></tr>
<tr><td>Not Adequate</td><td>Not Applicable</td></tr>
</table>

* If minor roadway volume is greater than 300 vpd, YIELD control is appropriate for intersections with less than 4 accidents in 3 years.

Note: The material in this table is intended to be used in conjunction with appropriate MUTCD criteria is assessing the need for STOP or YIELD control.

Geometric Studies

Several studies report on the impact of geometry on safety of stop-controlled intersections. Stockton (1981) concluded that geometry plays no significant role in safety or operations for choosing the type of control to be used. The report recommended that major roadway volume should be the major factor in the determination of control type and that sight distance has no significant impact on the number of crashes that occur at an intersection. Mounce (1981) completed a study in which he agrees with Stockton, indicating that the choice of control should be strictly based only on the volume of the major roadway, as long as the available sight distance is greater than the minimum required. The study recommended that no control be used on an intersection with a major roadway volume between 0 and 2,000 vehicles, yield control be used with a major roadway volume between 2,000 and 5,000 vehicles, and stop control be used when the major roadway volume is greater than 5,000 vehicles. (The intersection with the highest DEV being investigated in this study is about 600 vehicles per day.)

Although Stockton and Mounce do not advocate use of sight distance to determine control type, it should be noted that there are at least two sight distance methods commonly used in the traffic engineering industry. One was included in the Traffic Control Devices Handbook (TCDH) published by the Federal Highway Administration in 1983. However, this method was not included in the handbook when it was updated in 2001. The second can be found in A Policy on Geometric Design of Highways and Streets 2001, Fourth Edition, published by the American Association of State Highway and Transportation Officials (AASHTO).

Using the TCDH method, "the decision as to whether to use a STOP sign or a YIELD sign is primarily based upon sight distance at the intersection." The method uses a "critical approach speed," which is the lowest speed that a motorist would be able to travel and fail to avoid a collision with an approaching vehicle on the cross street.

The distances to sight obstructions at the intersection under study are measured, parking is considered, and a nomograph is used to determine the critical approach speeds. An intersection with a critical approach speed of less than 10 mph is controlled by a stop sign. An intersection where the critical approach speed is between 10 and 15 mph may be controlled with a yield sign. If the critical approach speed on all approaches is above 15 mph, then the intersection can be left uncontrolled.

The TCDH assumes that motorists approaching uncontrolled intersections reduce their speed to account for prevailing conditions at the intersection. Consequently, the use of this method can result in a number of intersections remaining uncontrolled. The method can be useful in urban areas. When Mounce refers to sight distance, he is using the TCDH method.

The AASHTO method assumes that motorists approaching uncontrolled intersections reduce their speed by 50% based on observations. The method uses this assumption to determine the length of sight distance necessary along each leg of the intersecting roadway, resulting in a desirable sight distance triangle. If this sight distance triangle exists, then no control is needed. If it does not exist, then control is needed. This method can also be adjusted for grades.

The AASHTO method tends to be more suited for rural areas. Use of this method almost always results in control being installed.

METHODOLOGY

A survey was sent to all 99 Iowa county engineers. Respondents were asked the following:

- Number of uncontrolled intersections, all-way stops, and yield-controlled intersections that used by the county
- Criteria used for installation of stop signs, including references used to determine warrants (MUTCD, ITE, etc.)
- Whether an engineering study was completed prior to installation
- Whether any formal or general policy exists for stop sign usage (e.g., placing a stop sign at all intersections, only at intersections with sight distance issues, only at intersections with paved roadways, etc.)

A stop sign database has also been created for 19 counties. A crash analysis was conducted using ten years of crash data (1994–2003). The average number of crashes, crash rate, average severity loss, and cost per intersection was computed and summarized for each county and stratified by the number of intersection legs and type of control. An example county summary is shown in Table 2. All data were pooled to create a statewide summary table, shown in Table 3.

Table 2: Crash analysis in Story County

Unpaved - Unpaved Intersections - Story County

	3 Intersection Legs			4 Intersection Legs		
	Control Type			Control Type		
	Uncontrolled	Yield Controlled	Stop Controlled	Uncontrolled	Yield Controlled	Stop Controlled
Value per intersection						
Avg Rate per MEV	0.30	4.98	0.30	1.06	0.00	1.94
Avg DEV	86	55	93	88	114	115
Avg crashes	0.09	1.00	0.20	0.33	0.00	0.57
Avg sev_index	0.15	1.00	0.40	0.73	0.00	1.14
Avg severity cost	$1,266	$2,500	$500	$20,344	$0	$3,571
Number of intersections	156	1	5	160	3	7
Max Rate	24.35	NA	1.52	9.13	0	7.83
Min Rate	0	NA	0	0	0	0
Max DEV	395	NA	180	435	170	255
Min DEV	18	NA	25	25	73	35
Max Crashes	4	NA	1	4	0	1
Min Crashes	0	NA	0	0	0	0
Max Severity Cost	$150,000	NA	$2,500	$1,000,000	$0	$10,000
Min Severity Cost	$0.00	NA	$0.00	$0.00	$0.00	$0.00
Total Crashes	14	1	1	52	0	4

Pooled Data

	Control Type		
	Uncontrolled	Yield Controlled	Stop Controlled
Value per intersection			
Avg Rate per MEV	0.68	1.25	1.26
Avg DEV	87	100	106
Avg crashes	0.21	0.25	0.42
Avg sev_index	0.44	0.25	0.83
Avg severity cost	$10,926	$625	$2,292
Number of intersections	316	4	12
Max Rate	24.35	4.98	7.83
Min Rate	0	0	0
Max DEV	435	170	255
Min DEV	18	55	25
Max Crashes	4	1	1
Min Crashes	0	0	0
Max Severity Cost	$1,000,000	$2,500	$10,000
Min Severity Cost	$0.00	$0.00	$0.00
Total Crashes	66	1	5

	All Data
Value per intersection	
Avg Rate per MEV	0.71
Avg DEV	88
Avg crashes	0.22
Avg sev_index	0.45
Avg severity cost	$10,489
Number of intersections	332
Max Rate	24.35
Min Rate	0
Max DEV	435
Min DEV	18
Max Crashes	4
Min Crashes	0
Max Severity Cost	$1,000,000
Min Severity Cost	$0.00
Total Crashes	72

Table 3: Statewide crash analysis

Unpaved - Unpaved Intersections - 19 Counties - 10 years worth of crash data

	3 Intersection Legs		4 Intersection Legs	
	Control Type		Control Type	
	Uncontrolled	Stop Controlled	Uncontrolled	Stop Controlled
Value per intersection				
Avg Rate per MEV	0.17	0.14	0.50	0.36
Avg DEV	67	80	69	94
Avg crashes	0.04	0.05	0.13	0.14
Avg sev_index	0.08	0.07	0.33	0.29
Avg severity cost	$836	$291	$12,470	$8,370
Number of intersections	2268	1373	1561	1607
Max Rate	24.35	13.70	18.26	10.96
Min Rate	0	0	0	0
Max DEV	465	440	443	455
Min DEV	8	8	15	10
Max Crashes	4	2	4	4
Min Crashes	0	0	0	0
Max Severity Cost	$150,000	$150,000	$1,150,000	$1,020,000
Min Severity Cost	$0.00	$0.00	$0.00	$0.00
Total Crashes	99	62	205	217

Pooled Data

	Control Type	
	Uncontrolled	Stop Controlled
Value per intersection		
Avg Rate per MEV	0.31	0.26
Avg DEV	68	88
Avg crashes	0.08	0.09
Avg sev_index	0.18	0.19
Avg severity cost	$5,578	$4,648
Number of intersections	3829	2980
Max Rate	24.35	13.70
Min Rate	0	0
Max DEV	465	455
Min DEV	8	8
Max Crashes	4	4
Min Crashes	0	0
Max Severity Cost	$1,150,000	$1,020,000
Min Severity Cost	$0.00	$0.00
Total Crashes	304	279
Average cost per crash	$70,265.00	$49,645.00

All Data	
Value per intersection	
Avg Rate per MEV	0.28
Avg DEV	77
Avg crashes	0.09
Avg sev_index	0.18
Avg severity cost	$5,171
Number of intersections	6809
Max Rate	24.35
Min Rate	0
Max DEV	465
Min DEV	8
Max Crashes	4
Min Crashes	0
Max Severity Cost	$1,150,000
Min Severity Cost	$0.00
Total Crashes	583

Initial Analysis

Statewide stop-controlled intersections have a slightly lower average crash rate per million entering vehicles, as well as a lower average cost per crash. On the surface, it would appear that savings could be realized by switching all of the uncontrolled intersections to stop-controlled intersections.

$$\text{Crashes saved by changing to stop control} = (0.05*68*365*10*3829)/1000000 = 47.5 \text{ crashes} \quad (1)$$

To calculate the expected savings, the cost of the crashes after the conversion (256 crashes x average cost per crash for stop-controlled intersections, $50,000) is subtracted from the cost of the crashes before the conversion (304 crashes x average cost per crash for uncontrolled intersections, $70,000). The savings would be approximately $8.5 million, not including the cost of installing and maintaining stop signs, which may be on the order of $100/sign x 2 x 3829 intersections = $770,000. The relationship between crashes and volume that one would expect to observe can be seen in Figure 1. Each point represents an intersection where the uncontrolled intersections are expected to have a higher number of crashes per average volume than stop-controlled intersections.

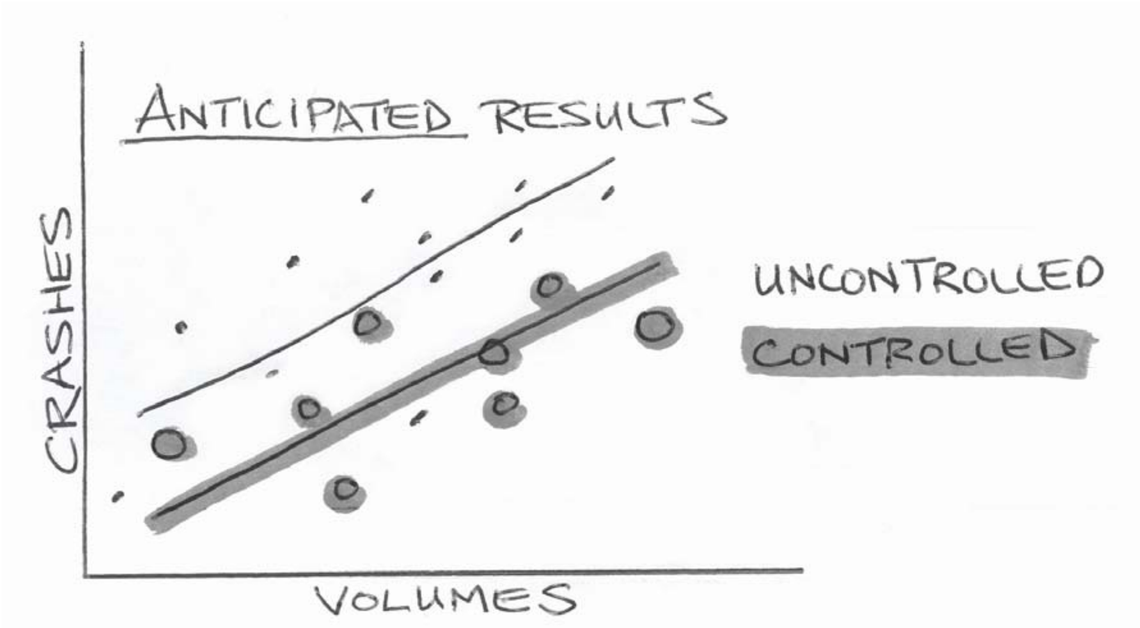

Figure 1. Relationship between crashes and volume

Regression

Logistic regression was used to determine the probability of one or more crashes occurring during 10 years based on type of control and DEV. Figure 2 shows the regression equation and resulting probabilities. Type of control is treated as a dummy variable (-1 for stop controlled, +1 for no control). Below 100 DEV, uncontrolled intersections have a lower probability of crashes than stop-controlled intersections. Sight distance is not included as a variable in this analysis.

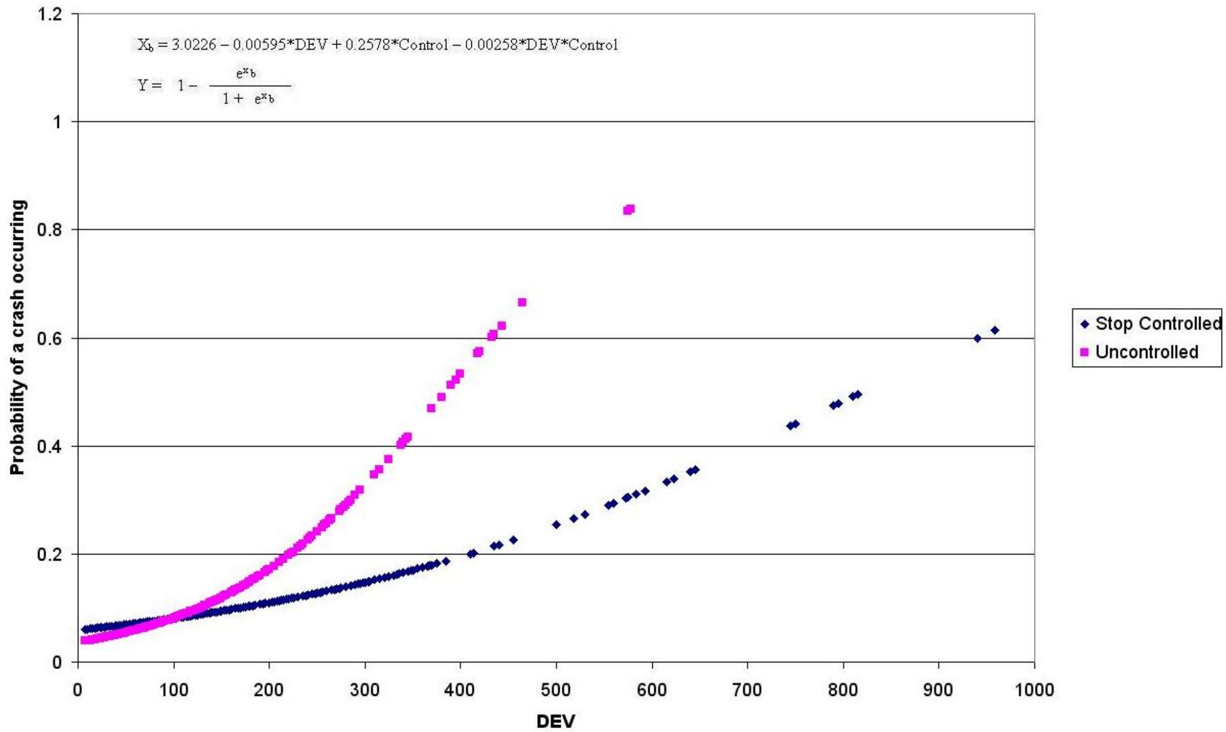

Figure 2: Probability of a crash occurring in 10 years based on type of control and DEV

Overuse Factor

The analysis presented above makes the simple assumption that conversion of all intersections to stop control would result in a reduction of crashes. The procedure does not take into account the potential for increasing disrespect for signs or any underlying factors that may make the previously uncontrolled intersections different from their controlled counterparts. Therefore, an overuse factor was developed that is a function of how many stop signs a county has that are unwarranted (expressed as a percentage). To determine the number of unwarranted intersections per county (on a volume basis alone), a volume warrant was chosen, as indicated in Table 4, and the corresponding percentage of intersections exceeding that warrant were used in the following equation to determine the percentage of unwarranted stop-controlled intersections for that county:

Overuse factor (based solely on volume) =

(Total # of stop int – warranted # number of stop int) / total # of stop int (2)

The chart below shows a sensitivity analysis for an overuse factor based on volume alone. Volume warrants ranged from 50 to 200 vehicles per day. After the number of intersections meeting each volume warrant was determined, an overuse factor for each county was calculated.

Table 4. Table of overuse factors based on volume alone

Volume "Warrant" Values							% "warranted" on volume alone				% overuse on volume alone			
County	50	100	150	200	Total # of intersections	# of stop controlled intersections	50	100	150	200	50	100	150	200
	Number of intersections meeting warrant													
Adams	149	17	4	2	361	252	41%	5%	1%	1%	41%	93%	98%	99%
Boone	282	60	18	6	379	94	74%	16%	5%	2%	-200%	36%	81%	94%
Bremer	224	77	39	14	268	113	84%	29%	15%	5%	-98%	32%	65%	88%
Calhoun	237	50	16	5	379	140	63%	13%	4%	1%	-69%	64%	89%	96%
Carroll	343	144	46	21	401	245	86%	36%	11%	5%	-40%	41%	81%	91%
Cedar	313	142	58	29	396	167	79%	36%	15%	7%	-87%	15%	65%	83%
Cerro Gordo	282	89	23	8	359	208	79%	25%	6%	2%	-36%	57%	89%	96%
Cherokee	183	33	9	6	350	93	52%	9%	3%	2%	-97%	65%	90%	94%
Clay	198	40	10	3	332	49	60%	12%	3%	1%	-304%	18%	80%	94%
Emmet	90	10	4	3	183	92	49%	5%	2%	2%	2%	89%	96%	97%
Henry	263	130	51	23	346	181	76%	38%	15%	7%	-45%	28%	72%	87%
Madison	347	166	101	55	496	143	70%	33%	20%	11%	-143%	-16%	29%	62%
Montgomery	183	54	8	1	349	208	52%	15%	2%	0%	12%	74%	96%	100%
Osceola	121	19	4	0	219	98	55%	9%	2%	0%	-23%	81%	96%	100%
Pocahontas	201	26	3	0	325	66	62%	8%	1%	0%	-205%	61%	95%	100%
Sac	269	54	16	5	379	88	71%	14%	4%	1%	-206%	39%	82%	94%
Story	253	98	33	15	328	12	77%	30%	10%	5%	-2008%	-717%	-175%	-25%
Washington	337	151	61	24	483	398	70%	31%	13%	5%	15%	62%	85%	94%
Woodbury	291	125	81	57	489	368	60%	26%	17%	12%	21%	66%	78%	85%

Figure 3 shows the expected relationship between counties with higher and lower overuse factors.

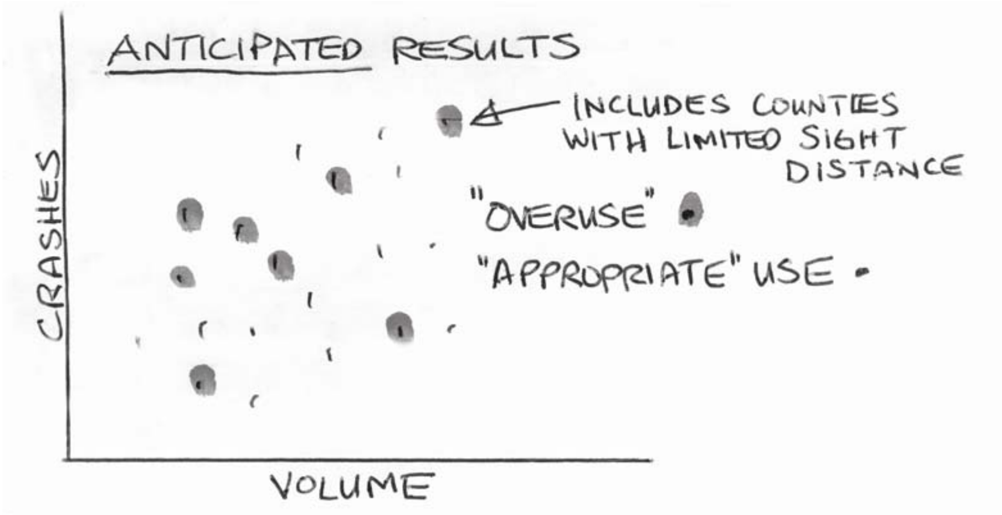

Figure 3. Expected relationship between crashes and volume (not accounting for sight distance)

While an overuse factor may be used to estimate the number of unwarranted stop-controlled intersections based on volume, it does not account for the use of stop signs warranted based on sight distance issues.

Terrain Factor

To attempt to account for the intersections with sight distance issues, a terrain factor has been developed. The overuse factor (based on volume alone) can be adjusted by this terrain factor to more appropriately represent the effect of stop sign overuse. This factor was qualitatively created based on topography and land cover on a county by county basis. The terrain factor was created using a United States Geographical Services (USGS) land cover and shaded relief map for the state of Iowa. Each county was given a value of one to three (one = low, flat land, agriculture cover; three = high, river land, forest cover) based on topography and land cover. See Figures 4 through 6 below.

Figure 4. USGS land cover map

Figure 5. USGS shaded relief map

Figure 6. Maps showing values for each county based on topography and land

Warranted Intersections Based on Volume and Terrain

Volume and terrain are incorporated in the following equation to determine the fraction of warranted stop-controlled intersections for each county:

Fraction of warranted stop int = 1 – (1 – volume fraction) x (1 – terrain fraction) (3)

An adjusted overuse factor was calculated as follows:

Overuse factor (based on volume and sight distance) =
(Total # of stop int – warranted # number of stop int) / total # of stop int (4)

The development of the overuse factor based on both volume and terrain is seen below in Tables 5 through 7. Volume warrants again ranged from 50 to 200 vehicles per day, and these were combined with three sets of terrain factors to comprise 12 inputs for regression and sensitivity analysis.

Table 5. Development of the overuse factor (1)

Terrain values				Warranted stops based on both volume and terrain				Overuse factors based on volume and terrain			
County	Landcover	Topography	Terrain factor	50	100	150	200	50	100	150	200
Adams	0.7	0.7	0.49	253	186	179	178	0.00	0.26	0.29	0.29
Boone	0.5	0.5	0.25	306	140	108	99	-2.26	-0.49	-0.15	-0.06
Bremer	0.5	0.5	0.25	235	125	96	78	-1.08	-0.10	0.15	0.31
Calhoun	0.5	0.5	0.25	273	132	107	99	-0.95	0.06	0.24	0.30
Carroll	0.5	0.5	0.25	358	208	135	116	-0.46	0.15	0.45	0.53
Cedar	0.7	0.7	0.49	354	266	224	209	-1.12	-0.60	-0.34	-0.25
Cerro Gordo	0.5	0.5	0.25	301	157	107	96	-0.45	0.25	0.49	0.54
Cherokee	0.5	0.5	0.25	225	112	94	92	-1.42	-0.21	-0.01	0.01
Clay	0.5	0.5	0.25	232	113	91	85	-3.72	-1.31	-0.85	-0.74
Emmet	0.5	0.5	0.25	113	53	49	48	-0.23	0.42	0.47	0.48
Henry	0.7	0.7	0.49	304	236	196	181	-0.68	-0.30	-0.08	0.00
Madison	0.9	0.7	0.63	441	374	350	333	-2.08	-1.61	-1.45	-1.33
Montgomery	0.5	0.7	0.35	241	157	127	123	-0.16	0.24	0.39	0.41
Osceola	0.5	0.5	0.25	146	69	58	55	-0.48	0.30	0.41	0.44
Pocahontas	0.5	0.5	0.25	232	101	84	81	-2.52	-0.53	-0.27	-0.23
Sac	0.5	0.5	0.25	297	135	107	99	-2.37	-0.54	-0.21	-0.12
Story	0.5	0.5	0.25	272	156	107	93	-21.65	-11.96	-7.90	-6.77
Washington	0.7	0.7	0.49	409	314	268	249	-0.03	0.21	0.33	0.37
Woodbury	0.5	0.7	0.35	360	252	224	208	0.02	0.31	0.39	0.43

Table 6. Development of the overuse factor (2)

Terrain values				Warranted stops based on both volume and terrain				Overuse factors based on volume and terrain			
County	Landcover	Topography	Terrain factor	50	100	150	200	50	100	150	200
Adams	0.6	0.6	0.36	225	141	133	131	0.11	0.44	0.47	0.48
Boone	0.4	0.4	0.16	298	111	76	66	-2.17	-0.18	0.19	0.30
Bremer	0.4	0.4	0.16	231	108	76	55	-1.04	0.05	0.33	0.52
Calhoun	0.4	0.4	0.16	260	103	74	65	-0.86	0.27	0.47	0.54
Carroll	0.4	0.4	0.16	352	185	103	82	-0.44	0.24	0.58	0.67
Cedar	0.6	0.6	0.36	343	233	180	161	-1.05	-0.40	-0.08	0.04
Cerro Gordo	0.4	0.4	0.16	294	132	77	64	-0.42	0.36	0.63	0.69
Cherokee	0.4	0.4	0.16	210	84	64	61	-1.26	0.10	0.32	0.34
Clay	0.4	0.4	0.16	219	87	62	56	-3.48	-0.77	-0.26	-0.14
Emmet	0.4	0.4	0.16	105	38	33	32	-0.14	0.59	0.65	0.65
Henry	0.6	0.6	0.36	293	208	157	139	-0.62	-0.15	0.13	0.23
Madison	0.8	0.6	0.48	419	324	291	267	-1.93	-1.27	-1.03	-0.86
Montgomery	0.4	0.6	0.24	223	125	90	85	-0.07	0.40	0.57	0.59
Osceola	0.4	0.4	0.16	137	51	38	35	-0.39	0.48	0.61	0.64
Pocahontas	0.4	0.4	0.16	221	74	55	52	-2.35	-0.12	0.17	0.21
Sac	0.4	0.4	0.16	287	106	74	65	-2.26	-0.20	0.16	0.26
Story	0.4	0.4	0.16	265	135	80	65	-21.08	-10.23	-5.68	-4.42
Washington	0.6	0.6	0.36	390	271	213	189	0.02	0.32	0.47	0.52
Woodbury	0.4	0.6	0.24	339	212	179	161	0.08	0.42	0.51	0.56

Table 7. Development of the overuse factor (3)

Terrain values				Warranted stops based on both volume and terrain				Overuse factors based on volume and terrain			
County	Landcover	Topography	Terrain factor	50	100	150	200	50	100	150	200
Adams	0.6	0.6	0.36	225	141	133	131	0.11	0.44	0.47	0.48
Boone	0.3	0.3	0.09	291	89	50	40	-2.09	0.06	0.46	0.58
Bremer	0.3	0.3	0.09	228	94	60	37	-1.02	0.17	0.47	0.67
Calhoun	0.3	0.3	0.09	250	80	49	39	-0.78	0.43	0.65	0.72
Carroll	0.3	0.3	0.09	348	167	78	55	-0.42	0.32	0.68	0.77
Cedar	0.6	0.6	0.36	343	233	180	161	-1.05	-0.40	-0.08	0.04
Cerro Gordo	0.3	0.3	0.09	289	113	53	40	-0.39	0.46	0.74	0.81
Cherokee	0.3	0.3	0.09	198	62	40	37	-1.13	0.34	0.57	0.60
Clay	0.3	0.3	0.09	210	66	39	33	-3.29	-0.35	0.20	0.33
Emmet	0.3	0.3	0.09	98	26	20	19	-0.07	0.72	0.78	0.79
Henry	0.6	0.6	0.36	293	208	157	139	-0.62	-0.15	0.13	0.23
Madison	0.9	0.6	0.54	427	344	314	293	-1.99	-1.41	-1.20	-1.05
Montgomery	0.3	0.6	0.18	213	107	69	64	-0.02	0.49	0.67	0.69
Osceola	0.3	0.3	0.09	130	37	23	20	-0.32	0.62	0.76	0.80
Pocahontas	0.3	0.3	0.09	212	53	32	29	-2.21	0.20	0.52	0.56
Sac	0.3	0.3	0.09	279	83	49	39	-2.17	0.05	0.45	0.56
Story	0.3	0.3	0.09	260	119	60	43	-20.65	-8.89	-3.96	-2.60
Washington	0.6	0.6	0.36	390	271	213	189	0.02	0.32	0.47	0.52
Woodbury	0.3	0.6	0.18	327	191	154	135	0.11	0.48	0.58	0.63

Equation 3 takes both volume and a proxy for sight distance warrants into account and is expected to result in a relationship such as the one depicted in Figure 7. Each point represents one county and the counties with higher adjusted overuse factors are expected to have a higher average crash rate.

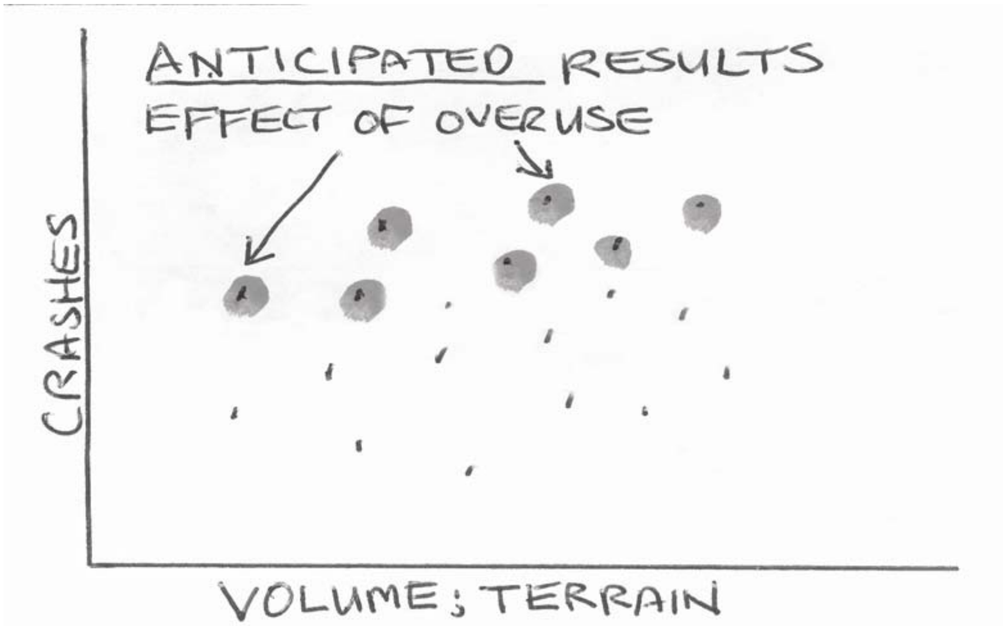

Figure 7. Expected effect of stop sign overuse

Age Group Analysis

An age group analysis was completed to determine whether older or younger drivers have more problems at rural stop-controlled and uncontrolled intersections. Tables 8 and 9 show the involvement of younger (less than or equal to 19 years) and older (greater than or equal to 65 years) drivers in all crashes and in all multi-vehicle crashes in Iowa during 2003. All multi-vehicle crashes in the study are rural gravel stop-controlled and uncontrolled intersections between 1994 and 2003.

Table 8 shows that the representations of older and younger drivers are approximately equal statewide, but at stop-controlled gravel intersections, there is a significant overrepresentation of younger drivers (about 35% of crashes and 20% of drivers) as opposed to older drivers (about 14% of crashes and 7% of drivers). These differences could be explained if inexperienced drivers have more difficulty judging distances than older drivers.

Table 9 shows that there is a significant overrepresentation of younger drivers at uncontrolled gravel intersections (about 30% of crashes and 18% of drivers) as opposed to older drivers (about 18% of crashes and 9% of drivers). Overall, younger drivers are also overrepresented at study area gravel intersections, (about 33% of crashes and 19% of drivers) as opposed to older drivers (about 16% of crashes and 8% of drivers).

Table 7. Age group analysis, stop-controlled intersections

Statewide - 2003 - All crashes						Younger drivers' %		Older drivers' %	
Crashes	Drivers	Crashes <= 19	Drivers <= 19	Crashes >= 65	Drivers >= 65	Crashes	Drivers	Crashes	Drivers
59472	101987	15259	16828	14476	17035	25.66%	16.50%	24.34%	16.70%

Statewide - 2003 - > 1 vehicle crashes						Younger drivers' %		Older drivers' %	
Crashes	Drivers	Crashes <= 19	Drivers <= 19	Crashes >= 65	Drivers >= 65	Crashes	Drivers	Crashes	Drivers
38593	81108	11331	12900	12479	15038	29.36%	15.90%	32.33%	18.54%

Stop Controlled intersections							Younger drivers' %		Older drivers' %	
County	Crashes	Drivers	Crashes <= 19	Drivers <= 19	Crashes >= 65	Drivers >= 65	Crashes	Drivers	Crashes	Drivers
Adams	8	16	1	1	5	6	12.50%	6.25%	62.50%	37.50%
Boone	29	60	12	12	3	3	41.38%	20.00%	10.34%	5.00%
Bremer	24	48	11	12	1	1	45.83%	25.00%	4.17%	2.08%
Calhoun	13	26	3	3	2	3	23.08%	11.54%	15.38%	11.54%
Carroll	37	75	12	14	3	3	32.43%	18.67%	8.11%	4.00%
Cedar	28	57	14	15	4	4	50.00%	26.32%	14.29%	7.02%
Cerro Gordo	11	22	1	2	1	1	9.09%	9.09%	9.09%	4.55%
Cherokee	5	10	1	1	0	0	20.00%	10.00%	0.00%	0.00%
Clay	5	10	3	3	3	4	60.00%	30.00%	60.00%	40.00%
Emmet	8	16	2	2	0	0	25.00%	12.50%	0.00%	0.00%
Henry	32	66	9	13	5	5	28.13%	19.70%	15.63%	7.58%
Madison	17	34	9	10	1	1	52.94%	29.41%	5.88%	2.94%
Montgomery	11	22	4	5	2	2	36.36%	22.73%	18.18%	9.09%
Osceola	6	12	1	1	0	0	16.67%	8.33%	0.00%	0.00%
Pocahontas	6	12	1	1	2	2	16.67%	8.33%	33.33%	16.67%
Sac	8	16	1	1	0	0	12.50%	6.25%	0.00%	0.00%
Story	6	13	1	2	2	2	16.67%	15.38%	33.33%	15.38%
Washington	30	61	11	15	2	3	36.67%	24.59%	6.67%	4.92%
Woodbury	37	75	15	17	8	8	40.54%	22.67%	21.62%	10.67%
Totals	321	651	112	130	44	48	34.89%	19.97%	13.71%	7.37%

Table 8. Age group analysis, uncontrolled intersections

Uncontrolled intersections							Younger drivers' %		Older drivers' %	
County	Crashes	Drivers	Crashes <= 19	Drivers <= 19	Crashes >= 65	Drivers >= 65	Crashes	Drivers	Crashes	Drivers
Adams	1	2	0	0	0	0	0.00%	0.00%	0.00%	0.00%
Boone	40	81	7	7	4	4	17.50%	8.64%	10.00%	4.94%
Bremer	32	65	13	17	9	9	40.63%	26.15%	28.13%	13.85%
Calhoun	9	18	5	5	1	1	55.56%	27.78%	11.11%	5.56%
Carroll	12	26	4	5	3	3	33.33%	19.23%	25.00%	11.54%
Cedar	16	34	4	4	4	4	25.00%	11.76%	25.00%	11.76%
Cerro Gordo	16	32	6	6	4	4	37.50%	18.75%	25.00%	12.50%
Cherokee	20	40	6	9	4	4	30.00%	22.50%	20.00%	10.00%
Clay	25	50	6	7	3	3	24.00%	14.00%	12.00%	6.00%
Emmet	3	6	2	2	0	0	66.67%	33.33%	0.00%	0.00%
Henry	6	12	5	6	0	0	83.33%	50.00%	0.00%	0.00%
Madison	27	56	11	16	2	2	40.74%	28.57%	7.41%	3.57%
Montgomery	4	8	2	2	0	0	50.00%	25.00%	0.00%	0.00%
Osceola	5	10	2	2	2	2	40.00%	20.00%	40.00%	20.00%
Pocahontas	15	30	6	7	2	2	40.00%	23.33%	13.33%	6.67%
Sac	19	38	5	5	5	5	26.32%	13.16%	26.32%	13.16%
Story	76	152	17	19	14	16	22.37%	12.50%	18.42%	10.53%
Washington	13	28	3	4	3	3	23.08%	14.29%	23.08%	10.71%
Woodbury	7	14	1	1	1	1	14.29%	7.14%	14.29%	7.14%
Totals	346	702	105	124	61	63	30.35%	17.66%	17.63%	8.97%

All gravel to gravel rural intersections							Younger drivers' %		Older drivers' %	
County	Crashes	Drivers	Crashes <= 19	Drivers <= 19	Crashes >= 65	Drivers >= 65	Crashes	Drivers	Crashes	Drivers
Adams	9	18	1	1	5	6	11.11%	5.56%	55.56%	33.33%
Boone	69	141	19	19	7	7	27.54%	13.48%	10.14%	4.96%
Bremer	56	113	24	29	10	10	42.86%	25.66%	17.86%	8.85%
Calhoun	22	44	8	8	3	4	36.36%	18.18%	13.64%	9.09%
Carroll	49	101	16	19	6	6	32.65%	18.81%	12.24%	5.94%
Cedar	44	91	18	19	8	8	40.91%	20.88%	18.18%	8.79%
Cerro Gordo	27	54	7	8	5	5	25.93%	14.81%	18.52%	9.26%
Cherokee	25	50	7	10	4	4	28.00%	20.00%	16.00%	8.00%
Clay	30	60	9	10	6	7	30.00%	16.67%	20.00%	11.67%
Emmet	11	22	4	4	0	0	36.36%	18.18%	0.00%	0.00%
Henry	38	78	14	19	5	5	36.84%	24.36%	13.16%	6.41%
Madison	44	90	20	26	3	3	45.45%	28.89%	6.82%	3.33%
Montgomery	15	30	6	7	2	2	40.00%	23.33%	13.33%	6.67%
Osceola	11	22	3	3	2	2	27.27%	13.64%	18.18%	9.09%
Pocahontas	21	42	7	8	4	4	33.33%	19.05%	19.05%	9.52%
Sac	27	54	6	6	5	5	22.22%	11.11%	18.52%	9.26%
Story	82	165	18	21	16	18	21.95%	12.73%	19.51%	10.91%
Washington	43	89	14	19	5	6	32.56%	21.35%	11.63%	6.74%
Woodbury	44	89	16	18	9	9	36.36%	20.22%	20.45%	10.11%
Totals	667	1353	217	254	105	111	32.53%	18.77%	15.74%	8.20%

Crash Type Analysis

A crash type analysis was completed based the Iowa crash database and validated using crash narratives available from the crash reports. A summary table of this information can be seen in Table 10. As expected, the most prominent contributing circumstance for uncontrolled intersections is failure to yield right of way. The contributing circumstances from the crash data as well as the reports are very similar, but 28 crashes did not have a contributing circumstance from at least one of the drivers involved in the crash. This may mean that the driver did not provide that information on the report used to create the crash data or that the reporting officer did not record any contributing circumstance. The most prominent crash type for uncontrolled intersections is a broadside/right-angle crash.

Failure to yield right of way was the most prominent contributing circumstance at stop-controlled intersections as well. Another common contributing circumstance for stop-controlled intersections was failing to stop at the stop sign. This suggests a level of disrespect for stop signs in rural areas.

Table 9. Crash type analysis

Uncontrolled intersections

Contributing circumstances from crash reports	Number of crashes
FTYROW - pulled out in front of another vehicle	28
Rear end - not paying attention	4
Sight distance related	4
Passing another vehicle	3
Corn crop causing sight distance issues	2

Crash type from crash data	Number of crashes
Broadside/Right angle	21
NA	6
Sideswipe	4
Rear end	2
Head on	2
Other	1
Non-collision	1

Contributing circumstances from crash data	Number of crashes
NA	28
FTYROW	21
Failure to have control	5
No improper action	4
Vision obscured	4
Too fast for conditions	3
Crossed Centerline	2
Other improper action	2
Following too close	1
Illegal/improper parking	1
Improper signal	1
Improper turn	1
Inattentive/distracted	1
Passing where prohibited	1
Wrong way/side of road	1

Stop Controlled intersections

Contributing circumstances from crash reports	Number of crashes
FTYROW - made full stop but pulled out in front of another vehicle	17
Failed to stop at stop sign and pulled out in front of another vehicle	10
Attempted to stop at stop sign but slid through intersection colliding with another vehicle	3
Sight distance related	3
Passing another vehicle	1
Rear end - not paying attention	1
Alcohol involvement	1

Crash type from crash data	Number of crashes
Broadside/Right angle	27
Head on	1
Sideswipe	1
Angle	1
Other	1
NA	1

Contributing circumstances from crash data	Number of crashes
NA	25
FTYROW	14
No improper action	11
Ran stop sign	9
Speed too fast for conditions	3
Other improper action	2
Vision obscured	2
Crossed centerline	1
Failure to have control	1
Passing where prohibited	1
Improper signal	1
Inattentive/distracted	1
Passed stopped school bus	1

CONCLUSIONS

Use of Overuse Factors in Regression

The ranges of overuse factors were incorporated into the regression analysis in an attempt to quantify the effect of traffic control device overuse and disrespect and improve the regression results. However, the use of these factors did not improve the models, indicating either no effect of county-wide overuse of stop signs, or, as is more likely, an error in the construction of the overuse factors themselves, as no site-specific sight distance information was available (i.e., the proxies did not work).

Removal Techniques

Although not currently supported by the models developed in this research, removal of unneeded stop signs may be considered by operating authorities. Should this be the case, procedures need to be developed. ITE suggests procedures for the removal of traffic signals that may also be useful for removal/conversion of stop signs. ITE recommends the distribution of newsletters prior to the activation/removal to inform the public of the future change. ITE also recommends that “signal ahead” signs be installed/removed to warn drivers just before conversion actually takes place. The final ITE recommendation involves the distribution of news releases to all local newspapers, radio, and television stations of the impending change to ensure that all potential users of the intersection are aware of the conversion.

The FHWA has published several techniques for the removal of multi-way stops with minimum hazard. These techniques occur in three phases: pre-conversion phase, conversion phase, and the post-conversion phase. These techniques are as follows:

1. Pre-conversion phase
   - Conduct traffic engineering studies
   - Publicize impending conversion (radio, newspapers, and bills)
   - Post notice signs beneath stop signs 30 days prior to conversion
2. Conversion phase
   - Remove unwarranted signs
   - Remove notice signs
   - Install caution signs beneath stop signs
   - Remove/install “stop ahead” signs as appropriate
   - Improve sight distance
3. Post-conversion phase
   - Monitor intersection
   - Police enforcement
   - Remove caution signs 90 days after conversion

Urban Extension

The overuse of stop control is also a problem in urban areas. To attempt to quantify the differences between the uses of a stop-controlled intersection and an uncontrolled intersection, a similar study will be completed on two cities: Ames and West Des Moines, Iowa.

A video log of all the intersections in the City of Ames has been created as part of this project. Along with the help of a data collection program, this video log was used to create a traffic control database for the City of Ames. (A database for the City of West Des Moines had already been developed previously.) These databases will make it possible to compare the safety at controlled intersections to that of uncontrolled intersections. An analysis similar to that of the rural analysis will be completed.

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Safety Assessment of Installing Traffic Signals at High-Speed Expressway Intersections

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ABSTRACT

This paper reports the status and preliminary results of an ongoing study investigating the safety benefit of signalizing intersections of high-speed (50 mph and greater) divided expressways. Before and after analysis was conducted and compared to empirical Bayesian (EB) techniques. A safety performance function was developed using negative binomial regression for a control group of 67 non-signalized intersections and compared to the performance of 20 intersections signalized between 1994 and 2001. Cross-sectional analysis was performed on 67 signalized and 67 non-signalized locations using negative binomial regression analysis to control for traffic volume. The paper reports on the effect of signalization on crash frequency and compares the results of the EB method with traditional statistical techniques.

Keywords: empirical Bayes—expressway intersections—traffic signals

BACKGROUND

This paper reports on an ongoing study of the safety effectiveness of installing traffic signals on high-speed expressways. A high-speed expressway is defined as a roadway with at-grade intersections with at least two lanes of traffic traveling in each direction that are separated by a median, with a speed limit of 50 mph or greater. The study sought to determine the safety effectiveness of installing traffic signals along the expressways. The study also evaluated empirical Bayes (EB) as an alternative statistical approach (e.g., effort vs. benefit).

At-grade expressway intersections are the location of many serious crashes and are typically controlled by a stop sign or a traffic signal. At stop-controlled intersections, as traffic levels increase, cross-street drivers may be forced to accept increasingly shorter and fewer gaps. At light traffic levels, mainline drivers may experience unnecessary delay if signal-controlled. Adding signals may not increase safety, as many would expect; rather, the types and severities of crashes may shift. If signalized, turn lanes may be used to separate some movements to reduce some rear-end crashes. Intersection skew is also a factor, particularly at angles of less than 75°. Research is underway (e.g., National Coorperative Highway Research Program [NCHRP]) to examine geometric improvements that might be used to increase traffic flow and enhance safety in lieu of signalization or grade separation.

This study used traditional statistical methods to examine the safety effectiveness of signalization and compares the results with those computed using the EB statistical method, widely noted as the state of the art in crash analysis, as it reduces the effect of small data samples (resulting in regression to the mean) and provides better estimates than older methods. Traditional methods include cross-classification and before and after analysis using pooled data sets. Cross-classification was used to compare the safety performance of non-signalized intersections (including intersections prior to signalization) to that of the signalized intersections (including intersections after signalization), accounting for covariates using regression techniques.

RESEARCH OBJECTIVES

As mentioned, signalizing a high-speed expressway intersection is likely to change crash type, but does not necessarily reduce crash rate or severity. To provide a baseline for comparison, the first objective of the study was to assess the safety impact of signalization at high-speed expressway intersections using traditional statistical methods.

The second objective was to compare the results (and input requirements) using EB analysis to the traditional methods. As Bayesian methods are new to many practitioners and have yet to find widespread implementation, the comparison should be useful to analysts considering various statistical approaches.

RESEARCH APPROACH

For the present study, an initial set of intersections was selected from a previous study of signal phasing to perform a preliminary analysis. The first task was to identify the expressway intersections in Iowa, a rather nontrivial exercise, as centralized data for road characteristics are maintained by the Iowa DOT while information on signals is maintained by local agencies that own and maintain them. This database contained information on the number of lanes, access control, median type, speed limit, and control devices[1] on road segments. The first step in identifying additional candidate intersections was to select all expressway and intersecting roads in the state. Next, segments were examined for indications of traffic signals. These segments were spatially compared to a separate database of known intersection locations (nodes). If any node was found to be within 200 ft of any signalized high-speed expressway segment, the node was labeled and stored as a high-speed intersection location. The method also designated some nodes erroneously (see Figure 1). These extra intersections were manually removed from the database. From remaining candidate intersections, aerial imagery was examined to verify the presence of a traffic signal.

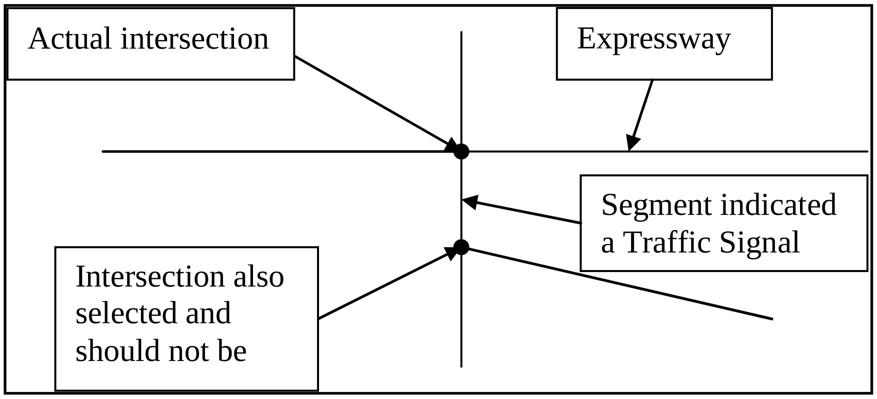

Figure 1. Intersection identification problem

While examining the aerial photographs, data were collected for each intersection. Figure 2 displays high-resolution aerial photography of the intersection of IA 28 and Park Avenue in Des Moines. Imagery used for the project ranged from six-inch to one-meter resolution. From the imagery, number and length of turn lanes, median type and width, and skew angle can be measured. More information about the intersections (traffic volume and date of expressway construction) was obtained from the Iowa DOT road network database.

Figure 2. Aerial image of IA 28 and Park Ave

[1] The database was not completely consistent with regard to identifying presence of signal, and did not include signal installation or modification date and hence was not capable of identifying all of the intersections of interest in the state.

In the absence of a statewide intersection control database, date of installation had to be approximated. To do this, crash records were examined for each intersection, and presence or absence of signal control was noted. Figure 3 illustrates the method used to approximate date of installation. Date of installation was identified to coincide with consistent reporting of signal presence in the crash database. In the example, note that prior to 1996 most crash reports indicated no signalization. Similarly, after 1996 stop control was reported infrequently. It is unclear why all crash records are inconsistent with regard to traffic signal presence. Therefore, judgment was used to approximate dates of installation. The final list of installation dates was shared with state DOT personnel and, in one case, follow up communication was initiated with local officials to confirm the installation date.

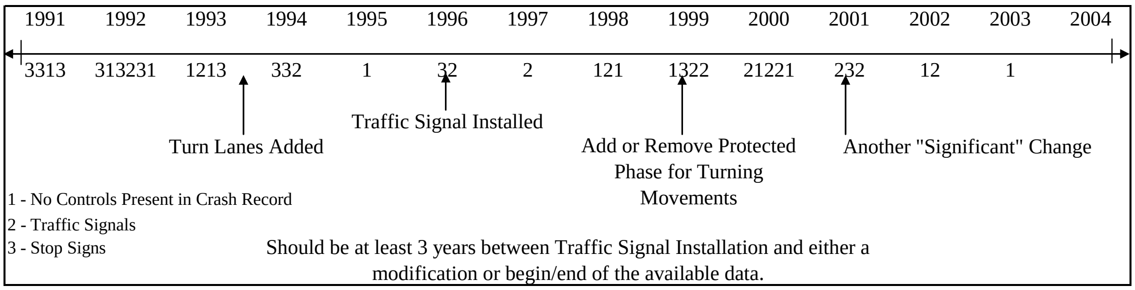

Figure 3. Example of estimating the date of traffic signal installation

To create a comparison group of non-signalized intersections, intersections were selected where minor road volumes fell within the range of minor road volumes on the signalized dataset. Aerial images at each of the intersections were examined for turn lanes, median, speed limit, skew angle, and traffic volume, trying to match as closely as possible the signalized intersections to facilitate pair-wise comparison. To verify that the comparison group consisted of only non-signalized intersections, aerial imagery was examined. In a few instances (see Figure 4), there was evidence of signalization. In these cases, alternate imagery was obtained and examined. For example in Figure 4, there is a faint indication that a signal may be at the intersection. From Figure 5, signals can be clearly identified.

Figure 4. Aerial image signalized expressway intersection

Figure 5. Higher resolution image

After the study intersections were identified, crashes were selected for each. Crashes were initially spatially selected (within 150 feet, the distance commonly used by the Iowa DOT in their rural analyses). However, spatial proximity does not always indicate relevance of crash to intersection. Figure 6 illustrates this. Each circle shows the approximate number of crashes at an intersection. The blue circle indicates the number of crashes that meet the attribute representation of “intersection-related.” The yellow circle shows the number of crashes that are within the spatial proximity. The red circle shows the number of crashes that meet both (attribute and spatial). Each classification gives a significantly different number of total crashes at the intersection. Several items from the crash data were also used to indicate “intersection-related” crashes. Values for some items are indicative of intersection proximity (at or near intersections, group one), while others indicate the type of crashes that generally occur or do not occur at intersections (i.e., right-angle, rear-end, parked vehicles, and driveway related crashes, group two).

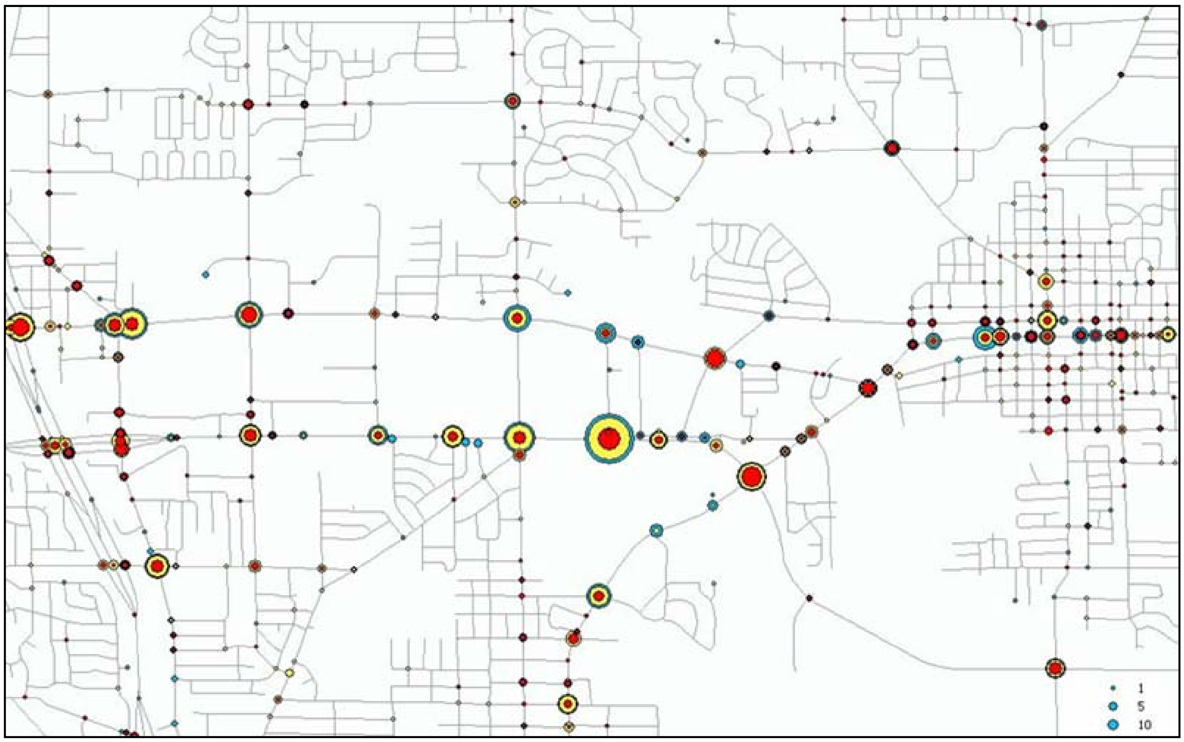

Figure 6. Intersection crashes

Figure 7 shows how the number of intersection related crashes would vary based on selection criteria over the time of interest of this study. Methods illustrated include the following:

- Crashes within 500 ft of the intersections (500)
- Crashes where one group of properties indicate intersection-related crashes and within 500 ft (possible at 500)
- Crashes where both groups of properties indicate intersection-related crashes and within 500 ft (likely at 500)
- Crashes within 150 ft of the intersection (150)
- Crashes where one group of properties indicate intersection-related crashes and within 150 ft (possible at 150), used for the present analysis
- Crashes where both groups of properties indicate intersection-related crashes and within 150 ft (likely at 150)

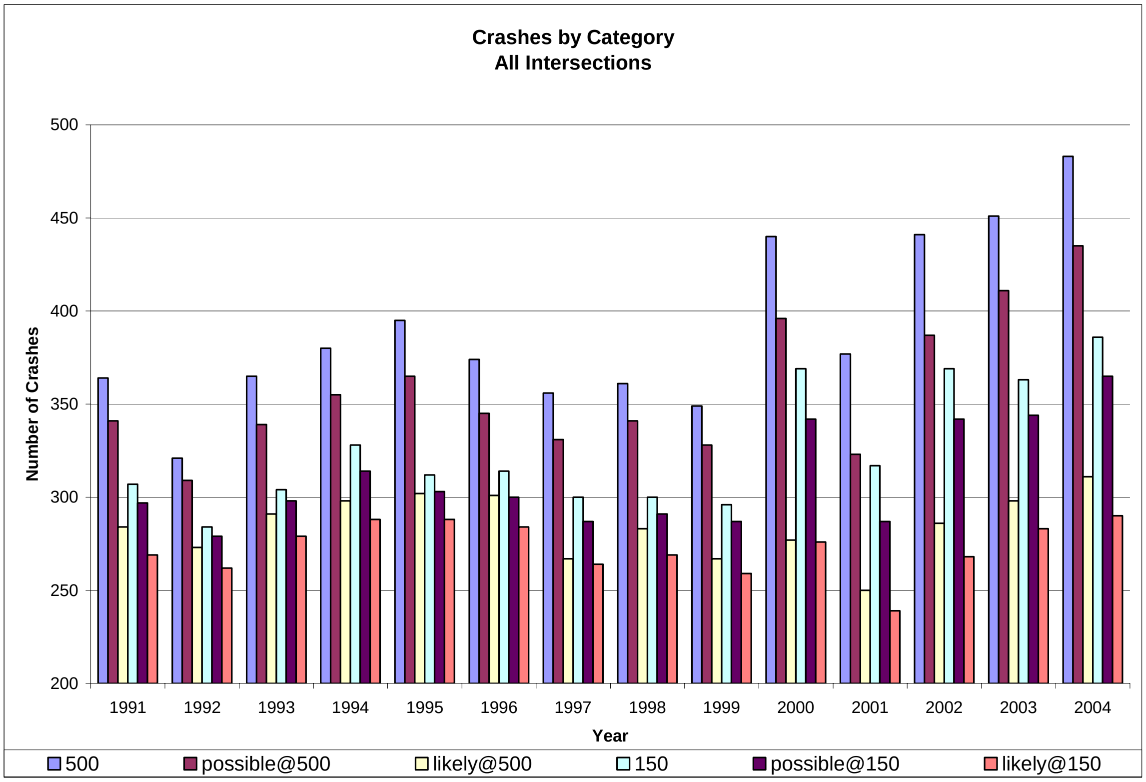

Figure 7. Total number of crashes by classification category

It should be noted that crashes prior to 2000 were located based on a distance from a node (typically an intersection). After 2000, crashes were located spatially along the highway segment. Nodes (prior to 2000) and cartography (after 2000) were updated periodically, resulting in some discrepancies between crashes located on older versions to more recent crashes, and more importantly to underlying road cartography and attributes. To account for these spatial inaccuracies, intersection locations were located using both 1998 and the 2003 alignments. Another issue with creating a second set of intersections is illustrated in Figure 8. If a second intersection location is offset from the expressway and/or from the side road, crashes should be selected that are outside the 150 ft tolerance. (Intersection-related criteria may have eliminated these crashes, but has not been checked at this time.) Within the study timeframe, the minimum crash reporting threshold also changed. Furthermore, the crash reporting form itself changed in 2001. In the changeover, approximately 5,000 crashes were reported on the old form and have not been changed to the new form (and consequently are not in the database). During this period, the crash database architecture also changed. To account for the report form change, a second list of crash properties were created and separated into two groups that are similar to the previous groups.

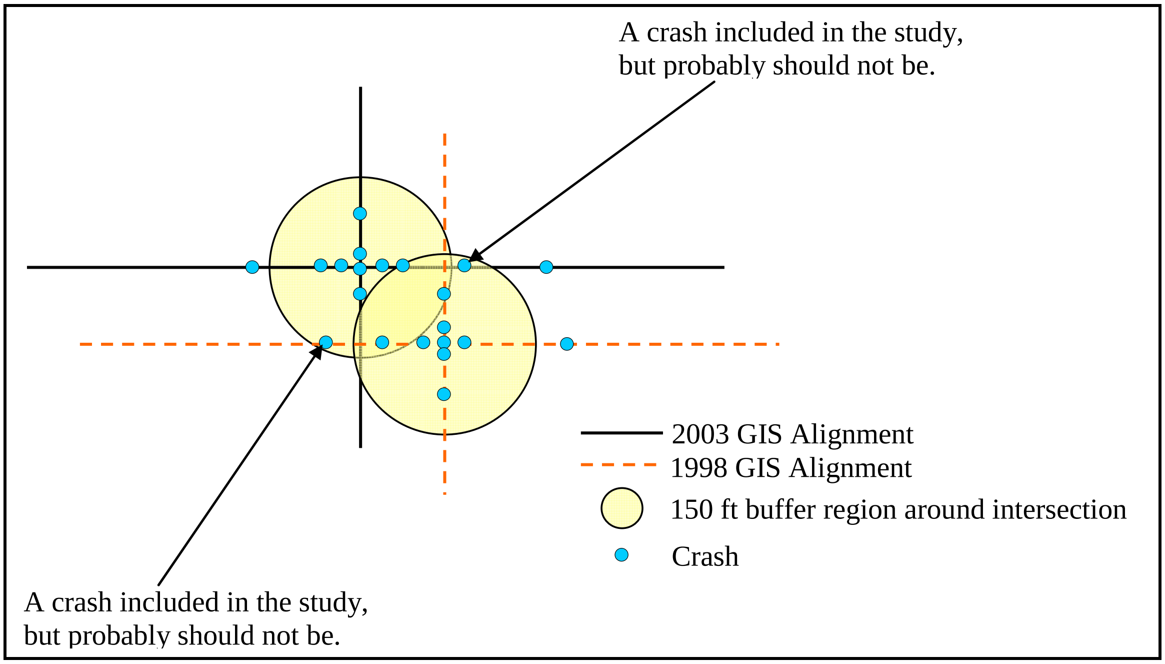

Figure 8. Crash selection problem

Once all the data were collected, analysis began by analyzing only the intersections that had at least three years of crash data before and after the traffic signal was installed. This limited the number of intersections to 33. At some locations, construction or significant widening occurred within the three years of installation, leaving only 20 intersections for before and after statistical analysis. A control group of non-signalized intersections was also created with 67 intersections. Using SAS software, an initial negative binomial regression model was created for both the before and after signal installation periods and the control group to produce safety performance functions (SPFs). The SPFs modeled the total number of crashes for each intersection as a function of traffic: daily entering vehicles (DEV). Using the SPFs, EB was used to reduce the impact of regression to the mean. Additional variables are to be included in future analysis.

To increase the number of intersections in the analysis, a cross-classification statistical method was used to compare all signalized intersections to similar non-signalized intersections. Sixty-seven signalized intersections in Iowa were appropriate for comparison. Sixty-seven non-signalized intersections were selected from the list of comparison intersections based on minor road volume. SPFs were created for both the signalized and non-signalized intersections.

RESULTS

Results indicate that the 20 high-speed signalized intersections in Iowa had an overall crash rate of 0.79 crashes per million entering vehicles prior to installation and 0.76 after. Before installation, these intersections experienced 237 total crashes and 241 total injuries, with an average DEV of 14,300. The crash and injury severity follows (injury severity is not limited to the same crash's severity: i.e., a minor injury may have occurred in a major injury crash):

- 2 fatal crashes with 3 fatalities
- 11 major (incapacitating) injury crashes and 16 major injuries
- 47 minor (abrasion) injury crashes and 87 minor injuries

- 60 possible (unknown) injury crashes and 135 possible injuries
- 117 property damage only crashes

After installation, these intersections experienced 253 crashes and 208 injuries with an average DEV of 15,400. The breakdown of crash and injury severity follows:

- 3 fatal crashes with 4 fatalities
- 12 major injury crashes and 15 major injuries
- 37 minor injury crashes and 60 minor injuries
- 69 possible injury crashes and 129 possible injuries
- 132 property damage only crashes

Figure 9 shows a graph of the total number of crashes by DEV for the 20 intersections used in the simple before and after analysis. The arrows show the relationship of an intersection from the before period crashes to the after period. The green vertical lines indicate a reduction in the crash rate. The red horizontal lines indicate an increase in the crash rate. The purple lines indicate approximately no change in the crash rate. The shaded curved lines are models from SAS that represent the actual crashes of the before and after periods. The diagonal lines represent equal crash rates (crashes per DEV). The graph shows that intersections below a total of 15,000 DEV tend to have a crash rate that decreases in the after period. Intersections that have DEV greater than 15,000 tend to have a crash rate that increases in the after period. These models confirm this.

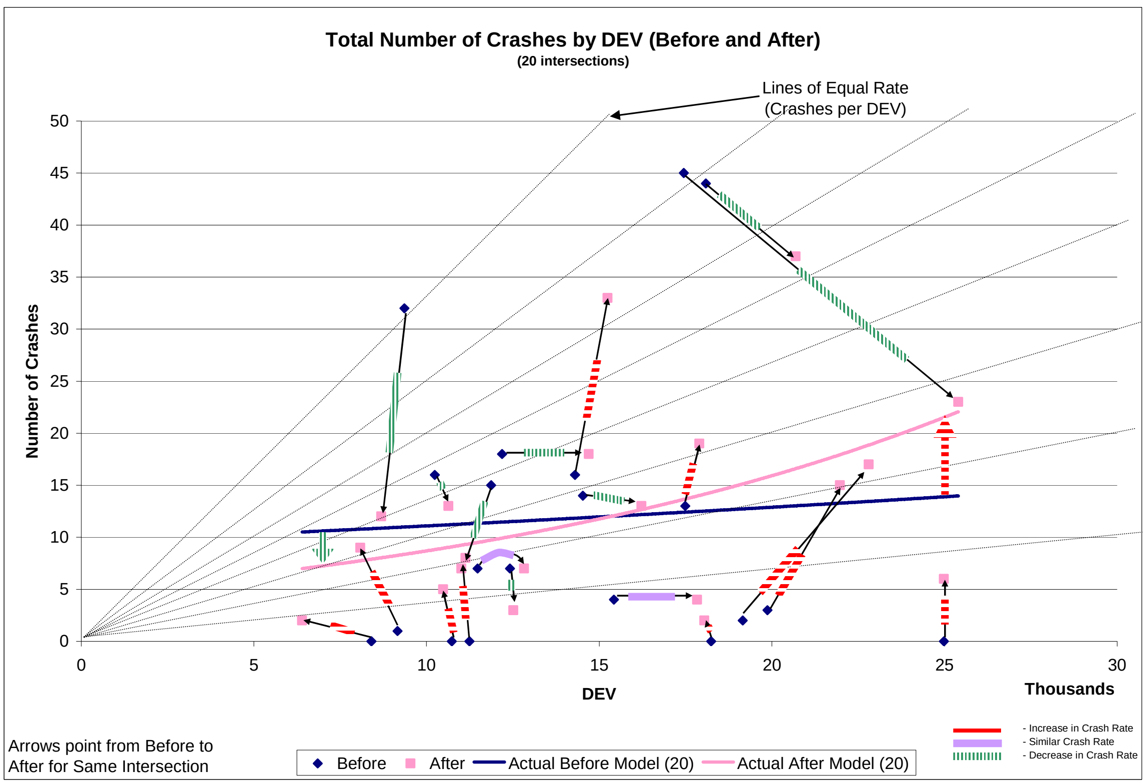

Figure 9. Total number of crashes before and after signal installation

Figure 10 is a graph representing the same information as that in Figure 9; however, EB has been used to create a weighted estimate of the before crashes. Unlike the traditional analysis, EB considers the control group model and the actual number of crashes as information and weights each to get a better estimate. EB adjusts the crashes at lower DEV intersections more so than the higher DEV intersections. Naturally, the intersections with the greatest difference in crashes from the model changed a larger amount. The arrows indicate the shift from the EB before-period estimate of crashes to the actual number of after-period crashes. EB reduced the estimated effect (decreased the slope of the arrows) of installing signals, except at a few intersections. However, EB did not affect the direction of change resulting from installation, except at one intersection.

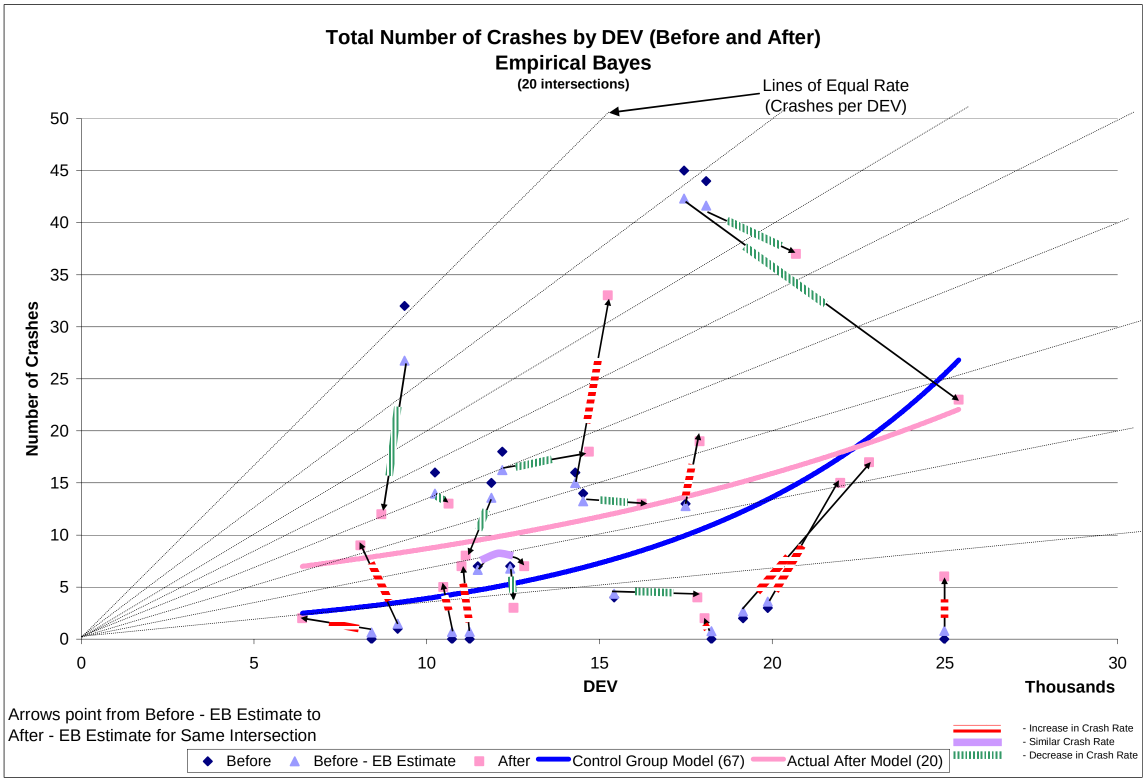

Figure 10. Total number of crashes before and after signal installation, EB applied

Results from the 67 similar expressway intersections' cross-classification analysis in Iowa indicate an overall crash rate of 21.3 crashes per hundred million entering vehicles (HMEV) at non-signalized intersections for the latest three years of crash data. The intersections experienced 488 total crashes with 363 injuries in the same time period and an average DEV of 14,500. These intersections had a fatal crash rate of 0.47 crashes per HMEV and a fatality rate of 0.53 fatalities per HMEV. The fatal and injury crash rate was 15.1 crashes per HMEV.

The signalized intersections experienced 1,209 crashes with 637 injuries and an average DEV of 18,300. This correlates to an average overall crash rate of 84.3 crashes per HMEV. These intersections had a fatal crash rate of 0.47 crashes per HMEV and fatality rate of 0.47 fatalities per HMEV. The fatal and injury crash rate was 29.4 crashes per HMEV.

Figure 11 shows a graph of the total number of crashes by DEV for the 67 comparison intersections. Unlike Figure 9, the SPF for the signalized intersection indicates a crash rate increase at lower DEV amounts. At 22,000 DEV, the SPF for the non-signalized intersections crosses the signalized SPF and has the higher crash rate. Note that there were few non-signalized intersections with more than 20,000 DEV, whereas there were several signalized intersections with over 30,000 DEV.

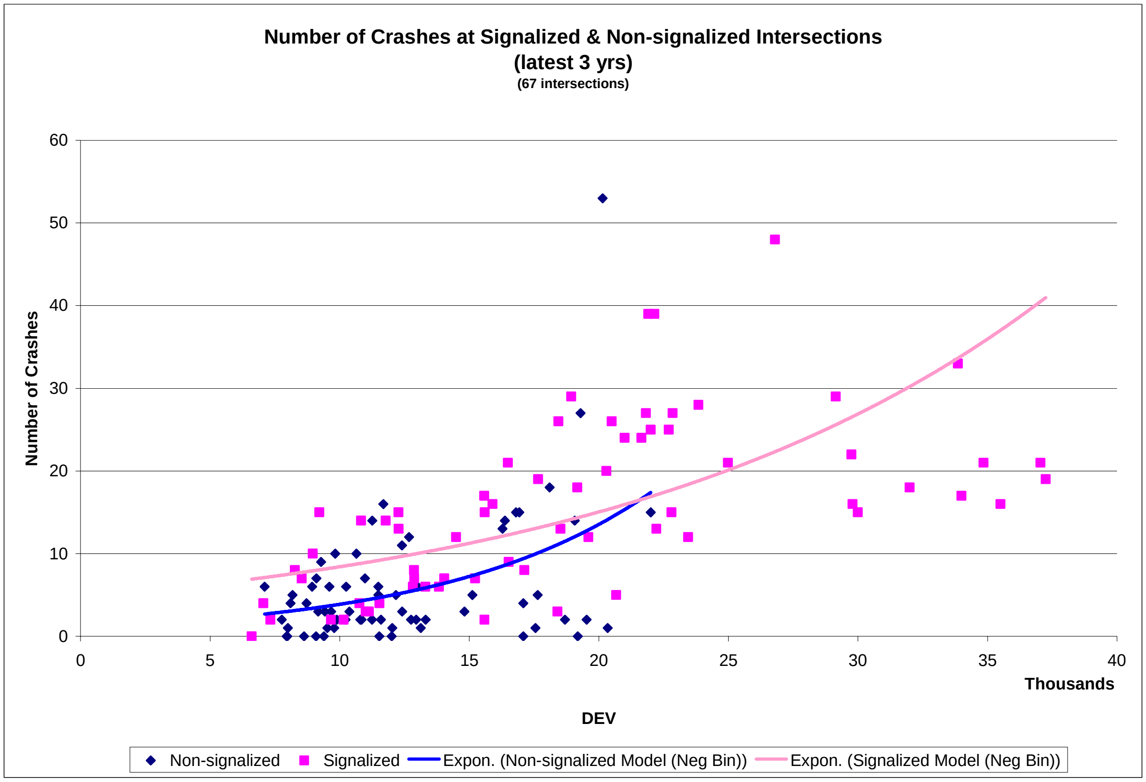

Figure 11. Total number of crashes at non-signalized and signalized intersections

CONCLUSION

The results are mixed for the safety impact of installing traffic signals. The before and after analysis shows more crashes with fewer injuries occurred after the traffic signals were installed. Both the traditional and the EB results indicate that installing traffic signals reduced the number of crashes at lower DEV amounts for the 20 intersections involved in the before and after analysis. At higher DEV amounts, traffic signal installation seems to have increased the number of crashes. Upon looking at the results from the cross classification analysis, the non-signalized intersections had fewer crashes with a lower crash rate than the signalized intersections. However, the non-signalized intersections had higher fatal crash and fatality rates. As can be seen from the data and models in Figure 11, the non-signalized model indicates fewer crashes in the Iowa DEV domain.

EB had minimal impact on the results of the before and after analysis. EB reduced the crash rate change (decreased the slope) for most of the intersections. EB did not greatly increase the time needed to perform the analysis.

Signals and Meters at Roundabouts

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ABSTRACT

With the emergence of the modern roundabout as an effective form of traffic control in the United States, more and more information is needed when situations arise in which additional traffic control is required. The signalization and metering of roundabouts can relieve congestion during peak hours of the day as well as possibly provide safer access for pedestrians and cyclists.

Although the signalization or metering of a roundabout may prove to be effective in solving access and congestion issues, it is not common in the United States and few have been installed. Of the roundabouts where signals or meters have been installed, these were done with little or no formal experience. With that in mind, a need exists for professionals to understand the basics of roundabout signalization and metering.

Several tasks were accomplished to generate a set of guidelines, and upon completing these tasks several options for the signalization and metering of roundabouts were found, including options for means of control, time of operation, and approach control. Unbalanced flow and pedestrians are also discussed, since these have been found to be the reasons for most signalization or metering. Several existing signalized and metered roundabouts are detailed from current literature and survey responses and were compiled in a table as part of the guidelines. Using all of the information from survey responses and literature seven guidelines were established, including the following:

- Observe and obtain data
- Review data
- Identify main concerns
- Choose means of control
- Choose time of operation
- Choose approaches to control

Key words: meters—roundabouts—signals

INTRODUCTION

With the emergence of the modern roundabout as an effective form of traffic control in the United States, more and more information is needed when situations arise in which additional traffic control is required. The signalization and metering of roundabouts can relieve congestion during peak hours of the day as well as possibly provide safer access for pedestrians and cyclists (Robinson 2000).

Signalizing and metering traffic at a roundabout can be considered reverse thinking. Roundabouts are installed to gain greater capacity and lower delays, and an added signal defeats this purpose. It should also be made clear that a signalized or metered roundabout is still far better than a regular signalized intersection. Even with some loss of capacity and greater delays, they still offer the benefits of improved safety over signalized intersections (Baranowski 2004).

There have been several reported cases of roundabout signalization in the United States. In Florida and Utah, pedestrian-actuated traffic signals have been installed, while in Maryland, a metering device is being considered to help ease congestion during certain periods of peak traffic flow (Sides 2000). Although the Florida and Utah locations have worked well, they were installed without a set of general roundabout signalization and metering guidelines. If a set of guidelines were developed, it would help professionals make decisions when considering signalization or metering at a roundabout while still providing the flexibility of good intersection design engineering.

PROBLEM STATEMENT

The signalization or metering of roundabouts may prove to be effective in solving access and congestion issues at roundabouts. Although several roundabouts in the United States have used traffic control signals, and metering is being seriously considered at one site, there is still a need for a set of guidelines to aide in the decision for implementation. Such signalization and metering appears to have fairly widespread use in Europe and Australia.

RESEARCH OBJECTIVES

To develop a set of guidelines applicable to the installation of traffic control signals and meters at roundabouts, several objectives were established. The primary research objectives were the following:

- Obtain general information and methods on the signalization and metering of roundabouts
- Determine locations and features of roundabouts that have traffic signals and meters in the United States, Europe, and Australia
- Determine the reasons for signalization and metering
- Determine locations where signalization and metering have been planned or could help operations
- Highlight the results of traffic signalization and metering and discuss their effectiveness
- Develop a set of guidelines for traffic control signals and meters at a roundabout
- Gain feedback on the developed set of guidelines from known roundabout experts in roundabout design and operations relative to the applicability of the guidelines
- Finalize a set of guidelines applicable to the signalization and metering of roundabouts

SCOPE

The finalized set of guidelines on signalization and metering of roundabout is only applicable to what is known as the modern roundabout and does not consider rotaries or traffic circles. Cases involving roundabout interchange signal coordination and signalizing for pedestrians were also not considered. Guidelines were based on information from individual case studies, literature, and recommendations from known authorities on the subject.

RESEARCH METHODOLOGY

To meet the objectives established and obtain the information needed to complete the research, five steps needed to be completed. The five steps included reviewing the literature, developing a state of the practice survey, developing guidelines, obtaining comments from experts on the developed guidelines, and finalizing a set of guidelines.

Literature Review

A literature review of documents relating to the signalization and metering of roundabouts was completing using resources from the library and internet. Approximately seven documents were used in the review, with a vast majority of the documents coming from countries outside of the United States.

State of the Practice Survey

A web survey was created to gain information by asking known experts in the field of roundabout design and operation to provide the information listed below relative to traffic signalization at roundabouts.

- Location of existing signalized roundabouts
- Type of signal system in place
- Location of signal system
- Main cause for signalization
- Effect of volume on the decision to signalize
- Effect of the location on the decision to signalize
- Human factors
- The addition of signs and striping with the installation of the signal
- Literature used to make decisions regarding implementation
- Peak hour considerations
- Software used to analyze the roundabout

The web survey was sent to known experts in the field of roundabout design and operations, as well as the Kansas State roundabout listserv. The survey is still available and can be accessed at www.cstevensandcomp.com/roundaboutsurvey.html. Seven surveys were received, including two indicating no experience with signalization or metering, two surveys detailing the same location, and one of the surveys providing great information on two pedestrian signals that were not within the scope of the research. Three sites pertinent to the research were detailed with survey responses and will be described in detail in a later section of this document.

Developing Guidelines

After completing the literature review and reviewing all of the survey responses, the information was used to establish a set of guidelines. The guidelines are not technical in nature and should be easily understood by someone not familiar with roundabout signalization and metering. The guidelines are mainly based from reoccurring patterns in literature and survey responses.

Expert Review

Another web-based form was created to obtain comments on the developed guidelines for the signalization and metering of roundabouts. The website address was sent to the survey respondents as well as other experts in roundabout design and operation and can still be found at www.cstevensandcomp.com/rbguidelines.html. Readers can access this page and comment on the guidelines.

Finalize Set of Guidelines

After obtaining comments on the developed set of guidelines, suggestions were used to finalize a set of guidelines for the signalization and metering of roundabouts.

BACKGROUND

Although signalization of a roundabout superficially defeats its intended purpose, there are still several underlying benefits that the geometry of a roundabout provides. The deflection of vehicles due to the presence of the central island and splitter island not only reduces entry speeds, but also eliminates right angle collisions. The FHWA concludes that reduced speeds within the roundabout gives drivers more time to react to possible incidents, reduces crash severity and allows for safer merging, and especially makes the intersection safer for drivers who are unfamiliar with the area (Robinson 2000).

Another key safety element that remains after a roundabout has been signalized is fewer conflict points. A four-leg, single-lane roundabout has eight conflict points, compared to an intersection with four two-lane entering roadways, which has 32 possible conflict points. If a roundabout is metered, it is important to remember that during non-peak times the roundabout still offers all of the benefits discussed in the previous section (Robinson 2000).

Some may ask why put in a roundabout if it is likely to be signalized? The reason is that although a signalized roundabout may not provide the same improved capacity and delay values, it is still safer than a normal signalized intersection. A signal or meter can extend the life of the roundabout while still providing added safety and aesthetic value (Robinson 2000).

Signalization and Metering Options

Once a decision has been made to signalize or meter a roundabout, there are several options. Means of control, time of operation, and approach control are the three most important signalization characteristics. Due to the differences between signalization and metering, these options vary. The different options for signalization and metering are described in Figures 1 and 2 in the form of a flow chart.

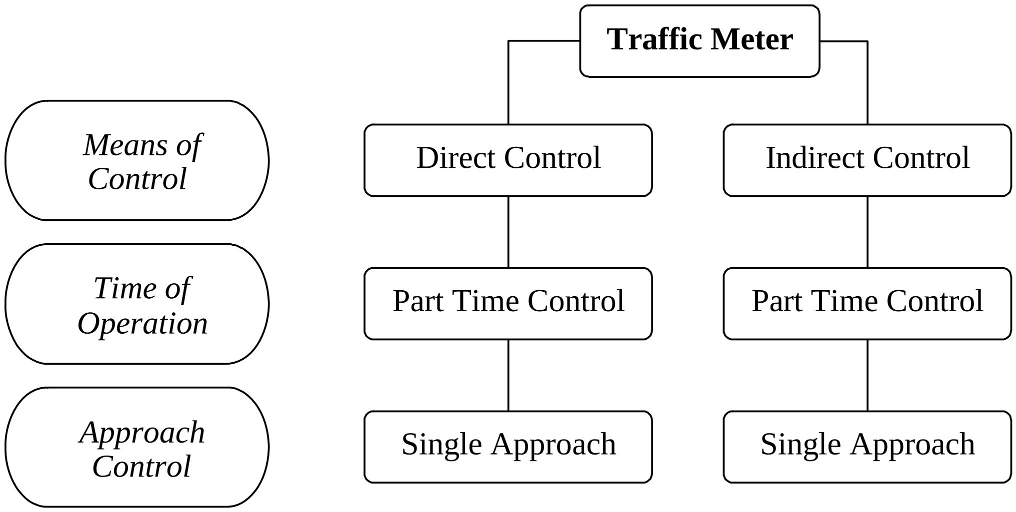

Figure 1. Traffic metering options

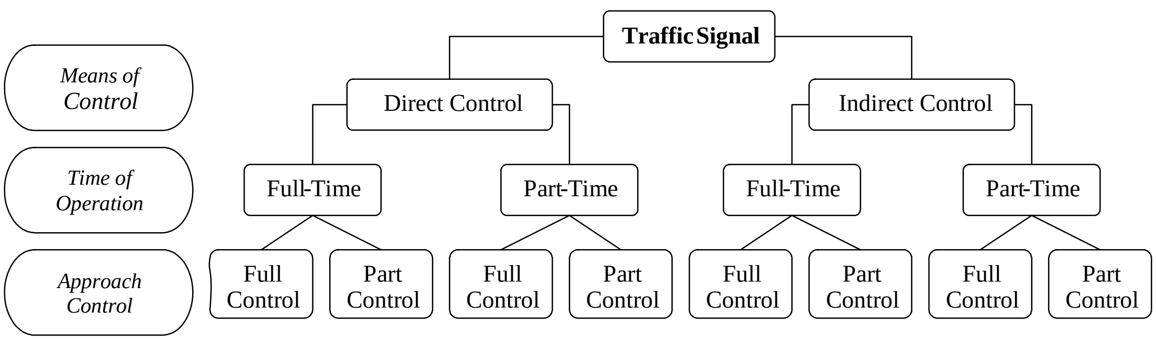

Figure 2. Traffic signal options

Means of Control

The means of control at a signalized or metered roundabout describes how the signal system controls entering and exiting vehicles. There are two main means of control at a signalized or metered roundabout: direct control and indirect control. A direct means of control affects both external and internal approaches, influencing traffic entering the roundabout as well as vehicles leaving from within the roundabout. For a metered approach, a direct means of control usually only affects vehicles entering the roundabout. Indirect control affects external traffic at a distance from the entry point of the roundabout. The circulatory traffic within the roundabout is not affected. Indirect control of vehicles is sometimes established with the addition of pedestrian signals, where crosswalks are at a distance from the roundabout entry (Hallworth 1992).

Time of Operation

The time of operation at a signalized or metered roundabout focuses on the period of time a signal or meter operates. There are two times of operation common at signalized or metered roundabouts: full-time and part-time. For full-time operation, the installed signals operate permanently and do not turn off during non-peak times. For part-time operation, the installed signal does not operate at all times. The signal is activated by time of day or by detectors. Detectors are usually placed at a distance from the controlled approach on a delay setting to determine when a queue has built up. For a metered approach, the time of operation varies according to queue length and dissipation; once a queue is no longer detected, metering signals will go blank and normal operation will resume (Hallworth 1992).

Approach Control

Approach control describes the number of approaches controlled with a signal or meter. There are two main types of approach control: full control and part control. Full approach control oversees all approaches of the roundabout. Part approach control at a signalized or metered roundabout is defined as control of one or more, but not all, legs of a roundabout while remaining approaches operate under right-of-way control. Roundabout metering signals usually control a single lane and are considered part control (Hallworth 1992).

Reasons to Signalize or Meter a Roundabout

A problematic roundabout usually falls into two main areas: unbalanced flow and high circulatory speeds. It should also be mentioned that there are several characteristics that each situation may create within a roundabout, such as loss of capacity, delays, elevated numbers of crashes, excessive queues, gap acceptance problems, and circulatory lockup.

Unbalanced Flow

Hudaart (1983) details unbalanced flow by stating, "The capacity of roundabouts is particularly limited if traffic flows are unbalanced. This is particularly the case if one entry has very heavy flow and the entry immediately before it on the roundabout has light flow so that the heavy flow proceeds virtually uninterrupted. This produces continuous circulating traffic which therefore prevents traffic entry on subsequent approaches." Huddart also states that in such situations, "signals can be used to initiate gaps in the traffic flow and hence balance the capacity" (Huddart 1983).

High Circulatory Speeds

With the need to weave and merge within a roundabout, sometimes higher than desired speeds can occur within the circulating sections and make it difficult for entering traffic. Another example of high speeds, brought to the author's attention by Tom Hicks of the Maryland State Highway Administration, is that of roundabouts with elliptical-shaped central islands (Hicks 2004). The discussion resulted in the conclusion that vehicles tend to increase speed on the longer side of the central island, causing incidents as the vehicle meet slower entering traffic on the short sides of the island. Signals can regulate traffic directly, if desired, by placing the signal within the circulating traffic, reducing speeds and allowing for safer and more efficient movement of traffic (Hallworth 1992).

Benefits of Signal Control

Although signalization and metering are against the nature of a true roundabout's purpose, they may provide solutions in such situations, as discussed earlier with unbalanced flows and high circulatory speeds. There are four main benefits to signalizing or metering a roundabouts, including shorter delays, reduced queue lengths, increase in capacity, and safety.

Shorter Delays

Delays at non-signalized roundabouts increase due to unbalanced flows or interactions with other intersections. Signals and meters can be used to balance delays and can reduce delays among a coordinated network (Hallworth 1992).

Reduced Queue Lengths

With unbalanced flow, queues can become excessive, sometimes backing up into other intersections or roadways. This situation is most likely to take place on frontage road off-ramps. A signal or meter can reduce queues by allowing queued traffic the right-of-way once a critical queue is detected (Hallworth 1992).

Increase in Capacity

If an excessive amount of traffic due to growth or new developments is entering a roundabout, traffic may not be able to circulate freely and can sometimes lock up. In situations where traffic is excessive, traffic signals may improve operations. It should also be noted that additional improvements to the roundabout design may be need to supplement the addition of signals (Hallworth 1992).

Safety

With the need for weaving and merging within a roundabout, internal circulatory speed can increase and hinder the ability of entering traffic to accept a gap. Traffic signals and meters better regulate the entry and sometimes exit of the roundabout, slowing the speeds of weaving and merging traffic, giving more time for drivers to react, and increasing safety at a roundabout (Hallworth 1992).

Roundabout Signalization and Metering in Literature

Several examples of signalization and metering are in the literature, and four are summarized in Table 1.

Table 1. Signalized and metered roundabouts from literature

Roundabout	**Means of control**	**Time of operation**	**Approach control**	**Problem areas**
Park Square, South Yorkshire, UK (Barnes)	Direct	Full	All	Excessive delays and accidents; high circulatory speeds
Newbridge, Scotland (Anderson)	Direct	Full	All	Traffic congestion, poor safety, high circulatory speeds, large queue lengths, and delay
Sheaf Square, South Yorkshire, UK (Barnes)	Indirect	Part	All	Delays due to excessive pedestrian traffic
Moore Street (Barnes)	Indirect	Metered	All	Unbalanced flow, excessive turning traffic, and delay

Guideline Development

The literature provided a detailed look into the process of signalization and specific triggers for added traffic control. Several themes resounded throughout the literature, including data collection and observations and the three categories of signalization options: means of control, time of operation, and approach control.

KEY FINDINGS

A web survey was created to gain information by polling known experts in the field of roundabout design and operations. Four survey responses detailing three roundabout locations are listed in Table 2.

Table 2. Signalization and metering from survey responses

Roundabout	Means of control	Time of operation	Approach control	Problem areas
Charles Street, MD, USA	Indirect	Full	All	Unbalanced flow
Clearwater, FL, USA	Indirect	Full	One	Unbalanced flow during spring break
Penn Inn, Abbot, UK	NA	Full	All	Unbalanced flow and excessive turning movements

Survey Responses

Charles Street at Bellona Avenue, Lutherville, Maryland, USA

A traffic signal was installed at the Charles Street roundabout because “the traffic flow is very unbalanced, with two legs of the roundabout representing over 90% of the flow” (Survey). The two heavy legs are northbound Charles Street and westbound Bellona Avenue. Charles Street did not have to yield the right-of-way very often due to the absence of conflicting flow, resulting in very few gaps in traffic for the westbound movement. A signal existed approximately 600 feet south of this intersection. The timing was changed to add a dummy phase to create artificial gaps for the northbound movement. Roundabout location, human factors, and pedestrians were not issues at this site. No new signing or striping was added, and no literature aided implementation. “We actually just went in the field with the signal technician and played with the timing until we reached the balance we were looking for” (Survey). No special software was used to evaluate signal timing, but aaSIDRA was used to evaluate the roundabout.

Causeway at Coronado at Mandalay, Clearwater Beach, Florida, USA

Information on this location was received from two respondents. The first author provided the following: A metering signal was installed on the main approach of this roundabout to create downstream gaps. The signals were located 150–250 ft upstream of entry and are actuated by a queue detector on the downstream approach. The respondent states: “This is an ambitious topic for which it might be difficult to obtain useful results at this stage. The number of roundabouts that are currently metered directly or with signalized pedestrian crossings is very small. Use of simulation as the sole basis for guidelines, if that is what is being considered, requires accurate modeling of the roundabout itself. Whatever the outcome, these guidelines need to fit in with ongoing operational and accessibility research” (Survey).

The second author also provided detailed information about the signal system: "On occasions there is a massive flow onto the island of Clearwater Beach. As a result of a deficiency of parking paces, congestion starts south of the roundabout continues northward through the roundabout and sometimes almost to the mainland. The roundabout was a vast improvement over the signals. This backup limited access to the roundabout from other legs. Installing metering signals on the causeway approach with queue loops on other approaches enabled the automatic stoppage for traffic from the mainland for 90 seconds wherever the queue conditions were met. It has been a great success. It only works late at night and Saturday lunchtime, Saturday late afternoon" (Survey).

Penn Inn, Newton Abbot, Devon, UK

Traffic signals were installed to balance complex turning movements on this overloaded roundabout. The site has recently been improved and updated, but traffic lanes are still not perfect. "Many drivers tend to drift laterally across the lanes" (Survey). The signals are located on all approaches adjacent to the circulatory roadway. The author was not sure of software or literature used to implement signalization.

No Experience with Signalization

Of the seven survey responses received, two of them stated that they had no experience with signalization. Three emails outside of the survey were also received expressing their lack of experience with signalized or metered roundabouts.

Guideline Development

The information obtained from survey responses present areas of concern at roundabouts, and although detailed information was not received regarding official data collection, observational comments were a standard in all three responses. Information was also given regarding the means of control, time of operation, approach control, and software packages used for analysis.

An Additional Location Where Signalization and Metering May Improve Operations

The roundabout at the intersection of Maryland state highway 100 and Snowden River serves both northbound and southbound lefts (see Figure 4). A traffic signal is located just south of the roundabout. When the green releases, vehicles travel towards the roundabout arriving in large platoons and take over the roundabout. Since there are no vehicles coming from the left that would cause entering traffic to yield, the vehicles proceed uninterrupted through the roundabout. This obviously prevents vehicles coming from the ramp of MD 100 from entering the roundabout and has caused a large queue to form that approaches the freeway during the evening peak hour (Niederhauser 2004).

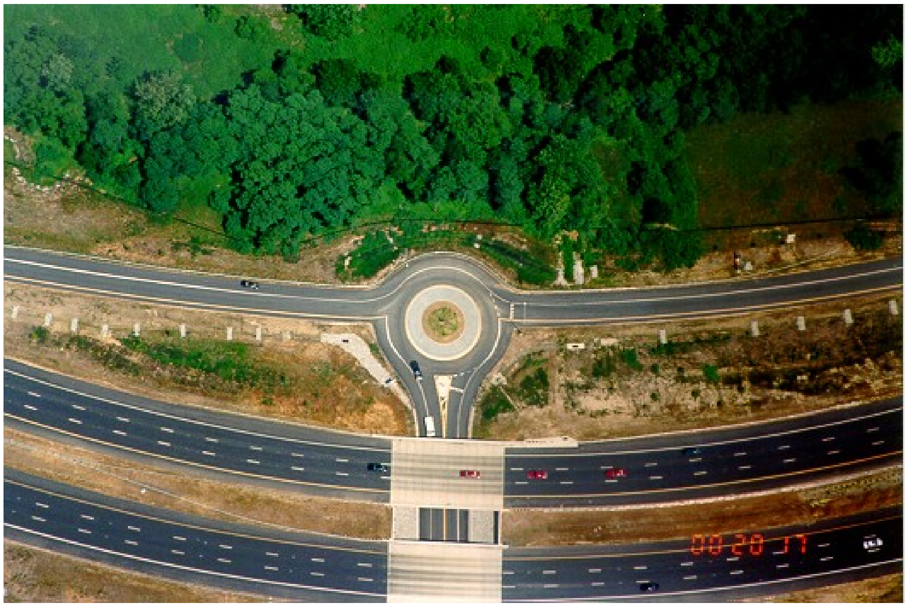

Figure 4. Aerial of MD 100 at Snowden River roundabout

GUIDELINES FOR THE SIGNALIZATION AND METERING OF A ROUNDABOUT

Before presenting the established guidelines for the signalization and metering of roundabouts, it should be mentioned that lack of detailed information and difficulty in obtaining specific information about signalized and metered roundabouts may limit the utility of the guidelines. Due to this, the guidelines are not technical in nature and can only realistically aim to inform engineers of their options and the corresponding process of implementing a signal or meter. Completing this research only leads one to ask, “What’s next?” and “How can these guidelines be improved to be of more use?” More detailed and accurate information is required and could be the next step in terms of research for this topic.

Observe and Obtain Data

Observation of a problematic roundabout will most likely provide much information into what is causing the problem. The following data should be obtained for further analysis:

- Entry and exit volumes
- Circulatory volumes
- Circulatory speeds
- Accident data
- Queue lengths
- Delays
- Headways
- Gap acceptance data

Review the Data

Upon obtaining data and observing the operation of the roundabout in question, it may be appropriate to compare data from previous signalization experiences when facing the decision to implement a signal or meter. Although not every situation is alike, the following values obtained from the literature can help characterize a problematic roundabout (see Table 3).

Table 3. Values from literature and survey responses for review

Roundabout or guidelines	Signal system	Geometry	Volume design/actual (vph)	ADT design/actual	Location of signal	Queue lengths	Other
Clearwater, FL	Traffic Meter	Oval 150/180 m	3,655/ NA	39,500/58,000 during spring break	150 to 250 from entry		
Park Square, UK	Signal	200 m across	NA/6500		At entry		12 injuries per year, 3:1 ratio of approaches during peak hour
Granville Square, UK	Signal	Oval 70/30 m	NA/3500-4000		At entry		
Moore Street, UK	Part-Time Signal		NA/3,300-1800 u-turn		25 m from entry		
Newbridge, Scotland	Signal	60 m diameter		NA/60,000	At entry	1.5 km (max)	30 mph

Identify Main Concerns

Upon observing the roundabout, a list of concerns should be established. The following questions can help identify those concerns:

- Is the roundabout suffering from unbalanced flow?
- If the roundabout has unbalanced flow, which approach has the heaviest volume?
- Do all approaches need to be signalized?
- Are there excessive speeds within the circulatory roadway?
- Are there more than a normal or expected number of crashes at this roundabout?
- Are there any geometric restrictions that will influence the placement of the signals?
- Does the roundabout affect, or is it affected by, other nearby signalized intersections or roadways?
- Is there a need to accommodate pedestrian traffic?

Choose Means of Control

Once the main concerns for signalization have been identified, it is time to choose the means of control for the intersection. Direct control is what the name implies; it directly controls traffic at the entry points of a roundabout. Direct control is one of the most common forms of control, and signals are usually placed at roundabout entries to alter the natural progression of a heavy volume that tends to prevent weaker approaches from entering the roundabout. Indirect control is when the signal is placed a distance from the entry of the roundabout. Indirect control often utilizes a metering signal, similar to a ramp meter, providing gaps in the traffic stream. In another form of metering control is the presence of an indirect pedestrian signal at a crosswalk. With indirect control, the literature has shown strengthening or widening the yield line is a common practice; this is because vehicles must still yield the right-of-way after moving through the indirect signal.

Choose Signal Operation

Signal operation can come in many forms, including full-time control, part-time control, and metering. Full-time control means that the signal will operate permanently and never switch off. There have been several noted cases of full-time control. In one case, the signal system had set cycle lengths for morning and evening peaks, as well as a cycle length for all other times. Another example of full-time operation is a traffic response control system that operates at different cycle lengths depending on actuated queue measurements. With part-time control, the signal usually only operates during peak periods. Part-time operation is usually the combination of a set period of operation or operation for a period of time due to the detection of excessive queue lengths. Metering of vehicles can also be an effective form of traffic control at problematic roundabouts. A metering signal is usually placed on the approach carrying high volumes that may prevent other vehicles from entering the roundabout. A traffic metering system can help alleviate this problem by detecting excessive queues and activating the signal, stopping traffic on the high-volume approach long enough for some vehicles to escape the impeded approach. Once the queue has dissipated on the impeded approach and is no longer detected, the signal will go blank and the roundabout will revert back to normal operation. This is similar to a ramp metering operation.

Choose Approaches to Signalize

Not all approaches need to be signalized. In many cases, metering one heavily traveled approach can be very effective. When establishing a queue balancing system, it is sometimes necessary to signalize all approaches. A queue balancing system monitors all approaches of the roundabout and uses the input to operate the signal or metering system.

CONCLUSION

After completing this research, one conclusion that can be made is the definite need for more investigation. One problem that seems to be facing engineers, planners, and government officials is the absence of a formal set of values to justify signalization or metering. A table similar to the one presented in the finalized guidelines would be an ideal place to begin. Obtaining more detailed information from the before-state of the signalized or metered roundabout can give engineers more values on which a decision to signalize or meter a roundabout can be based.

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Analyzing Crash Risk Using Automatic Traffic Recorder Speed Data

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ABSTRACT

In this paper, an evaluation is made of the relationship between crashes and various parameters of speed distributions as measured utilizing automatic traffic recorders (ATR) on highways in Iowa. Data on crashes were obtained from the Iowa DOT crash database; speed data are calculated from raw ATR data provided by the Iowa DOT. Standard statistical tests, including the T-test and the F-test, are used to compare the speed distributions as well as their mean and variance. A logistic regression model is developed to explore the relationship between the dependent variable crash risk and the explanatory variables' variance, road type, number of lanes, time of day, and day of week.

Key words: automatic traffic recorders—crash risk—speed distributions

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PROBLEM STATEMENT

For a number of years, there have been discussions regarding crash risk and the factors affecting it. One group argues that it is speed that kills, arguing from the energy equation ($e = ½ mv^2$), which holds that as the speed of a vehicle doubles, its energy quadruples, or from physiology, which holds that as speed increases, there is less time to react to problems. Another group argues that it is the variance of the vehicle stream (i.e., the differences in speeds within that stream) that causes crashes; if everyone were traveling with the same velocity, there could be no crashes. The purpose of this paper is to report on an effort to evaluate just how crash risk is affected by speed, variance, and other characteristics of the vehicle stream.

The Iowa DOT maintains a network of automatic traffic recorders (ATRs) on various classes of highways across the state. Figure 1 presents the Iowa ATR locations. There are a total of 142 ATRs, of which 72 collect speed data; these speed data are available beginning in 1998. The Iowa DOT also maintains a base of crash data that includes these years.

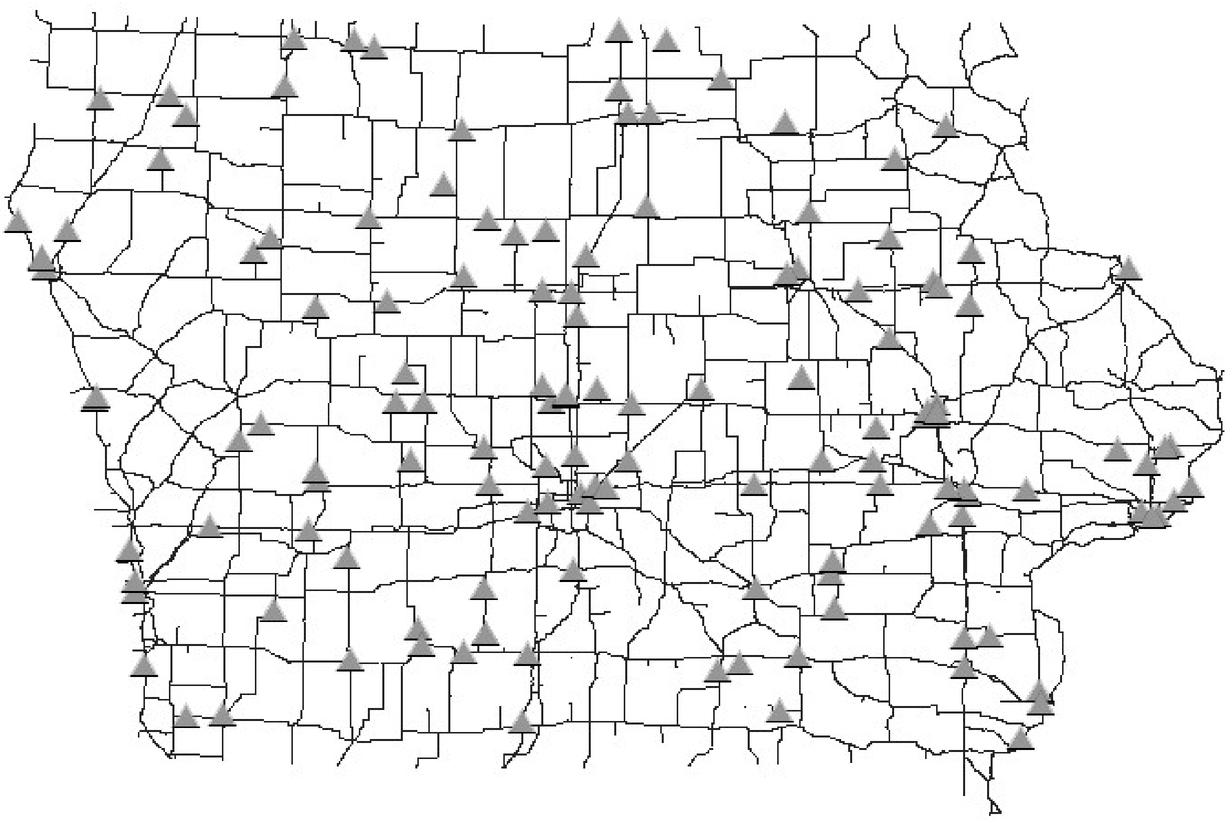

Figure 1. Iowa ATR locations

This research has two related objectives. The first is to establish a method for using and analyzing speed data from automatic traffic recorders. The second is to use these data to evaluate the relationships between crash risk, speed, and speed variance.

LITERATURE REVIEW

Perhaps the most exhaustive and likely the most often-cited research into the relationship between crash risk, speed, and variance was conducted in 1964 by David Solomon of the Federal Highway Administration (Solomon 1964). In his evaluation of data from 1955 and 1956 on two-lane and four-lane rural highways, he reported that the relationship between crash risk and speed is a U-shaped curve, with its minimum at about 65 mph. He also reported that the crash risk versus speed variation curve reached a minimum at about 8 mph above the average speed. These results would indicate that there is an optimal speed in terms of crash risk, as well as a benefit to keeping a speed that is close to that of other vehicles.

Other studies that have addressed the speed or variance issue, or that have raised questions regarding Solomon's findings, include West and Dunn (1971), Cirillo (1968), Kloeden et al. (1997), and Kloeden, Ponte, and McLean (2001). In Kloeden et al. (1997), the researchers found that crash risk increased exponentially for vehicles traveling above the urban area speed limit of about 35 mph. In Kloeden, Ponte, and McLean (2001), the researchers found "the risk of a free travelling speed passenger vehicle being involved in a casualty crash, relative to the risk for a passenger vehicle travelling at an average speed, increased at greater than an exponential rate. No evidence was found of a U-shaped risk curve whereby slower vehicles were also at a greater risk." In both of these studies, the researchers used a case-control methodology in which the speeds of vehicles involved in injury crashes (as estimated by forensic specialists) were compared to the speeds of vehicles not involved in crashes. Also of interest to the current study is their assessment of the Solomon and Cirillo study conclusions. Kloeden, Ponte, and McLean (2001) consider (as have others) that there was a bias in the Solomon and Cirillo studies that generated the U-shape of their risk curves. The source of the bias common to both studies is their use of a speed measured at a single point in a section but applied to the overall section. In Solomon's case, the average section length was 17 miles, with a maximum length of 91 miles. Also, with regard to Solomon's study, Kloeden, Ponte, and McLean (2001) considered two other likely sources of bias to be the possible underestimation of pre-crash speed by drivers and the possibility that crashes at driveways and intersections overcontributed to the number of low-speed crashes.

In Kloeden et al. (1997), the researchers found that the risk of crash involvement did not change for speeds of 60 kph (roughly equivalent to 35 mph) and below, and that crash risk doubled "for each 5-km/h increase in travelling speed above 60 km/h." The study approach was the case-control method.

In Kloeden, Ponte, and McLean (2001), the researchers focused on rural roadways with speed limits of 80 kph or greater (roughly equal to 50 mph). Among other qualifying criteria, the research focused on injury crashes and required a minimum of 10 control vehicles for each case vehicle. The study conclusion was that the risk of crash involvement on rural roadways more than doubled (actually 2.2 times) for a 10-kph increase in speed above the mean speed of the traffic stream. Finally, they recommended that the enforcement of speed limits be increased and that it be coupled with a reduction in or elimination of the tolerance in enforcing speed limits.

RESEARCH METHODOLOGY

The current study utilizes a modified case-control approach. The cases are stipulated as the crashes occurring on a roadway segment. The selected roadway segments are chosen according to the following criteria:

- On freeways, the segment extends to the nearest interchange on each side of the ATR location. It is assumed that there will be no change in volume and likely no change in flow speed without the interferences of the interchanges.
- On expressways and undivided roadways, the segment extends on either side of the ATR to the first side road that has a traffic volume of 10% or more of the mainline roadway's volume, or that has any crashes reported.

A database of coordinates was obtained from the Iowa DOT and was utilized in ArcGIS to select the study segments. Using the selected segments and the Iowa DOT crash database, still within ArcGIS, crashes along the segments were identified; they were then joined to records that included the date and time data. These selected records were compiled into a single database (for each ATR) using Microsoft Excel. A Visual Basic program developed for the project was used to perform the following tasks:

- Determine the date and hour immediately preceding the crash for the study case; determine the same hour one week earlier for the control case
- Access the six years of speed data; determine the mean speed, standard deviation, variance, and volume for the case and control hours
- Check for missing or erroneous data within the database and provide messages as appropriate
- Enter the calculated values into the ATR's Excel database

The next step in the process was to compare the variances of the speed distributions for the case and control hours, using the F-test to determine whether the differences were significant at the 95% confidence level (one-tailed test at 0.025). Because most tables of the F-test parameter have no detail beyond a degree of freedom of (typically) 120, an F-test calculator located on the internet was used to calculate the p value for each pair of distributions. P values returned from this calculator and within the range of the tables were compared to the values in those tables and were found to be identical.

Because large selection sets are subject to a phenomenon called false discovery, a Benjamini-Hochberg False Discovery Rate (FDR) correction was applied to the F-test results. This process involves determining the rank (order) of the p value of each pair of distributions, dividing each individual value's rank by 1,760 (the total number of pairs), multiplying each p value by this quotient, and finally comparing to the 0.025 criterion for acceptance of the null hypothesis (that there is no difference between the variances).

The results of the initial analyses were processed for input into the statistical software package SAS for further analysis. Because of the nature of the data, logistic regression was used to determine the relationships between the variables. The specific nature of the data that make logistic regression the modeling method of choice is that many of the data are binary outcome variables, although quantitative variables can also be included. For example, the crash/no-crash variable has a binary outcome; a crash either happens or it doesn't. Binary outcome variables (in SAS terms, categorical variables) used in the analyses are as follows:

- Crash (or not)
- Type of roadway: freeway, expressway, or undivided roadway
- Special times of day: morning rush period, evening rush period, late night
- Weekend (or not)

Speed and variance were quantitative variables that were analyzed in the model. Various combinations of variables were modeled in SAS 9.1 using logistic regression.

KEY FINDINGS

The 72 study segments had 1,769 crashes, for which there were speed data in both the study and control hours. Using the basic F-test comparison of the distributions, 445 (25%) of the case-hour distributions had (statistically) significantly larger variances than those of the control-hour distributions. In the balance of the comparisons, there was no significant difference between the variances (case-hour versus control-hour), or the control hours had larger variances than did their associated case hours. After the Benjamini-Hochberg FDR correction was applied, 375 (21%) of the case hours had significantly larger variances than the control hours. As before, the balance of the comparisons were either not significant or had larger values for the control hours.

The data for all study hours (case and control) were separated by type of facility (freeway vs. non-freeway), and descriptive statistics were computed using the Excel data analysis tools. These values are presented in Table 1.

Table 1. Summary statistics and T-test results

	Mean speed			Variance		
Facility type	**Case hour**	**Control hour**	**T-test**	**Case hour**	**Control hour**	**T-test**
Freeway	64.65	67.14	Yes	50.52	45.14	Yes
Expressway	59.15	59.74	No	50.64	50.00	No
4-lane undivided	51.17	50.99	No	29.81	26.58	No
2-lane	57.37	58.25	Yes	41.03	41.04	No

Mean speed and variance values are the means of the case hour or control hour speeds or variances.

The T-test result of "yes" indicates that the case-hour parameter (speed or variance) was significantly different from the control-hour parameter.

An evaluation was made of how deviation from the mean speed compares for the case and control hours. For this evaluation, the percentage of mean speeds that were 15 or more mph below the mean speed and those that were 10 mph or more above the mean speed for the category and type of facility were considered. Table 2 reports on the results of this evaluation.

Table 2. Variation from mean speed

Facility type	**Percent varying from mean speed**			
	15 + mph below		**10 + mph above**	
	Case	**Control**	**Case**	**Control**
Freeway	9.03	3.54	0.00	0.00
Expressway	3.54	3.09	0.46	0.23
4-lane undivided	0.00	0.00	0.80	0.80
2-lane	0.69	0.34	0.23	0.34

These results indicate a correlation between speeds below the mean speed and a greater risk of crashing, especially for freeways.

In the last analysis, logistic regression was performed; the analysis results coincide with the results of the previous statistical tests. A full model was run first, with all potential explanatory variables (both categorical and quantitative) being examined. The best-fit model included the quantitative variable mean speed at the 5%-level of significance (this level of significance was used throughout). A second series of model runs was made to evaluate each type of facility independently. In this series of evaluations, the full models were run; the only significant (in statistical terms) model was for the freeways and only the mean speed entered the model. Finally, a third series was run with only the variance modeled; again, the only significant model with the variance was for the freeways. It should be noted that although the variance did enter the model, the Wald Chi-Square statistic was a relatively low value of 10.8; this compares to the Wald Chi-Square statistic of 90.2 for the speed model. The Wald Chi-Square test compares the quotient of the estimated parameter divided by the estimated variance to the Chi-Square distribution. The two logistic regression models for the freeways are as follows:

- Probability (crash risk = 1) = exp (4.8756 - 0.0738 mean speed)
- Probability (crash risk = 1) = exp (-0.1747 + 0.00367 variance)

CONCLUSIONS

The results of the statistical analyses indicate that for the freeways there is a significant difference between the mean speed in the control hour and the mean speed in the case hours. As might be expected, the mean of the case-hour mean speeds is lower (2.49 mph) than during the control hour. The negative sign on the speed coefficient could be interpreted as indicating that as the mean speed increases the crash risk is lower. Because of the method of data storage and the resulting limits on the analyses, one should only infer from the results that crashes are more likely when the mean speed is lower than the free-flowing, non-crash mean speed for the same segment.

Also as might be expected, the mean of the case-hour variances is greater than that of the control hour. This result is consistent with the findings of Solomon, Cirillo, and West and Dunn, that variation from the mean speed of traffic on a roadway is (at least) a contributing factor to crash risk.

Because of the nature of the ATR data, it may not be possible to reach a definitive conclusion as to the relationship between crash risk, speed, and variance. Further research is needed to better characterize the role of speed in crashes. A methodology should be developed to access and utilize exact speed and vehicle action data from vehicle event data recorders. This data could then be compared to the distributions during the pre-crash hour.

LIMITATIONS

The most significant limitation on the speed data as used in these analyses is that they are only available on an hourly basis. It would considerably facilitate the analyses to have speed data for shorter periods, such as 10-minute or even 5-minute intervals. Some of the Iowa DOT's ATRs have the capability of reporting every vehicle's speed; unfortunately, these data are not routinely saved once they are processed and thus were not available.

NEXT STEPS

An avenue of investigation that promises to be fruitful is capture and analysis of the pre-crash data from automobile event data recorders (EDR) that have been part of some Ford and General Motors vehicles with airbags since 2001. As the number of EDR-equipped cars in the vehicle mix increases, the likelihood of having an EDR-equipped car crashing in an area where speed data are available should increase as well. A long-term study could be designed to work with the Iowa DOT to receive and process the files with the individual speed data as they are captured; they could then be used with the EDR data to provide a more refined evaluation of crash risk.

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Cost Effectiveness of Design-Build, Lane Rental, and A + B Contracting Techniques

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ABSTRACT

Many state DOT specifications are generally prescriptive, in that they describe how contractors should conduct certain operations using minimum standards of equipment and materials. These prescriptive specifications, known as method specifications, have performed admirably in the past. However, rehabilitation and reconstruction projects, especially in a rapid renewal scenario, demand more creativity and innovation. Prescriptive specifications, used in conjunction with traditional procurement and contracting, do not properly foster this innovation. This study compares relevant performance criteria for three alternative contracting techniques (A + B, lane rental, design-build) and for traditional contracting. The research methodology involved surveying national experts who rated each innovative contracting method for each performance factor on each of the project types. Results indicate that design-build and A+B contracts are the most effective methods when time is the primary driver of cost or when complex design issues require interdisciplinary coordination. Because design-build appears to hold much promise for dramatically accelerating schedules, we utilized in-depth personal interviews of project team members involved in a design-build urban corridor reconstruction project in Minnesota. Interview data suggest the following issues need to be addressed as use of design-build contracting continues to gain acceptance:

- Determination of appropriate level of design completion prior to issuance of the request for proposal
- Co-location of project team members
- Definition of responsibilities for quality control and quality assurance
- Adaptation of traditional state procedures, procurement systems, forms, and project information handling methods to better fit design-build delivery philosophy

Key words: best practices—innovative contracting—performance

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BACKGROUND AND PROBLEM STATEMENT

Many governmental agencies charged with delivering public infrastructure have been experimenting with innovative contracting methods over the last several years. Many of the more common techniques have recently been formally approved for use by the Federal Highway Administration (FWHA 2002). One particular federal program, Special Experiment Projects-14 (SEP-14), has helped to define and clarify many of these new innovative contracting methods accurately to ensure that the processes and practices involved with innovative contracting are implemented effectively. The primary objective of SEP-14 was to review specified innovative contract techniques as they were applied to specific projects, which were monitored closely to measure the effectiveness of innovative contracting compared to the traditional design-bid-build method or other acceptable methods.

The specific innovative contracting methods under investigation in the SEP-14 report are the following:

- A + B with an incentive/disincentive (I/D) option
- Lane rental
- Design-build

A + B contracting is sometimes referred to as cost plus time contracting or biparameter bidding. For the remainder of this report, the term A + B contracting will be used.

Each of the innovative contract types listed above have proved to be acceptable practice by contractors and transportation agencies in highway construction. A + B contracting (both with and without incentive/disincentive) and lane rental contracts have been labeled as acceptable practices by the FHWA since 1995 and are no longer considered experimental. These two contract types were subjected to the FHWA's protocol of approving new innovative contract types (Special Experiment Projects-14) from 1990 to 1995.

The FHWA is continuing to develop guidelines and regulations for design-build contracting as mandated by section 1307(c) of the Transportation Equity Act for the 21st Century (TEA-21), enacted on June 9, 1998. The TEA-21 required the Secretary of Transportation to issue regulations to allow design-build contracting for selected projects. The regulations list the criteria and procedures that will be used by the FHWA in approving the use of design-build contracting by state transportation departments. The regulation does not require the use of design-build contracting, but allows state DOTs to use it as an optional technique in addition to traditional contracting methods. Use of design-build was formalized by the Federal Highway Administration in 2002 with the issuance of the Final Rule (Federal Register 2002).

Now that innovative contracting methods have been practiced for several years in many states and the federal government has recognized and defined many standard practices for innovative contracting, the need has arisen to examine and compare the effectiveness of different innovative contracting methods to each other, instead of independently comparing them to the traditional method of delivery.

Several states have researched innovative contracting methods with the objective of developing a protocol to assist agency personnel in selecting the most effective contract type based on certain project parameters. There have also been reports by various non-governmental organizations and institutions that have researched one or more innovative contracting techniques. The main reports and most comprehensive studies are outlined in the following paragraphs to develop an integrated summary and synthesis of current thinking on the comparative effectiveness of innovative contracting methods.

Although extensive literature and agency reporting is available for review, for brevity we have highlighted a few of the most important, comprehensive, and/or innovative reports in the following literature review. The discussion below reflects a mix of comprehensive studies examining a variety of contracting methods as well as some studies that focused on a single contracting method, looked at performance criteria, or used project criteria as a basis for selection. This represents a reliable cross-section of the types of reports extant in the literature.

The South Dakota DOT hired Trauner Consulting Services, Inc., to assist them in defining criteria and guidelines that South Dakota could use to determine the most effective innovative contracting methods (South Dakota DOT 1996). Trauner researched such innovative contract types as I/D, A + B, and lane rental for their study. The South Dakota guidelines based the selection primarily upon project criteria with some consideration of performance characteristics such as cost, time, and road user cost.

The Ohio DOT has internally written a manual (Ohio DOT 2003) to assist in developing construction contracts through a selection criteria process. The innovative contracting methods matrix included in the Ohio manual lists approximately 15 project types and assigns a yes/no assessment on the suitability of various contracting methods for each project type. They examine I/D, lump sum incentive, work day, liquidated savings, design-build, A + B, and warranty contract types in their report. Each contracting method is analyzed in a specific report section, which includes “Definition,” “Objectives,” and “Project Selection Criteria” to help define which practice best suits certain project parameters. There is little discussion of performance criteria other than what can be inferred from project characteristics.

The Utah Technology Transfer Center also generated a best practices guide for innovative contracts (Bolling and Holland 2003). The contract types that were examined in their report include design-build, A + B, lane rental, warranty, and job order contracting. This report is similar in style and content to the Ohio DOT manual, but offers perhaps more definitive discussions of the performance implications of different contracting types. The Utah center examined the impact of different contracting methods on five performance parameters: administration, risk, time, cost, and complexity. In addition, the Bolling and Holland report listed project parameters that would lend themselves to the different contracting methods.

A University of Minnesota report by Cadenhead and Hippchen (2004) examined the design-build contracting method as it is used in the highway construction industry. This report gave examples of past and current design-build projects, and attempted to describe project parameters where the value of design-build delivery could best be captured. The report also described the performance benefits that can result from using design-build contracts for highway construction projects.

A study by Shr et al. (2000) examined A + B contracting as it had been practiced since 1990 and determined that some loss of value or suboptimum contracting was possible if state departments of transportation did not place an upper and lower limit on the time parameter of the bid. Shr, Ran, and Sung followed up this study in 2004, adding that the factors of I/D costs should be added to road user costs methods and then optimized against the A + B (cost plus time) parameters in each of the bids received in order to choose the lowest cost option. This optimization process shifts much of risk to the contractor while maximizing the agency’s resources. However, the optimization modeling can be cumbersome.

The primary intent of the lane rental contracting method is to bring the cost of inconvenience to the public into the contract award equation. Under the lane rental contracting method, contractors are forced to consider and include both construction costs and the costs to the public in their bid. The effect of lane rental is similar to liquidated damages in non-transportation construction. Lane rental is particularly valuable when alternative routing and detours are unavailable, and when the time savings can be readily calculated in dollar terms. (Herbsman and Glagola 1998).

Many of the studies referenced above used either project characteristics or performance criteria, but few used both. Also, most of the studies were single-agency studies. The research project described below attempts to create a balanced comparison of contracting methods by utilizing both performance and project characteristics and using both quantitative national survey data and a qualitative case study.

RESEARCH OBJECTIVES

The purpose of this research project is to compare the effectiveness of four different contracting methods: traditional system of design-bid-build, A+B contracting, lane rental, and design-build. The objective of the research is to provide transportation managers and educators with insight and recommendations for use in transportation policy. The research project is comprised of two principal components: a national survey of DOT construction engineers and a case study of a successful design-build project. The purpose of the national survey is to provide insight into the project and performance factors related to the different contracting approaches as well as outline the suitability of each of these methods to different types of transportation projects.

The second component of the research effort is a case study of the reconstruction of Trunk Highway 52 through Rochester, Minnesota (ROC-52). The Minnesota Department of Transportation (Mn/DOT) has utilized a design-build approach for ROC-52, and the purpose of the case study is to investigate the project and prepare a set of recommendations to improve the administration of future design-build projects. Design-build was chosen for more in-depth case study analysis because it is the newest and perhaps least understood of the innovative contracting methods considered under SEP-14.

SURVEY RESEARCH METHODOLGY

The research methodology in this study used multiple methods of analysis, incorporating qualitative and quantitative techniques. The first step in the methodology was to identify relevant performance criteria. Mn/DOT has identified relevant performance factors to be used in determining project success. We chose a subset of those performance factors related to construction procurement and contracting value. The eight relevant performance factors include the following:

- Administrative costs are defined as the different types of internal costs Mn/DOT incurred in tracking processes: contract administration, inspections, reviews, right-of-way acquisition, and environmental assessment and monitoring.
- Construction costs include first costs, costs of change orders, cost of engineering and design, and environmental remediation. Questions regarding the construction costs:
    - Time refers to the overall length of time spent in project planning, funding/appropriations, design, construction, and extensions.
    - Management complexity refers to the relative difficulty of coordinating issues encountered over the course of the project, specifically management-related aspects of the project such as planning and establishment of scope, logistical challenges, utility relocation and coordination, adjustments to unforeseen problems that arose during execution of the project, etc.
    - Third party impact includes disruptions to businesses, schools, churches, residential neighborhoods, and other establishments or destinations along the route.
    - Road user costs refer to the costs incurred by the motoring public resulting from the project. Examples include accidents, driver time, and additional vehicle mileage due to detours.
    - Quality refers to the level of workmanship and the end products' performance versus what is expected by the owner, as well as the amount of post-construction callbacks and required maintenance of the facility.
    - Innovation includes the degree to which contractors are able to use new or less conventional concepts, methods, or materials on the project, and their flexibility to make design changes and pursue alternative ideas or techniques aimed at reducing cost and scheduled time.

Because contract effectiveness is moderated by project type, we incorporated several project types into this analysis. The project types were intended to cover a range of conditions, such as design complexity, road user costs, third party disruptions, etc. The nine project types chosen for analysis include the following:

- Major corridor realignment/expansion
- Multi-lane highway rehabilitation through a city, with detours
- Multi-lane highway rehabilitation through a city, under traffic
- Rural bridge replacement
- Metropolitan bridge replacement
- Two-lane highway resurfacing
- Mill and overlay
- Unbonded concrete overlay
- Preservation project with culvert replacement during two-lane highway resurfacing

The DOT construction engineers from each of the 50 states were sent blank templates for each project type and asked to rank the four different procurement methods from 1 (best) to 4 (worst) on each performance factor. Nineteen usable responses were received. The 38% response rate was deemed acceptable for reliability of the data. The individual rankings were reverse scored (1=4, 2=3, 3=2, 4=1) so that high effectiveness scores would correspond to effective contracting methods. The mean effectiveness scores for each contract type were analyzed using a pairwise t-test for comparison of means. The mean effective scores of the four contract methods for each project type are reported in the following section. Those mean scores that did not achieve statistically significant differences are noted.

KEY FINDINGS FROM THE SURVEY

For major corridor realignment/expansion projects, the performance effectiveness scores are as follows:

- A+B………………..22.55
- Lane rental…………16.92
- Design-build……….21.08
- Traditional…………19.45

The mean differences in performance evaluation scores between A+B and design-build and between design-build and traditional were not significant at the 0.10 level.

For multi-lane highway rehabilitation projects through cities with detours, the performance effectiveness scores are as follows:

- A+B………………24.68
- Lane rental………17.18
- Design-build……..19.28
- Traditional………..18.57

The mean differences in performance evaluation scores between lane rental and design-build, between lane rental and traditional, and between design-build and traditional were not significant at the 0.10 level.

For multi-lane highway rehabilitation projects through cities under traffic, the performance effectiveness scores are as follows:

- A+B………………24.03
- Lane rental………17.18
- Design-build……..19.26
- Traditional………..18.57

The mean differences in performance evaluation scores between lane rental and design build, between lane rental and traditional, and between design-build and traditional were not significant at the 0.10 level.

For rural bridge replacement projects, the performance effectiveness scores are as follows:

- A+B………………..23.40
- Lane rental………..16.13
- Design-build………20.55
- Traditional……...…19.86

The mean differences in performance evaluation scores between design-build and traditional were not significant at the 0.10 level.

For two-lane highway resurfacing projects, the performance effectiveness scores are as follows:

- A+B………………..23.50
- Lane rental………..19.13
- Design-build………17.46
- Traditional…………19.51

The mean differences in performance evaluation scores between lane rental and design-build, between lane rental and traditional, and between design-build and traditional were not significant at the 0.10 level.

For metropolitan bridge replacement projects, the performance effectiveness scores are as follows:

- A+B……………..….24.52
- Lane rental………..17.25
- Design-build……....20.19
- Traditional…………18.00

The mean differences in performance evaluation scores between design-build and traditional were not significant at the 0.10 level.

For mill and overlay projects, the performance effectiveness scores are as follows:

- A+B……………..…24.60
- Lane rental………..20.12
- Design-build………15.50
- Traditional…………19.72

The mean differences in performance evaluation scores between lane rental and traditional were not significant at the 0.10 level.

For unbonded concrete overlay projects, the performance effectiveness scores are as follows:

- A+B………………..24.92
- Lane rental………..19.26
- Design-build………16.64
- Traditional…………18.64

The mean differences in performance evaluation scores between lane rental and traditional and between design-build and traditional were not significant at the 0.10 level.

For preservation projects with culvert replacement during two-lane highway resurfacing, the performance effectiveness scores are as follows:

- A+B…………………23.97
- Lane rental…………18.26
- Design-build………..17.18
- Traditional…………..20.57

The mean differences in performance evaluation scores between lane rental and design build and between lane rental and traditional were not significant at the 0.10 level.

DISCUSSION OF SURVEY RESULTS

A+B contracts received the highest effectiveness score for each of the project types, and the differences in mean effectiveness scores were statistically significant in all comparisons of A+B to other contract types, except for major corridor realignment and expansion, where design-build mean effectiveness score was not statistically significantly different from A+B. This suggests that for all project types considered in this study, A+B contracts will create the greatest value when all relevant performance factors are considered. Of course, procurement protocols that do not allow for multi-attribute value consideration (e.g., low-bid awards) will not capture the optimum effectiveness that innovative contract methods have to offer.

The other three contract methods varied in effectiveness based on project parameters. For major corridor realignment and expansion, where third party disruptions and road user costs can be significant and designs can be complex, design-build scored very high compared to lane rental and traditional contracting. However, on projects with very little design input, such as overlays, design-build scored very low. The benefits of design-build are difficult to capture when the substantial time reductions from overlapping design and construction do not translate into lower road user costs or reduced disruptions to third parties.

We were somewhat surprised to find that design-build and lane rental did not score higher for multi-lane highway rehabilitation through cities, either under traffic or with detours. The complexity of sequencing and management of traffic, along with high traffic volumes typically found on these projects, would suggest value added from decreased project time and greater innovative capabilities. These findings might be explained by the use of equal-weighting performance criteria. Subsequent research should utilize a weighted performance criteria factor to give higher weight to the rankings attached to the most important performance criteria for each project type.

CASE STUDY RESEARCH METHODOLOGY

Investigation of the effectiveness of design-build on the ROC-52 project required the research team to conduct interviews of appropriate project personnel. The insight obtained from these interview sessions forms the basis of this case study. Information gained from the interview process was expected to be mostly qualitative in nature.

Prior to the interviews, a set of project-related criteria were identified as a means of comprehensively evaluating a project's performance. The questions presented in the interview were geared to address these different criteria as applicable to ROC-52. Specifically, we were interested in learning how the use of design-build delivery may have impacted the project. Ultimately, conclusions can be made about the effectiveness of design-build and recommendations can be made that will enhance the performance of future design-build projects.

There were nine core project criteria addressed in the interview sessions: administrative costs, construction costs, time, management complexity, disruption to third parties, road user costs, quality of project, funding flexibility, and innovation. Two general questions were also asked during the interviews to assess perceptions about different types of delivery systems and determine ways that the administration of ROC-52 could have been improved. Interviewees were not necessarily expected to be able to comment on all of these factors, but those who were selected for interviews were able to speak to most of them.

Each interviewee was chosen because he or she had considerable knowledge of the project and would be able to provide insight and suggestions. Interviewees for this case study were chosen after receiving input from Mn/DOT team leaders. Twenty-seven interviews were conducted during January and February of 2005.

CASE STUDY FINDINGS

A detailed discussion of findings from the case study exceeds the scope and space limitations of this paper. In general, findings suggest the following:

- Innovative contracting goes hand-in-hand with innovative financing because of the increased demand on agency cash-flows.
- Project staffing and district staffing must be coordinated because of high-intensity, (relatively) short-term demands on district operations.
- Quality control and quality assurance protocols and responsibilities must be clearly delineated prior to the start of construction.
- Co-location of designer-builder and agency district project engineers promotes exceptional teamwork and communication.
- First costs were thought to be slightly higher, but the percentage increase through change orders was far below typical contracts.
- Time reduction was exceptional (11 years at initial planning, 7 years at first design, 5 years at RFP, slightly less than 3 years actual).
- Road user, third party stakeholders, and the community in general were extremely pleased with the execution of the project.
- Placing public relations and community communication responsibilities with the design-builder was very effective.
- Some DOT personnel have difficulty with the reduced scope of design detailing (buildable vs. biddable documents).

- Input from the design-builder through the alternative technical concept review process led to greater innovation, resulting in both time and cost savings.
- Mn/DOT clearly accepted the risk of unknown conditions and environmental remediation, reducing lengthy negotiations on change orders or contract interpretations.
- Agency decision making and approvals should be delegated to the project level whenever possible.
- Speed of construction put strain on agency resources and processes, creating the need for new methods, new documents, etc.
- Speed of construction places a premium on information management and coordination for complex issues such as utility relocations, right-of-way acquisitions, etc.

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Secondary Accident Data Fusion for Assessing Long-Term Performance of Transportation Systems

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ABSTRACT

The research that is reported describes two critical steps in developing a methodology for extracting secondary accidents from police accident databases. The eventual goal of the research is to produce an incident progression model from which the number of secondary incidents could be determined. These steps include the following: (1) the development of models for incomplete incident data and (2) an analysis of site differences in maximum queue lengths and incident duration. The first step is important because it allows the use of intranet incident data that is complementary to police accident data. The second step is also important because it helps determine whether site-specific models are necessary instead of a single general model. The methodology involves the use of data fusion from accident data and intranet traffic reports. The project results in a near-term technology for analyzing the safety impacts of transportation assets.

Key words: accident data—data fusion—traffic safety

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RESEARCH OBJECTIVES AND BACKGROUND

The main objective of this research is to develop a readily deployable methodology for extracting secondary accidents from an accident database. This methodology should be robust so that it can be used for analyzing the safety benefits of different transportation systems. In order to accomplish this main objective, several other objectives need to be achieved. First, a convention for formatting accident reports needs to be developed so that the appropriate information from the accident database is extracted and formatted for the derivation of secondary accidents. Second, a methodology for processing intranet traffic reports need to be developed. This methodology transforms the readily available traffic reports into useful information that tracks the progression of individual incidents. Third, incomplete incident reports need to be modeled using an appropriate methodology. Fourth, site-specific data is analyzed to determine whether different models are needed from different sites. Finally, the main objective of extracting secondary accidents is achieved through the use of a master incident progression curve.

This methodology has many potential applications. Long-term safety evaluation is critical in the decision making process for maintaining and operating equipment in incident management activities. For example, there have been legislators in Missouri who question the value of such programs and desire for such assets to be maintained by the private sector. The Motorist Assist (MA) program or Freeway Service Patrol, for example, consist of assets such as the maintenance garage, fuel station, field-deployable equipment such as variable message sign (VMS) trailers, MA and emergency management vehicles (specialized trucks), records management computing systems, communications infrastructure, and personnel. Since the inception of MA in Missouri in 1993, the feedback regarding MA has been mostly informal, in the appearance of public comment forms returned from motorists serviced by the program. There is a need to assess the value of this program in a systematic and logical fashion. It is often difficult to make statistically significant conclusions concerning the short-term using accident data because of small sample sizes. But accident data is useful for long-term asset management and planning because they capture long-term trends. The use of secondary accident data for evaluating the MA program provides concrete evidence. Since safety is arguably one of the most important transportation issues, the impact of the MA incident management approach on safety is a primary concern for all asset management systems.

Secondary accidents are accidents that result from an existing primary incident. Many times these accidents occur at the end of queues that had developed from the primary incident. Quickly opening the highway after an incident reduces the potential for secondary accidents. It is easy then to see the value of analyzing secondary accidents when considering traffic incident management strategies such as intelligent transportation systems and the MA program. On the other hand, the effects of such systems on primary accidents would be much smaller because many of these accidents are caused by driver error, such as fatigue, intoxication, or aggressive driving. Therefore, traditional analysis of primary accidents and accident rates will not reveal the full potential of such systems.

The development of this methodology is based on previous work in the area of accident analysis and safety. An important paper on the analysis of secondary accidents is Raub (1997), in which the author presents a methodology for the temporal and spatial analysis of incidents on urban freeways in order to identify the secondary crashes. He found that more than 15% of the crashes reported by police may be secondary in nature. He also found that such crashes result from external distractions instead of internal distractions or driver perception error. For the analysis, Raub assumed an accident effect duration of 15 minutes plus the clearance time. He also assumed a distance of effect of less than 1,600 meters (1 mile). In other words, if an accident occurred within these temporal and spatial boundaries, the accident is considered to be secondary.

More recently, Moore et al. (2004) examined secondary accident rates on Los Angeles freeways using accident records from the California Highway Patrol's First Incident Response Service Tracking system, as well as data from loop detectors on Los Angeles freeways. They defined secondary accidents as accidents occurring upstream of the initial incident in either direction within or at the boundary of the queue formed by the initial incident. A fixed threshold of 3,218 m (2 miles) and 2 hours was used for forming this boundary. Several levels of filters served to eliminate erroneous data.

These two studies exemplify the use of fixed thresholds for extracting secondary accidents. Figure 1 graphs the progression of an actual incident, including fixed queue length and time thresholds superimposed on this progression. If an accident falls within the influence of the primary accident, i.e., the accident happened within the queue of the primary accident, then the accident is considered to be secondary. Progression refers to the growth and decline of the queue length as the incident progresses through the various stages. In general, the various stages of an incident that influence the traffic queues include the onset, the arrival of response teams, the clearance to the shoulder, the completion of clearance, and the normalization of traffic. The progression is a function of both the demand (traffic) and the supply (road capacity). With the demand changing constantly, it is clear that the assumption of fixed thresholds would not capture field conditions properly. Some would argue that, on average, the total number of secondary accidents can still be estimated accurately with fixed thresholds if the area of the fixed threshold rectangle is the same as the area under the progression curve. This argument also requires the assumption that accidents are independent from the location and time of the primary accident. For example, Figure 1 shows that the same number of accidents (three) is classified as secondary using a fixed threshold or the actual incident progression. However, by definition, secondary accidents differ in cause from primary accidents. Therefore, even if the average number of accidents is captured accurately with fixed thresholds, the accidents themselves are still misclassified. Referring back to the example and looking at the fixed thresholds, the number of secondary accidents is estimated correctly, even though accident B is a false positive and accident E is a false negative. The inadequacy in the use of fixed thresholds is one primary motivation for this research.

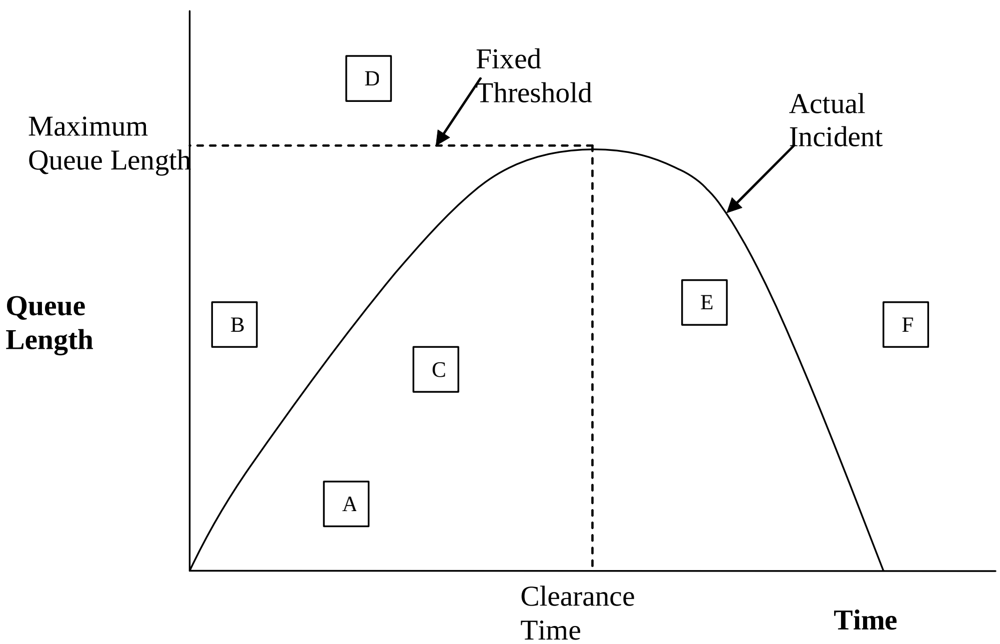

Figure 1. Fixed thresholds versus actual incident progression

Other articles relate to secondary accidents but do not address the extraction process directly. Karlaftis et al. (1999) examined the primary crash characteristics that influence the likelihood of secondary crash occurrence. They suggested that clearance time, season, type of vehicle involved, and lateral location of the primary crash are the most significant factors. The economic benefit of secondary crash reduction for the Hoosier Helper freeway service patrol program was also presented. There are several articles that address the magnitude and impact of incident delays. These include Garib et al. (1997), Giuliano (1989), Skabardonis et al. (1996), Morales (1987), Sullivan (1997), Smith et al. (2003), Lindley (1987), and Lee et al. (2003). Many of these articles try to estimate the impact of accidents. The present research, on the other hand, proposes the direct derivation of the impacts (queue lengths over time) from traffic reports.

RESEARCH METHODOLOGY

The primary source of data used in the methodology is the accident database obtained from the highway patrol in Missouri. The accident database for freeways is compiled by police agencies and consolidated by the state highway patrol. Table 1 shows an accident record snippet from I-70 in Missouri in 2002. The electronic accident records contain much more information than what is shown in Table 1. In addition, images of the four-page accident reports are also kept in the accident database. The primary fields that will be used in this analysis include name, direction, continuous log, date, severity, time, image number, and traffic condition. This is the consistent format used for accident analysis.

Table 1. Sample accident record of I-70 in Missouri

NAME	DIR	CONT LOG	DATE	ACCIDENT TYPE	ACCIDENT CLASS	SEVERITY	TIME	VEHICLE NO	VEHICLE TYPE	CIRCUMSTANCE	TRAFFIC COND
70	E	223.409	1/1/02	MOTOR VEHICLE IN TRAFFIC	REAR END	PROPERTY DAMAGE ONLY	350	1	PASSENGER CAR	DRINKING	NORMAL
70	E	223.409	1/1/02	MOTOR VEHICLE IN TRAFFIC	REAR END	PROPERTY DAMAGE ONLY	350	1	PASSENGER CAR	INATTENTION	NORMAL
70	E	223.409	1/1/02	MOTOR VEHICLE IN TRAFFIC	REAR END	PROPERTY DAMAGE ONLY	350	2	VAN	NOT STATED OR UNKNOWN	NORMAL
70	E	250.312	1/1/02	MOTOR VEHICLE IN TRAFFIC	REAR END	PROPERTY DAMAGE ONLY	1110	1	PASSENGER CAR	IMPROPER PASSING	NORMAL
70	E	250.312	1/1/02	MOTOR VEHICLE IN TRAFFIC	REAR END	PROPERTY DAMAGE ONLY	1110	2	PASSENGER CAR	NOT STATED OR UNKNOWN	NORMAL
70	W	15.57	1/2/02	MOTOR VEHICLE IN TRAFFIC	REAR END	INJURY	815	1	PICKUP	FOLLOWING TOO CLOSELY	NORMAL
70	W	15.57	1/2/02	MOTOR VEHICLE IN TRAFFIC	REAR END	INJURY	815	2	PASSENGER CAR	NOT STATED OR UNKNOWN	CON-GESTION AHEAD
70	E	234.229	1/2/02	RAN OFF ROAD-FIXED OBJECT	OUT OF CONTR OL	PROPERTY DAMAGE ONLY	847	1	PASSENGER CAR	NOT STATED OR UNKNOWN	NORMAL

In order to use secondary accidents as a performance measure, it is necessary to separate such accidents from the rest of the accidents. The police accident report contains a field that describes downstream conditions as “accident ahead” or “congestion ahead.” The difficulty with the police determining whether the accident is secondary is that they are limited spatially (at one location) and temporally (responding to the current accident). Since the effect of primary accidents can persist long after they have been cleared, it is difficult to determine at the scene of an accident whether it is due to recurrent or non-recurrent congestion. The use of the category “accident ahead” for finding secondary accidents would undercount the number of accidents while adding the category “congestion ahead” would severely overcount the number of accidents. Because of such problems, it is necessary to produce a methodology that can reliably distinguish the secondary accidents.

Traffic management centers and traffic news agencies can provide wide spatial coverage of incidents as well as track the incidents over time. They can use information from aircrafts, elevated traffic cameras, MA, emergency management (fire, ambulance, HAZMAT), and motorist calls. They can also monitor and update this information throughout the course of an incident. Such intranet traffic information can be independent from police information; therefore, such information can complement the accident database from the police. Data fusion helps incorporate all available information sources, including intranet traffic reports and the accident database. By analyzing individual traffic reports in detail, the reporting times of the incident and the dynamic locations of the back of the queue can be found. The difference between the initial and final times estimates the total incident duration, and the distance from the location of the incident to the back of the queue estimates the length of the roadway affected by the incident.

However, intranet reports need to be processed significantly to be in a usable format. The methodology for processing such reports is as follows. Pages of traffic reports are downloaded daily at regular, e.g., three-minute, intervals. A Unix script has been written to perform this automatically. These reports are then consolidated and parsed to extract pieces of information into specific fields, such as incident reporting time, incident type, and incident description. A major task is to extract the traffic information for a particular highway along a particular direction in the sequence they are reported in the files on a particular day. A computer program saves the information pertaining to a single incident through multiple reports in a single day. Since there is no unique identifier associated with the information pertaining to a particular incident, the lines containing information related to a particular incident need to be extracted through the use of keywords present in those lines and absent in other lines. There can be difficulties in this process, since traffic reports are human-generated and can include syntax variability as well as errors. As an example, consider the primary route eastbound interstate 70. In the reports, eastbound can also be expressed as “EB,” “E/B,” or “east,” and interstate 70 as “70,” “I70,” or “I-70.” There can also be descriptions of the route expressed in phrases such as “eastbound lanes of 70” or “east and westbound lanes of 70.” Figure 2 shows the result of this processing. The information pertaining to a single incident is tracked throughout the incident, giving the queue length as the incident progresses. Even though the processing of intranet traffic reports is laborious, a valuable incident dataset is produced.

```
3:03 PM EB I-70 BETWEEN CASS AND THE ENTRANCE TO THE KING BRIDGE, THE RIGHT LANE IS BLOCKED BY AN OVERTURNED TRASH TRUCK ;
4:02 PM EB I-70 BETWEEN CASS AND THE ENTRANCE TO THE KING BRIDGE, THE RIGHT LANE IS BLOCKED BY AN OVERTURNED TRASH
TRUCK...TRAFFIC BACKS UP NEAR SALISBURY. ;
4:35 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE CLOSED, DUE TO AN OVERTURNED TRASH TRUCK...TRAFFIC BACKS UP NEAR ADELAIDE...BROADWAY
TRAFFIC GETTING ONTO KING BRIDGE ALSO BLOCKED. ;
4:42 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE CLOSED, DUE TO AN OVERTURNED TRASH TRUCK...TRAFFIC BACKS UP TO EAST OF
GRAND...BROADWAY TRAFFIC GETTING ONTO KING BRIDGE ALSO BLOCKED. EB EADS BRIDGE TRAFFIC IS ALSO BACKED UP GETTING INTO ILLINOIS. ;
5:00 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE CLOSED, DUE TO AN OVERTURNED TRASH TRUCK...TRAFFIC BACKS UP TO EAST OF GRAND...4TH
ST TRAFFIC GETTING ONTO KING BRIDGE ALSO BLOCKED. EB EADS BRIDGE IS ALSO JAMMED FROM MISSOURI END OF THE BRIDGE INTO ILLINOIS. ;
5:04 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE CLOSED, DUE TO AN OVERTURNED TRASH TRUCK...TRAFFIC BACKS UP TO WEST OF
ADELAIDE...4TH ST TRAFFIC GETTING ONTO KING BRIDGE ALSO BLOCKED. EB EADS BRIDGE IS JAMMED FROM MISSOURI END OF THE BRIDGE INTO
ILLINOIS. ;
5:12 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE CLOSED, DUE TO AN OVERTURNED TRASH TRUCK...TRAFFIC BACKS UP TO WEST OF
ADELAIDE...4TH ST TRAFFIC GETTING ONTO KING BRIDGE ALSO BLOCKED. EB EADS BRIDGE JUST HEAVY ON THE ILLINOIS END. ;
EADS BRIDGE JUST HEAVY ON THE ILLINOIS END. ;
5:37 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE CLOSED, DUE TO AN OVERTURNED TRASH TRUCK...TRAFFIC BACKS UP TO EAST OF
ADELAIDE...4TH ST TRAFFIC GETTING ONTO KING BRIDGE ALSO BLOCKED. EB EADS BRIDGE JUST HEAVY ON THE ILLINOIS END. ;
5:41 PM EB I-70 ENTRANCE RAMP TO KING BRIDGE HAS BEEN REOPENED...BUT TRAFFIC STILL BACKS UP TO EAST OF ADELAIDE...4TH ST AT THE
KING BRIDGE ALSO REOPENED. ;
```

Figure 2. Example of intranet traffic report

Modeling Incomplete Incident Data

Many of the analyzed traffic reports are incomplete because the time of normalization is missing even though the clearance time is reported. For this reason, it is necessary to derive a model to estimate the incomplete incident progression data. Since detailed incident data are rare and valuable, it would be desirable to continue using the incomplete traffic reports. One way of accomplishing this is to extrapolate or model the incomplete incident data. The incomplete data usually contain a significant number of samples after the clearance of the incident, so the modeling of the incomplete portion of the data should not produce significant errors.

Intranet traffic reports from I-70 and I-270 in St. Louis, Missouri are used in this investigation. A total of 49 accidents have complete information. In other words, the accident is tracked from the beginning until traffic is normalized. Ninety-four accidents have some traffic information after the accident clearance, but not all of the information needed to verify traffic normalization. The 49 complete accidents are then used as a ground truth for testing the modeling process of incomplete incident data.

There are three assumptions that need to be made when using regression to estimate models of the incomplete traffic reports. The three assumptions are that (1) the model error has a mean of zero, (2) the error has a constant variance over all observations, and (3) the error corresponding to different points in time are not correlated (Ostrom 1978). If autocorrelation is present, the variances would be underestimated, which would result in overconfidence in the model.

A test dataset was constructed by taking the complete accidents and then artificially eliminating data samples after the clearance time. This test set tried to replicate the incomplete data and provided the ground truth. The performance of the models in estimating the missing portion of the progression curve was evaluated using this test dataset. Since the backup queue may not decrease linearly, second-, third-, and fourth-order polynomials were used for modeling the incomplete accidents. By using the test dataset mentioned, it was found that a third-order polynomial provided the best fit when compared to the second- and fourth-order polynomials. The third-order polynomial was able to reproduce the total delay estimates (or areas under the queue length/time curves) to within ±10% with an average difference of 1.4 % from the true value. While the average difference between second-order and real data was 5.3%, the difference between the fourth-degree model and real data was 6.5%.

Another criterion for evaluating the performance of the polynomial models was the R^2 value, which measures the proportion of the data that can be explained by the model. Figure 3 shows the R^2 values of several accidents being modeled by a second-, third-, and fourth-order polynomial. Figure 3 shows that the third-order polynomial provided the best fit for the accident data, since the third order results in the best R^2 over the entire test dataset.

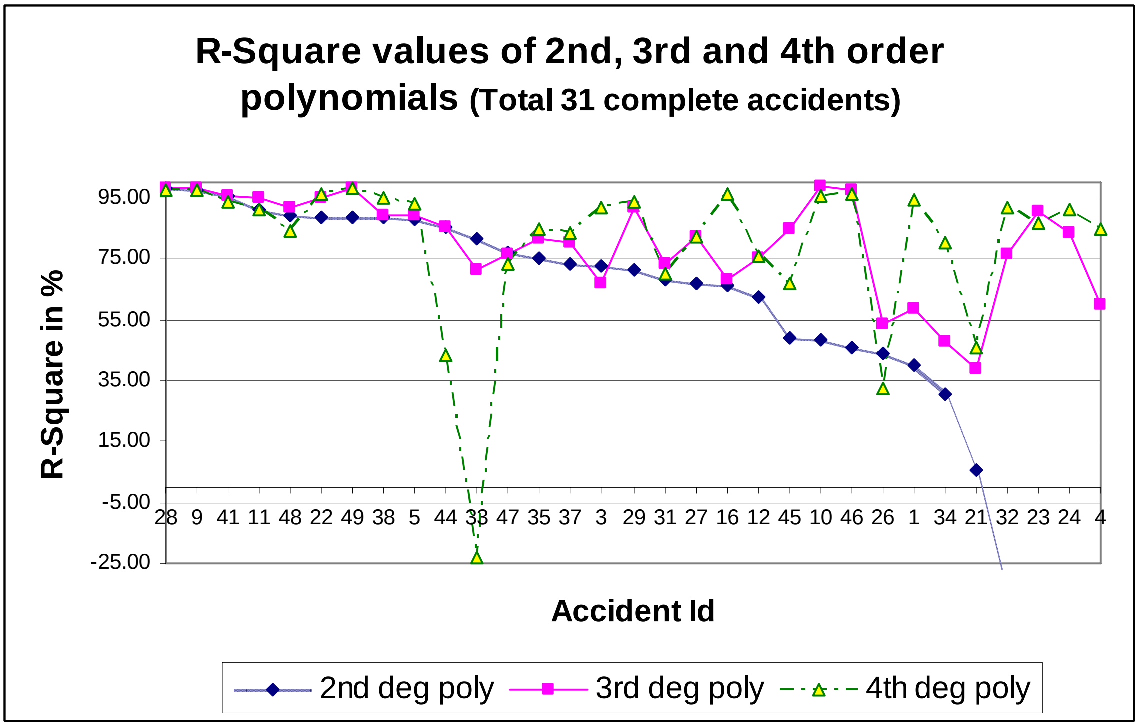

Figure 3. Polynomial model fit

Analyzing Site Differences

In analyzing accident data from different sites, it was helpful to determine whether there were any differences in the accident characteristics. Specifically, two techniques were used to examine accidents from I-70 as compared to I-270. These accidents were examined both spatially and temporally by looking at the maximum queue length and the duration of the accident. First, these accidents were plotted and examined visually to see if I-70 and I-270 accidents patterns were different. Even though no clean-cut boundary could divide accidents from these two freeways, Figure 4 shows that I-270 accidents tend to have longer queue lengths than I-70 accidents. Second, a statistical test was applied to confirm the visual observation.

A student-t test was applied to see if there were any differences in the means between the two freeway sites. Two variables were examined: maximum queue length and accident duration. The test showed that there was a statistically significant difference between the two freeway sites when the variable maximum queue length was considered ($p=0.041$). However, the test also showed that there was no statistically significant difference when the variable accident duration was considered ($p=0.95$). Therefore, the differences in the maximum queue length will need to be considered when the incident progression models are developed for I-70 and I-270.

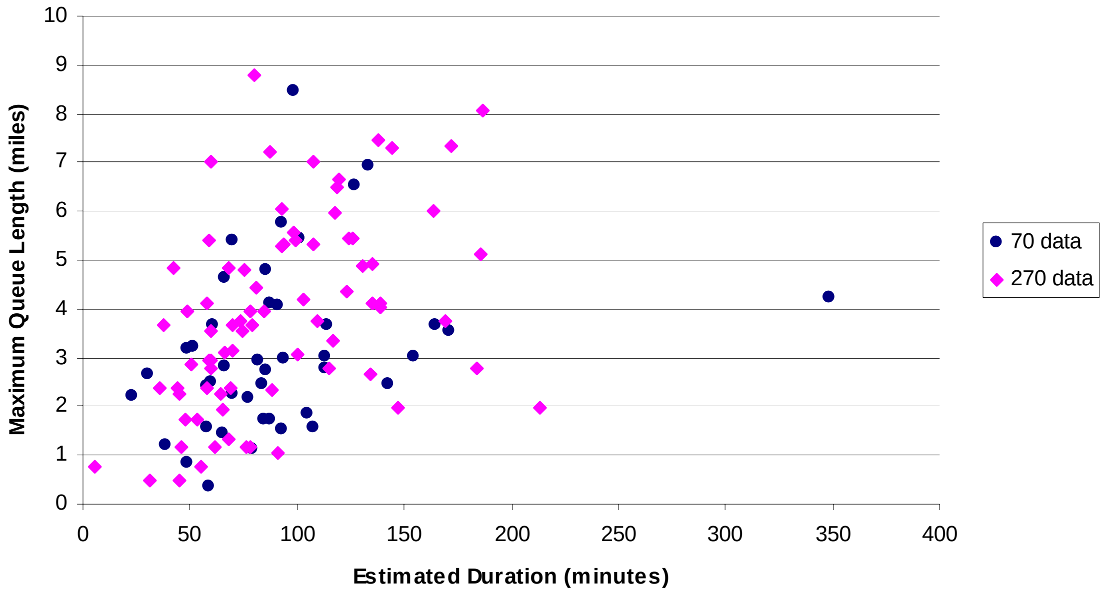

Figure 4. Visual inspection of I-70 vs. I-270 accident data

Secondary Accident Extraction Process

The process to extract secondary accidents from the accident database is as follows. First, the accident database is sorted by route and by year (e.g., I-70, 2002). Each of the 28 fields that describe each accident record is parsed and stored. The time and date fields are translated for computation so that they can be added and subtracted. The entire accident file is then converted into a doubly-linked list so that the file can be collapsed into one record per accident. In other words, a file with one record per vehicle involved in an accident is consolidated into one record per accident. The secondary accidents are extracted by using the queue-time (incident progression) curves, as shown in Figure 5.

The total number of secondary accidents are tallied and recorded for each route and year. A routine can be written in the MATLAB programming language (a C-like programming language for engineering) for this purpose. Figure 5 gives an overview of the secondary accident extraction process.

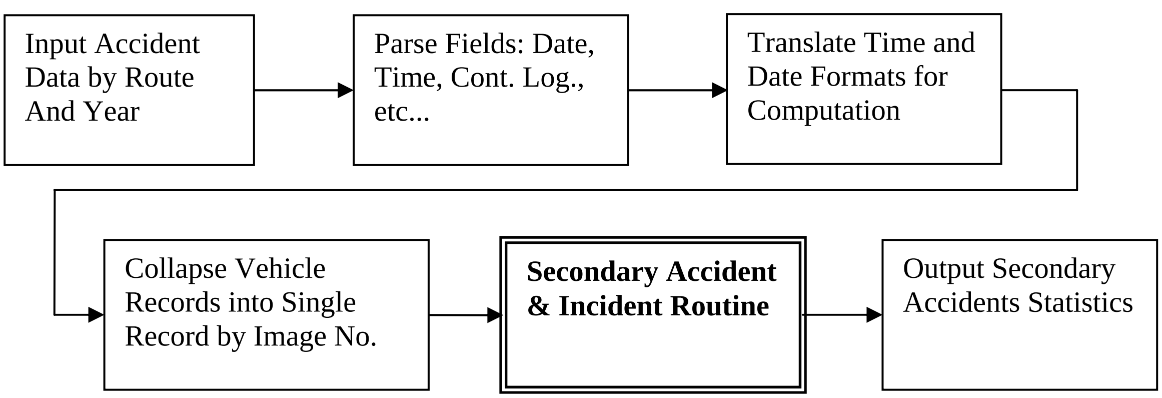

Figure 5. Pseudo-flowchart of the process of extracting secondary accidents

CONCLUSIONS

This paper described the progress of research that would develop a methodology for determining the secondary accidents from a police accident database. The eventual goal is accomplished by data fusion of the police database with intranet traffic reports. An important data processing methodology and two key findings were described. First, the processing of intranet traffic reports was described. Since these reports were human-generated, they presented significant challenges to data processing. A valuable dataset of incident progression data was produced. Second, a third-order polynomial was found to be the best for modeling incident progression curves, as it resulted in the smallest average error and the best model fit. Third, site-specific differences were investigated and the maximum queue length was found to be statistically different between two freeway sites.

The research described seeks to improve asset management decision making by developing a methodology for extracting secondary accidents from police accident databases. The methodology involves the use of data fusion accident data and intranet traffic reports. There is great potential for the immediate technology transfer and the implementation of the results of this research in Missouri and the Midwest. This implementation is in the form of a standard method for extracting secondary accidents from the primary accident database maintained by the police. This standardization would guarantee that the evaluation of all asset management systems will consider secondary accident data in a consistent manner.

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Digital Image Processing for Pavement Distress Analyses

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ABSTRACT

Local agencies have to collect distress data of their network system for building and implementing pavement management programs. Data collection for the whole network is expensive, time consuming, and dangerous, if pursued by traditional field surveys. Developments in computer technology, digital image acquisition, and image processing allow local agencies to use digital image processing for pavement distress analyses. In this project, pavement images obtained from the Long-Term Pavement Performance Program (LTPP) are used to detect horizontal and vertical cracks, crack lengths, and severity. The results are favorable for many images. Further development of the technique may allow adaptation to additional conditions in the images, such as more types of cracks, lane markings, etc.

Key words: digital image processing—pavement distress analyses

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PROBLEM STATEMENT

Local agencies have to collect distress data of their network system for building and implementing pavement management programs. Data collection for the whole network is expensive, time consuming, and dangerous, if pursued by traditional field surveys.

Every year, $17 billion are spent for pavement maintenance in the United States. The limited budget of local agencies with respect to the financial weight of data collection forces them to implement automated distress survey methods. The developments in computer technology, digital image acquisition, and image processing allow local agencies to use digital image processing for pavement distress analyses (McGhee 2004).

In the literature, different techniques are presented for crack detection, identification, and severity determination. These techniques generally use image processing and pattern recognition techniques together to enhance the image and to get the related information from the image.

Chan et al. (1992) used statistical data of the image, such as mean, standard deviation, and variance, to detect cracks and their types. A vertical and horizontal projection histogram of the image is used to get the vertical and horizontal shape factors, and by using other statistical data obtained from the image, a crack detection algorithm was developed. The developed algorithm has a reliability of 70% and 60% for asphalt concrete pavement (ACP) and a seal coat surface, respectively, while 70% accuracy is obtained for continuously reinforced concrete pavement (CRCP). The algorithm has problems detecting multiple cracks in one image and in detecting longitudinal cracks in ACP because of wheel path noise.

Cheng and Miyojim (1998) worked on enhancing the pavement image by removing non-uniform illumination. Thresholding and skeletonization are applied. The number of longitudinal, transverse, and diagonally oriented pixel pairs are obtained and put into a fuzzy classification system to obtain crack type. The authors claim 100% accuracy to be obtained on 42 images.

Wang (2004) applied stereovision to detect other distresses, such as rutting or faulting, that need the definition of 3D morphology. His research is in progress.

RESEARCH OBJECTIVES

In this project, pavement images obtained from the Long-Term Pavement Performance (LTPP) program are used to detect horizontal and vertical cracks, crack length, and severity. The developed algorithm mainly has three parts: pre-process, detection, and post-process algorithms. The results are favorable for many images. Further development of the technique may allow adaptation to additional conditions in the images, such as more types of cracks, lane markings, etc.

RESEARCH METHODOLOGY

Chan et al. (1992) developed algorithms for ACP and CRCP pavement crack detection. Four types of cracks were detected: longitudinal, transverse, block, and alligator. Images were subdivided into 48x48-pixel blocks and vertical and horizontal projection histograms were applied, as presented in Figure 1.

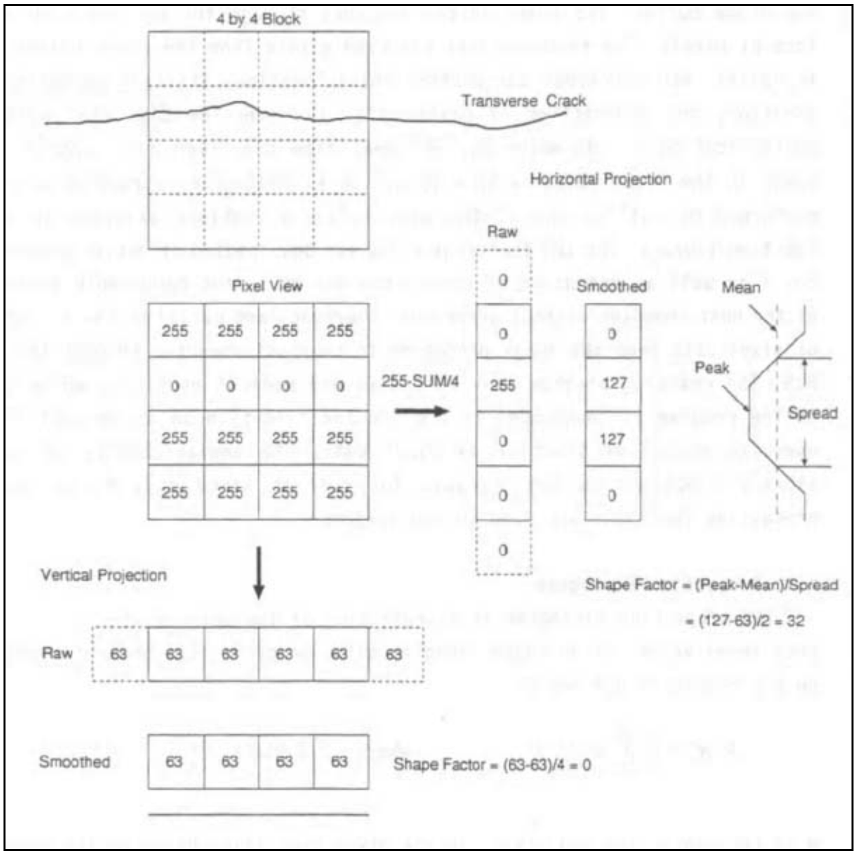

Figure 1. Horizontal and vertical projection histogram

For demonstration purposes, only a 4x4-pixel block was used in Figure 1. The average of the pixels in each row was subtracted from 255 and recorded as a horizontal histogram and than smoothed with a running average of 2 pixels (in the main system, a 7-pixel running average is used: 3 pixels previous, 3 pixels after, and 1 pixel itself). The mean of the sub-block is calculated. Spread is found as the number of pixels greater than the mean, around peak. This is shown graphically on the right in Figure 1. The shape factor is calculated in Equation 1.

$$ShapeFactor = \frac{(peak - meanofblock)}{spread} \tag{1}$$

The same calculations are applied for the vertical histogram. The horizontal shape factor is 32, while the vertical shape factor is zero. The comparison of shape factors leads us to the conclusion that the block has a horizontal crack.

Chan et al. (1992) compared the shape factors with respect to each other and to a threshold value. Thus, if the horizontal shape factor is greater than the vertical shape factor and the threshold, there is horizontal cracking, and vice-versa for vertical shape factors.

A projection histogram technique is used in this work with many modifications to detect crack type. Other techniques are used to enhance the input image and to detect crack length and severity in post-processing. These techniques will be presented in the following sections.

Data from LTPP Images

LTPP is a project of the Federal Highway Administration (FHWA), and consists of a database including 2,400 pavement sections in the United States and Canada. Climatic, structural, and traffic load data for these pavement sections have been collected periodically. Images of pavement sections were collected using 35-mm film and digitized (Elkins 2003).

One pavement section is 150 meters long and is represented by 23 digital images. The digital images are 2048x3072 pixels in dimension. An example is shown in Figure 2.

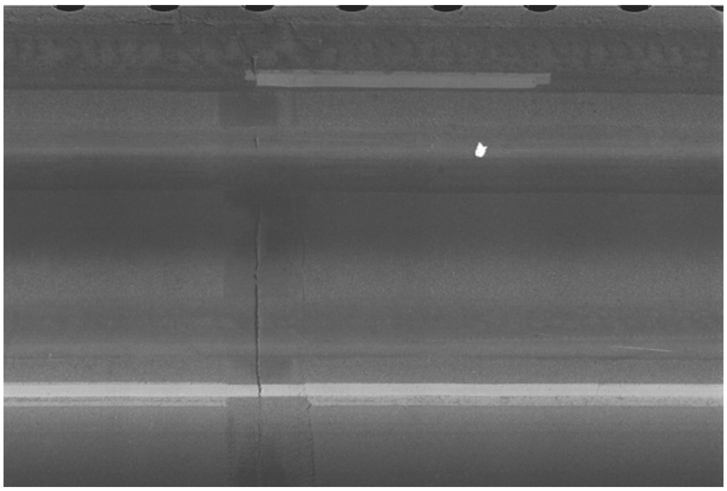

Figure 2. Original image from LTPP database

The images include different types of cracks, noise, lane markings, oil stains, illumination problems, and patches. Each of them is a challenge to work on. The 256x256 pixel images that have only one type of crack were cropped for this project. See Figure 3 for an example.

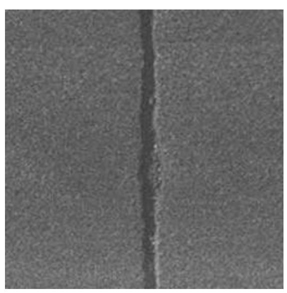

Figure 3. Cropped image (256x256)

Twelve cropped images having high-severity to low-severity cracks were prepared. All images have vertical cracks, and the algorithm may rotate the image before processing to generate horizontal cracks.

Software Design

The algorithms were developed in the MatLab programming environment. The software consists of one m file that reads the image and call functions with other m files.

A block diagram of the software is presented in Figure 4. From **begin.m,** the image is sent to **preprocess.m,** where contrast stretching and thresholding is applied. The image is sent to **morf.m** to remove noise using morphological operations. The returning data is an enhanced image.

The enhanced image is sent to **detect2.m,** where the modified projection histogram technique is applied. Vertical and horizontal shape factors and spreads are calculated and used to characterize a crack.

The output of **detect2.m** is processed in function **postprocess.m** to identify crack type, length, and severity.

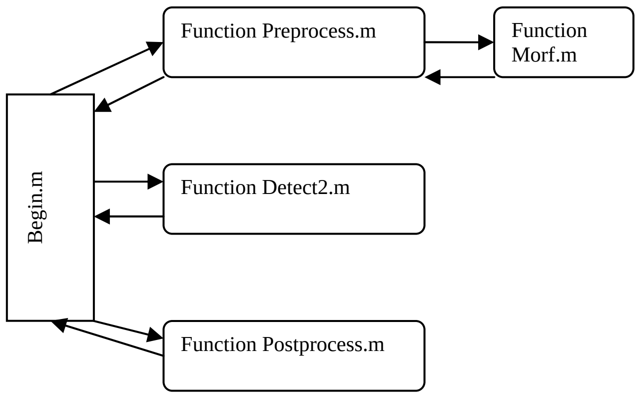

Figure 4. Software block diagram

To be clearer, the functions and their intermediate results will be presented with an example.

Begin.m m file

As shown in Figure 4, **begin.m** provides the structure for the program. **begin.m** reads the images and calls the functions **preprocess.m**, **detect2.m,** and **postprocess.m**. No calculations are done in the script.

All the images used in development are of vertical cracks. To test horizontal cracks, an option can be selected to transpose the image array, as seen in Figure 5.

The image is sent twice to **preprocess.m**, once with the image and once with the image transposed. The result of the first call is an enhanced image that is said to favor horizontal. The second result is transposed upon receipt to preserve proper orientation, and is said to favor vertical. Figure 6 displays this. The reason for these favored directions is explained in section **morf.m**.

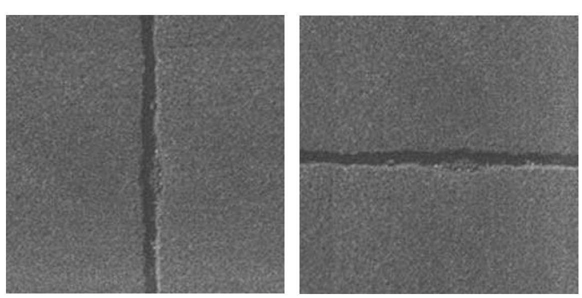

Figure 5. Vertical crack and horizontal (transposed) crack

Figure 6. Enhanced image, favoring horizontal (left), favoring vertical (right)

The next step within **begin.m** is to select the sub-block size in pixels used in the projection histogram technique. This size is sent along with an enhanced image to **detect2.m**. Because there are two enhanced images, favoring horizontal or vertical, there are two calls to **detect2.m**.

The result of this step is two color images that characterize the crack of the original input image. See Figure 7. They are obtained with a modified version of the projection histogram technique. Again, the first resulting image favors horizontal and the second favors vertical.

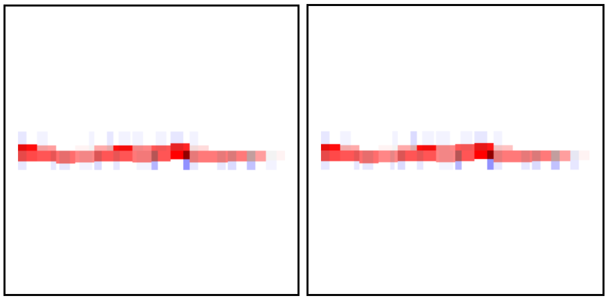

Figure 7. Results of modified projection histogram technique

The last step in the algorithm is **postprocess.m**. Here, crack type, length, and a measure of severity are calculated. An image, seen in Figure 8, showing the calculated size and shape of the crack is also produced. These are the primary products of the project.

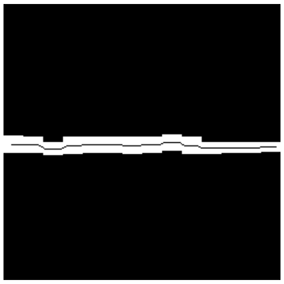

Figure 8. Identified crack with thinned line for finding crack length

Function Preprocess.m

The input to this function is an image and the output is an enhanced binary image ready for **detect2.m**.

Contrast stretching is applied to the image by using the p-code **intrans.p** of the DIPUM book. The **intrans.p** uses Equation 2 for contrast stretching (Gonzalez et al 2004).

$$s = T(r) = \frac{1}{1+\left(m/r\right)^{E}} \tag{2}$$

The gray scale of the image is scaled to [0 1] and the m is determined as 0.1 minus the mean of the image. The slope of the curve is determined by E, which is taken as 50. The values of these parameters have been determined by trial and error.

A thresholding value of 230 is used to produce a binary image.

The image is outsourced to function **morf.m,** described below, where the noise is reduced. The result obtained after running this function is presented in Figure 9. The output of **morf.m** is passed unchanged as the output of **preprocess.m**.

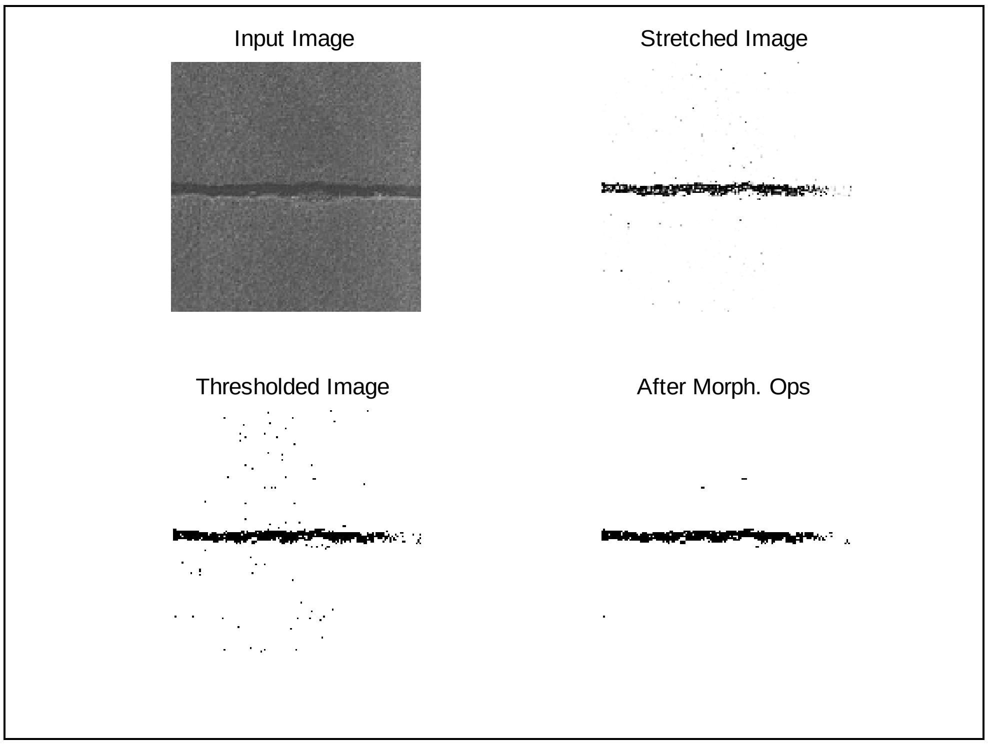

Figure 9. Result of the preprocess.m function

Function Morf.m

The morphological operations consist of a series of closing and opening operations with a horizontal line element of varying length. The input to this function is a binary image and the output is a binary image with less noise.

The initial task is to compliment the image. The first operation is closing using a medium-sized structural element and opening with a slightly smaller element to restore the remaining objects. The goal is to remove the small, single-pixel noise.

Another closing-opening operation is performed with larger structural elements. This step removes most of the remaining noise.

However, the pixels that hopefully represent the crack are joined in the process. To retain only the original pixel data, the morphological result is multiplied with the binary input to the function. The pixels are again complimented to return to a black and white image. A sample of results of **morf.m** is shown in Figure 10.

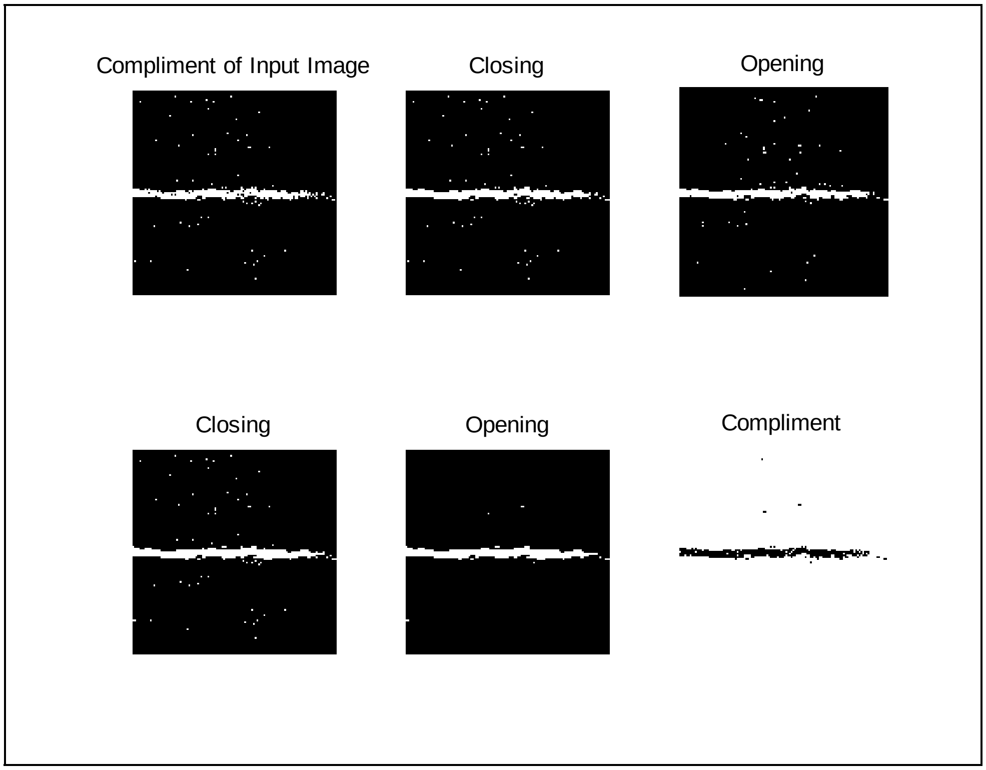

Figure 10. Results of morphological operations

The contrast stretching of **preprocess.m** and morphological operations performed in **morf.m** give clear images that have less background noise.

The result of **morf.m** is said to favor horizontal. This is because much of the information characterizing a vertical crack will be removed. A horizontal crack will have nearly all of its pixels retained. To get favorable results with a vertical crack, the image must be rotated 90° or transposed.

Function detect2.m

The **detect2.m** function is the backbone of the software as it evaluates the projection histograms and gives an output of shape factors and spreads.

The input to this function is a preprocessed image and the size of sub-blocks.

Sequentially, the algorithm works in the following way:

1. The algorithm determines the maximum number of sub-blocks and truncates the image to ensure an integer number of sub-blocks are used.
2. The image is divided into sub-blocks.
3. Within each block, the algorithm calculates the horizontal and vertical spread and shape factor.

4. The algorithm creates a color image for visualization of each sub-block: red for the horizontal spread and shape factor, and blue for the vertical spread and shape factor. The stripes represent the spreads and the stripe color intensity reflects the shape factor. Figure 11 shows an example.
5. The algorithm combines color visualizations into a full-sized composite image. See Figure 12.

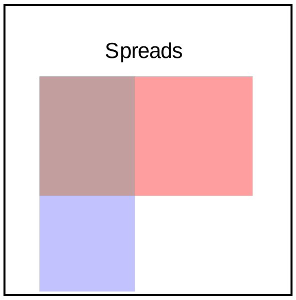

Figure 11. Visualization of spread and shape factor for a single sub block

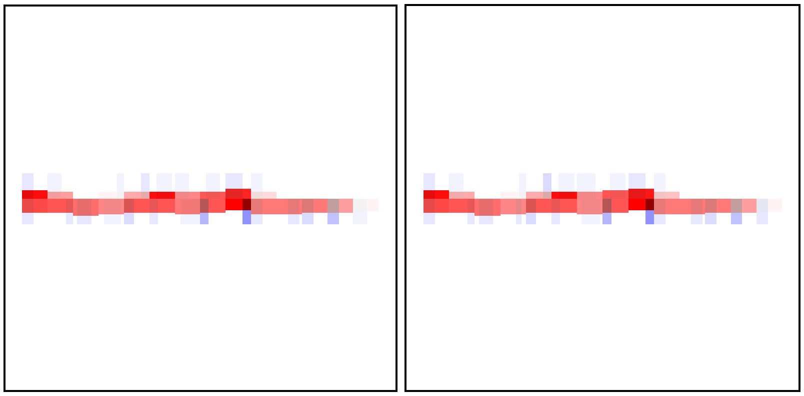

Figure 12. Results of detect2.m: favoring horizontal and favoring vertical

There are two preprocessed images. Therefore, **detect2.m** is called twice. Two color images are obtained. The first favors horizontal and the second favors vertical.

Function Postprocess.m

The decisions are made in **postprocess.m**. However, the first steps are additional processing of the color images.

The values of the red channel of each pixel are summed for the image that favors horizontal. Likewise, the values of the blue channel are summed. The crack type is determined by these sums. If the red sum is higher, the crack is horizontal. If the sum of the blue channel is higher, a vertical crack is present. This is the first and simplest product of the project.

The **postprocess.m** function next takes the winning channel, red if horizontal or blue if vertical, and thresholds. The resulting binary image is input to the MatLab function **bwlabel()**. This function identifies connected groups of pixels and labels them sequentially from 1 to n. The background is set to zero.

These groups are further processed one by one. Each group is thinned using **bwmorph()**, which removes pixels around the edges of the group until a line width of one pixel remains. The number of pixels in this line is also the length of the line. This value is used as the length of the crack. This is another product of the algorithm.

The area of each group of connected pixels is found with MatLab's **find()** and **numel()** functions. This area is divided by the crack length to find the average thickness of each group. This can be interpreted as a measure of the severity of a crack. This is the final numerical product, as seen below.

```
VertCrop\190101i20021017_11V10.JPG
Crack Type: Horizontal
 IDnumber Length Area Severity
 1 241 3438 14.266
```

A visual output, which represents the shape and area of the image covered by the crack, is produced by displaying all groups of connected pixels. For visualization purposes, the thinned line used to find the crack length is included as a black line within each group. See Figure 13.

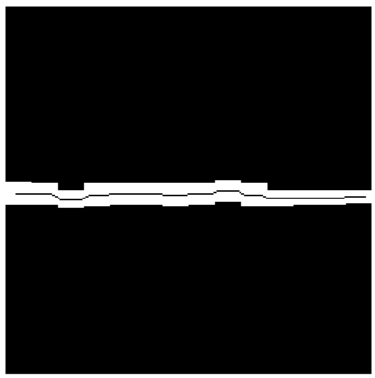

Figure 13. Visualization of crack shape and area and thinned line used for length calculation

GUI: Crevas

The algorithm developed above was implemented with a graphical user interface with the aid of MatLab's guide. The user interface was kept simple, requiring only the selection of an image from a list and an option to transpose the image to analyze a horizontal crack. This GUI is displayed in the following section. See the images there for a visualization.

KEY FINDINGS

For many images, the results of the algorithm are encouraging. However, some images caused poor performance. The reason for this, in many cases, is elusive.

In Figure 14, the program takes an image with a thick crack and returns a strong response. The shape of the detected crack matches the crack in the input image very well. The numerical output is as follows.

```
VertCrop\190101i20021017_11V10.JPG
Crack Type: Horizontal
 IDnumber Length Area Severity
 1 241 3438 14.266
```

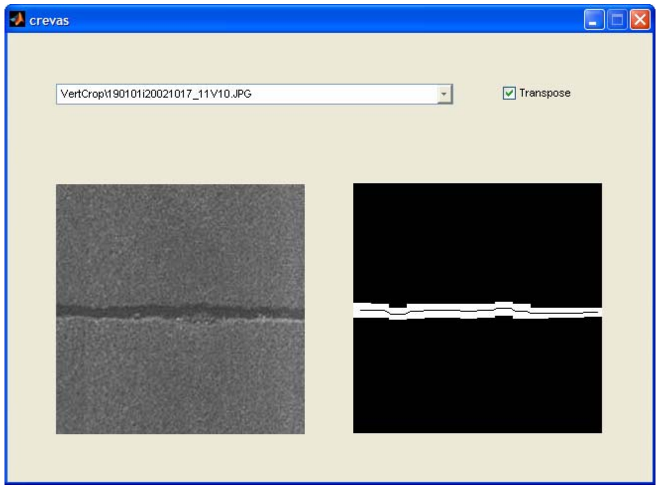

Figure 14. Strong, thick, single crack

The algorithm is able to find more than one crack, giving numerical statistics for each. The numbers calculated for Figure 15 are favorable results. In particular, note the length of the cracks.

```
VertCrop\190101i20021017_03V1.JPG
Crack Type: Vertical
 IDnumber Length Area Severity
 1 135 1602 11.867
 2 65 576 8.8615
```

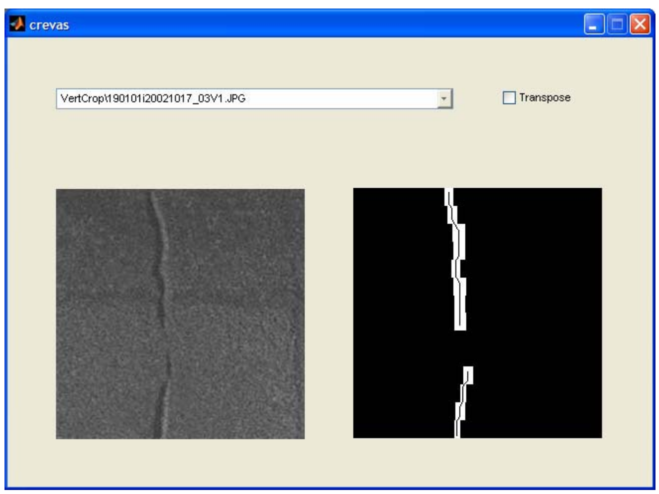

Figure 15. Strong, intermittent crack

While the roadway marking surprisingly causes little noise, the algorithm is very sensitive to noise that the human eye does not perceive as significant. See Figure 16. Below is the numerical output for this image. Note that the largest value in the length column corresponds to the actual crack of the input image (IDnumber 5).

```
VertCrop\190103i20021017_02V3.JPG
Crack Type: Vertical
 IDnumber Length Area Severity
 1 141 1062 7.5319
 3 190 1998 10.516
 5 264 3186 12.068
 8 117 1080 9.2308
 9 154 1566 10.169
 10 27 360 13.333
 11 27 342 12.667
 13 118 936 7.9322
 14 100 1134 11.34
 15 39 972 24.923
 18 261 2448 9.3793
 21 30 234 7.8
 23 48 468 9.75
 24 28 324 11.571
```

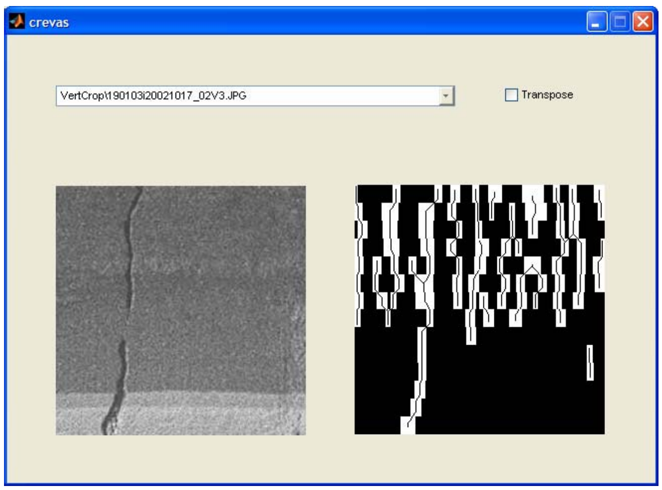

Figure 16. Strong crack, but noisy results

SUMMARY AND CONCLUSION

In this paper, we have taken the projection histogram technique developed by Chan et al. and made several modifications that improve its performance.

Pavement images from the state of Iowa, supplied from LTPP/FHWA, were used in this study. Before using the technique, the images were preprocessed by binerization and morphological operations. Modifications to the technique allow more accurate characterization of the crack. Post processing allows detection of multiple cracks and the calculation of length and average width, which can be a measure of a crack’s severity.

The current algorithm performs very well for many images. In almost all cases, the crack in the input is detected in the output. There are cases, however, for which noise causes false detections. These false cracks can be large in number and cause unacceptable clutter in the numerical results and the output image.

Further work focuses on two areas: improving the current algorithm and changing detection techniques to be able to detect distresses from whole images with high accuracy.

ACKNOWLEDGMENTS

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Lateral Load Tests on Small-Diameter Piles for Slope Remediation

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ABSTRACT

Slope reinforcement and the use of structural pile elements can be an effective slope remediation alternative when conventional remediation practices (e.g., improved drainage) fail to consider the causal factors leading to slope instability (e.g., strength loss due to weathering). An experimental research program was aimed at developing a rapid, cost-effective, and simple remediation system that can be implemented into slope stabilization practices for relatively shallow (< 5 m) slope failure conditions. The non-proprietary remediation technology consists of small-diameter, grouted micropiles. The research program described in this paper establishes the micropiles as a feasible remediation alternative. Details of the experimental testing and the results from selected measurements are presented in the paper. Lateral load tests on drilled and grouted pile elements of two diameters, in which the piles were installed through a shear box and loaded by uniform lateral translation of soil, advanced our understanding of the soil load transfer to the piles. The pile load test plan included three soil types, and the piles were installed into glacial soils of the experimentation site. Instrumentation of the shear boxes and pile reinforcement indicated the load distributions that developed along the piles and the pile response to the physically-imposed boundary conditions. Results show that piles installed in failing slopes will arrest or slow the rate of slope movement. Furthermore, the soil movement associated with slope failures induces lateral load distributions along stabilizing piles that vary with soil stiffness and strength, pile stiffness and section capacities, and the spacing of piles over the slope.

Key words: laterally loaded piles—slope reinforcement—slope stability

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INTRODUCTION

In situ reinforcement methods for stabilizing cut slopes and embankments have included stone columns, soil nailing, and structural pile elements (e.g., drilled piers, micropiles, recycled plastic pins). The use of structural pile elements can be particularly effective when conventional remediation practices fail to address the causal factors leading to slope instability. Pile elements offer passive resistance to downslope soil movement by transferring the loads developed along the pile to stable soil below the failure surface. The soil load transfer to pile elements is a complex soil-structure interaction problem, and the differences in existing design procedures for pile stabilization suggest that the stabilizing mechanisms are not fully understood.

An experimental research program was aimed at developing a rapid, cost-effective, and simple remediation system that can be implemented into slope stabilization practices for relatively shallow (< 5 m) slope failure conditions. The non-proprietary remediation technology consists of small-diameter, grouted micropiles. Recent investigations (e.g., Loehr and Bowders 2003, White and Thompson 2005) have evaluated the use of slender pile elements for stabilizing slopes. Slope remediation with small-diameter piles may more effectively address the cost, environmental impact, schedule, and construction constraints associated with larger drilled piers or driven pile sections. The research program described in this paper establishes the micropiles as a feasible remediation alternative, especially for slope remediations where overexcavation and improved drainage are not appropriate.

RESEARCH METHODS

To develop design concepts, soil-structure interactions for grouted micropiles subject to lateral soil movement were investigated by conducting full-scale lateral load tests. The load tests were performed in a manner similar to large-scale direct shear tests. The direct shear boxes contained compacted soil with known properties and piles that extended through the shear box into glacial soils of the experimentation site. Piles were installed approximately 1.5 m into the existing ground (2.1 m pile lengths) by bottom feeding the concrete mixture into individual boreholes prepared by solid-stem and hollow-stem augers. The shear boxes were pushed laterally to impose uniform lateral translation of soil, modeling the movement of a unit cell of a sliding soil mass. Figure 1 illustrates the load test setup. Instrumentation of the direct shear boxes, including a 222-kN load cell and three string potentiometer displacement gauges, indicated the load-displacement behavior of reinforced soil. Instrumentation of the pile reinforcement consisted of 10 strain gauges to indicate the loads induced on the piles due to lateral soil movement and the pile response to the loads.

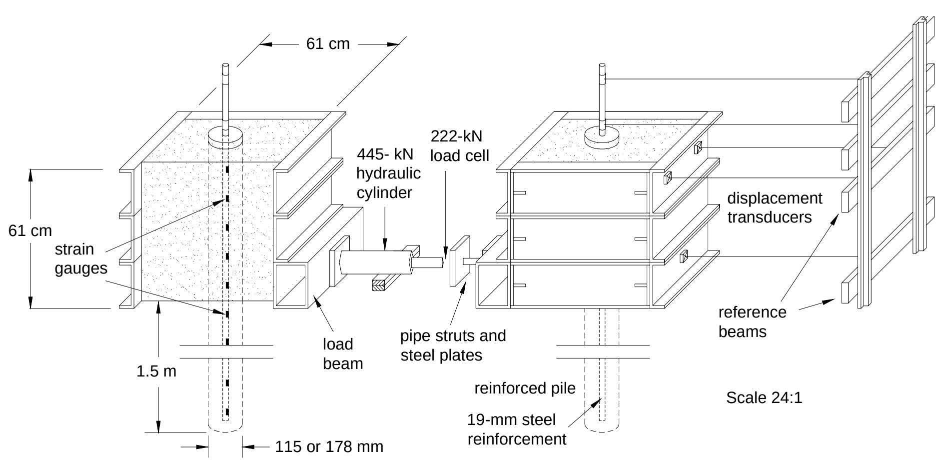


Figure 1. Lateral load test setup

The lateral load test plan evaluated soil type, pile size, and the effect of pile grouping as each parameter relates to the performance of the slope reinforcement system. Each reinforcement parameter influences the response of piles subject to lateral soil movement. The influence of the parameters on pile behavior is evidenced by the dependence of soil load-displacement (p-y) curves on the parameters. The lateral load test plan, provided in Table 1, included 14 different pile configurations. The full-scale load tests were performed to evaluate the performance of 115-mm and 178-mm piles, each reinforced with a centered 19-mm steel rebar.

Table 1. Lateral load test plan

Test Number	Box Number	Box Soil	Pile Size [a]
1	1	Loess	No pile
	2	Weathered shale	No pile
2	3	Glacial till	No pile
	4	Loess	114-mm
3	5	Glacial till	112-mm
	6	Weathered shale	117-mm
4	7	Weathered shale	114-mm [b]
	8	Loess	183-mm
5	9	Glacial till	178-mm
	10	Weathered shale	(2) 113-mm [c]
6	11	Loess	(2) 114-mm [c]
	12	Weathered shale	173-mm
7	13	Glacial till	(2) 113-mm [d]
	14	Glacial till	(2) 115-mm [c]

[a] Measured after pile exhumation
[b] No pile reinforcement
[c] Piles in a row
[d] Staggered piles

Opposing shear boxes were loaded against each other, such that each test involved the simultaneous loading of two boxes. The lateral load tests were performed by monitoring shear box displacement and controlling the load applied to each shear box. Load increments of approximately one kN were applied to the system, and the displacements of each shear box were monitored at the relatively constant load. The next load increment was applied when the rate of displacement for each shear box became small. The loading procedure resulted in test durations ranging from 90 to 180 minutes. The details of pile installation, load test setup, and data acquisition are detailed in White and Thompson (2005) and White et al. (2005).

EXPERIMENTAL TESTING MATERIALS

Soil Properties

The experimental testing was performed at the Iowa State University Spangler Geotechnical Experimentation Site in Ames, Iowa. The site soil profile consists of non-stratified glacial till approximately 1.5 m thick, underlain by sand. The glacial till soil classifies as CL Lean clay with sand; properties of this soil are provided in Table 2.

Table 2. Sampled soil properties

Soil	Classification	Soil property			
		γ_d (kN/m^3)	w (%)	c_u (kPa)*	ε_{50} (%)*
Glacial till (site)	CL Lean clay with sand	13.7	30	35	---
Loess	ML Loess	14.3	30	17	4.5
Glacial till	CL Sandy lean clay	18.9	16	53	2.0
Weathered shale	CL Lean clay	17.4	21	28	1.2

*Undrained shear strength (c_u) and stain at 50 percent of the peak strength (ε_{50}) values based on unconfined compression test results.

As the load test plan evaluated the influence of soil type on pile behavior, three soils (loess, glacial till, weathered shale) from different regions of Iowa were used to fill the shear boxes. The compacted soil from select shear boxes was sampled following the performance of lateral load tests. Soil sampling with thin-walled Shelby tubes and in situ testing devices, namely the dynamic cone penetrometer and K_o stepped blade, helped characterize the soil conditions. Soil properties of the loess, glacial till, and weathered shale are provided in Table 2.

Pile Characteristics

The selection of a concrete mixture design evolved from mixture designs of self-consolidating concrete (SCC) and controlled low-strength material (CLSM). Preliminary mixture proportions and testing results are provided in Table 3. The concrete mixture for the small-diameter piles approximately exhibited the flow properties of CLSM and the mechanical performance properties of SCC. At the test site, slump ranged from 20 to 24 cm, and compressive strengths ranged from 27 to 32 MPa. Compressive strengths were determined by performing compression tests on 76-mm test cylinders from each batch (one batch per pile). The concrete maturity curve from samples prepared in the laboratory and pile concrete strengths from field samples are provided in Figure 2.

Table 3. Preliminary concrete mixture proportions and testing results

Category	Criteria	SCC [a]	CLSM [b]	Selected mixture
Constituent	Cement (lb/cy)	600	100	600
	Fly ash (lb/cy)	n/a	400	125
	Fine aggregate (lb/cy)	1340	2600	2700
	Coarse aggregate (lb/cy)	1700	n/a	n/a
	w/cm	0.55	1.12	0.65
Admixtures	HRWR [c] (fl oz/cwt)	8	n/a	8
	VMA [d] (fl oz/cwt)	2	n/a	2
Performance	21-day strength (MPa)	54.35	2.68	30.34
	Slump (cm)	17.8	27.9	27.4

[a] Schlagbaum (2002)
[b] Center for Transportation Research and Education (2003)
[c] High range water reducer
[d] Viscosity modifying admixture

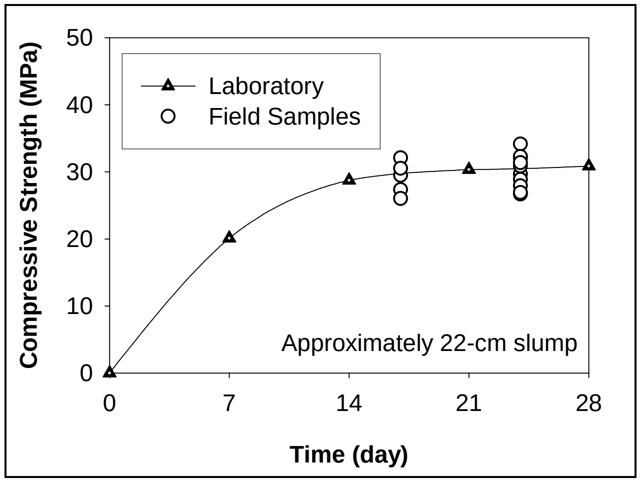


Figure 2. Grout maturity curve and pile compressive strengths

The tensile strength of reinforcing steel was determined by performing a tension test with a 19-mm steel rebar. The stress-strain data provided a yield strength of 455 MPa at 2.8×10^3 microstrains and a maximum stress of 759 MPa at 8.8×10^4 microstrains.

LOAD TEST RESULTS

Load Displacement

The measured load-displacement relationships of the shear boxes are provided in Figure 3. The load-displacement relationships for reinforced soil indicate the contribution of the pile to the shear strength of the system, assuming that the difference in load between reinforced shear boxes and unreinforced shear boxes, for a given soil type and lateral displacement, is that load carried by the pile elements. The load-pile head deflection relationships, for brevity not presented here, generally follow those for shear boxes. Pile head deflections were measured to aid the lateral load test analyses.

The 115-mm piles offered considerable resistance to lateral soil movement. The installation of small-diameter piles generally resulted in peak loads ranging from 215% to 325% of the loads for unreinforced soil. The use of 178-mm piles offered additional resistance, with peak loads ranging from 325% to 390% of the loads for unreinforced soil. The installation of two piles offered less resistance than twice that of isolated piles. The peak loads of tests with grouped piles ranged only from 120% to 205% of the loads for tests with isolated piles.

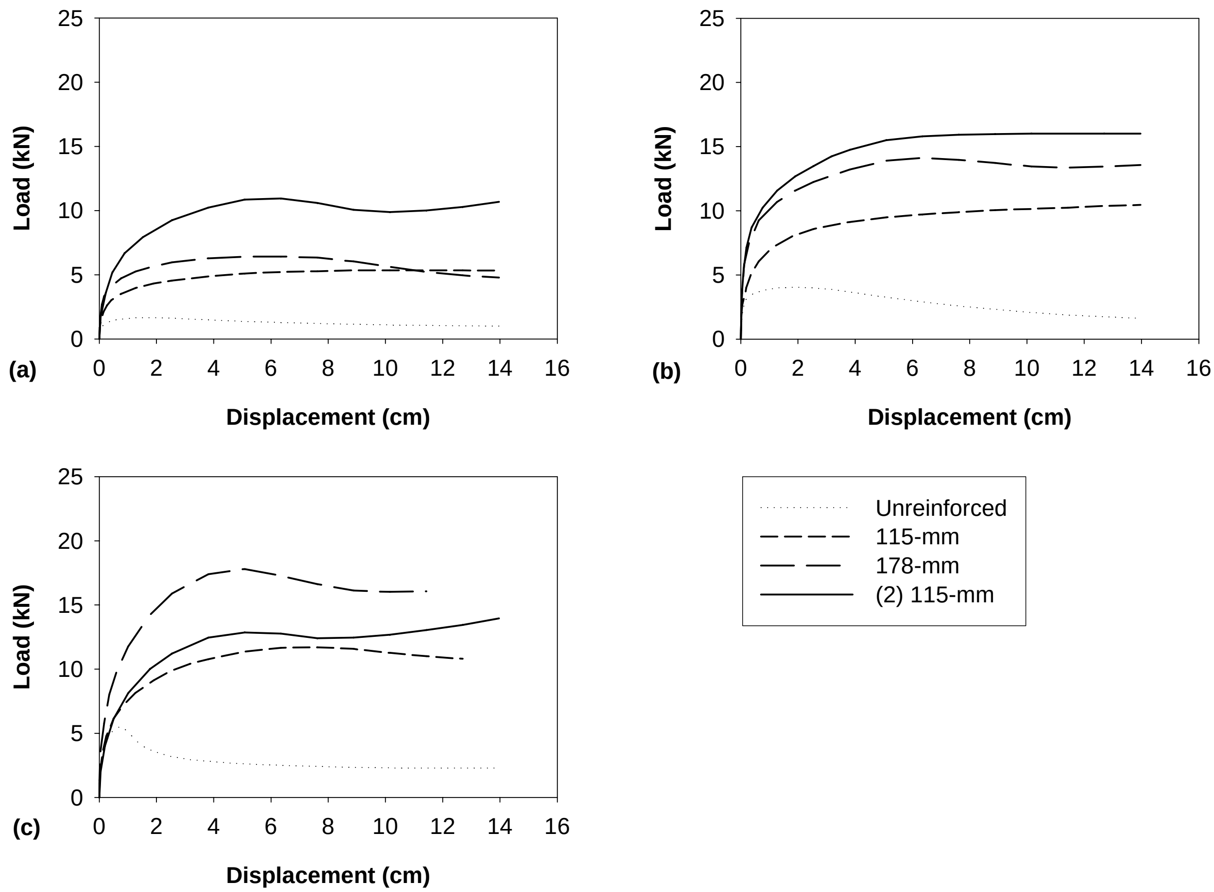


Figure 3. Load-displacement relationships

(a) Loess; (b) glacial till; (c) weathered shale

Behavioral Stages

Graphs of the relative displacement of the shear boxes and pile heads are used to support the observed pile behaviors during the performance of the load tests. During loading, a gap formed in front (i.e., load side) of the pile at the soil surface. Figure 4 relates gap width and load for each test. This figure, in addition to displaying the test data, indicates the behavioral stages of piles subject to lateral soil movement, as follows: Stage 1, mobilization of soil shear stresses and elastic bending of pile; Stage 2, mobilization of pile flexural stiffness; and Stage 3, incipient failure due to mobilization of pile moment capacity. Stage 1 is characterized by relatively linear behavior of the soil and the intact pile element. The stress development at the soil-pile interface is insufficient to cause yielding of the soil or cracking of the pile, such that a gap of negligible width forms. Stage 2 commences with the development of a bending moment in the pile element that causes the tension-carrying concrete to crack. The pile stiffness immediately drops, and the pile element becomes more flexible. Further loading of the pile causes more rapid pile rotation and pile head deflection. Coincidentally, the gap formation occurs more rapidly. Stage 3 commences with the mobilization of the pile moment capacity. Gap formation that occurs during Stage 3 occurs under constant load. The failed pile element is incapable of carrying additional load. Gaps of significant width (approximately 10 mm) form with the mobilization of pile moment capacity.

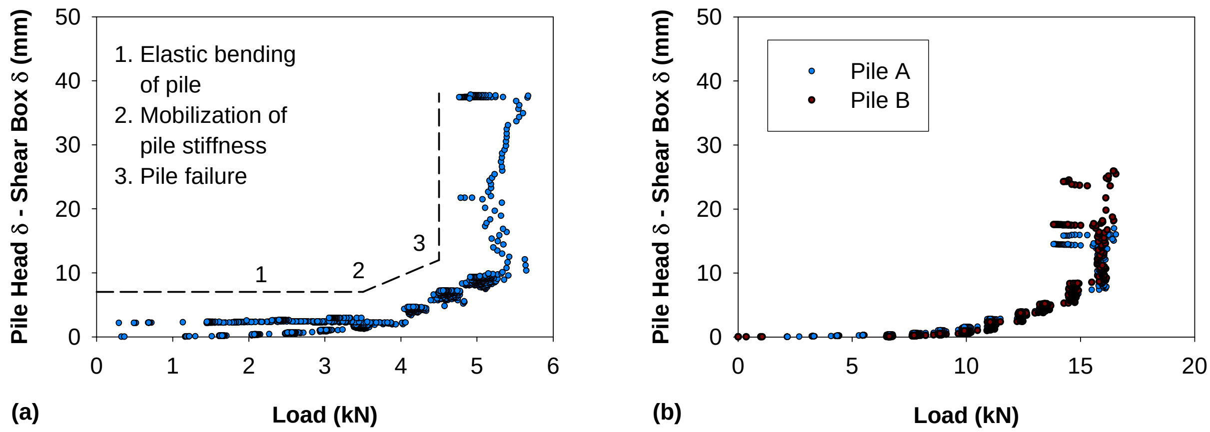


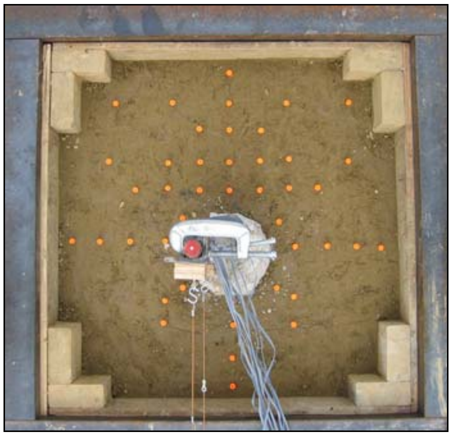
Figure 4. Behavioral stages of piles subject to lateral soil movement

(a) Pile 4; (b) Piles 13A and 13B

Cameras were mounted above the piles of each load test to document the observed behavior of pile heads and surface soil at different stages during pile loading. The pictures, some of which are provided in Figure 5, show the data used to indicate the behavioral stages of piles subject to lateral soil movement. Figures 5 (a) through (c) show the gap formation corresponding to Figure 4 (a), while Figures 5 (d) through (f) show the gap formation corresponding to Figure 4 (b). The pictures also indicate soil stress buildup around the piles. Cracking observed in the surface soil suggests decreasing tangential stresses around isolated piles and soil arching between grouped piles.

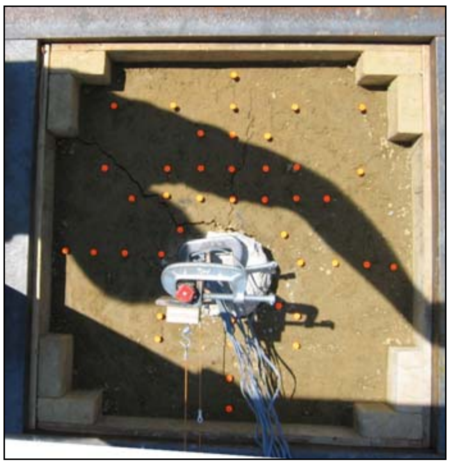
Gap formation occurs because the deflection of the pile head, due to rotation of the pile, exceeds the displacement of the surface soil. Pile head deflection exceeded soil movement, even at early stages of the test (see Figures 4 and 5). Poulos (1995) recognized that pile head movement that exceeds soil movement is associated with the intermediate mode, defined by mobilization of soil strength along the pile in both the moving and stable soil. The development of a gap was fundamentally important, because the load distributions along the piles were directly affected by the exposed—and, therefore, unloaded—length of the pile. The length of the pile in which lateral deflection exceeded shear box displacement was more accurately subject to passive soil pressures in the direction opposite that of soil movement.

The K_o stepped blade test (Figure 6 (a)) was incorporated into the soil sampling plan to support the observed soil behavior, specifically the formation of a gap at the soil surface in front of the pile. Associated with the gaps (Figure 6 (b)) are unloaded lengths of pile in the direction of shear box movement and lower lateral soil pressures in front of the pile. Conversely, bulging of the soil was observed behind the pile (Figure 6 (c)). Associated with the bulging soil are lengths of pile loaded in the direction opposing shear box movement and higher lateral soil pressure behind the pile. At greater depths, the lateral soil pressure in front of the pile exceeds the lateral soil pressure behind the pile, confirming that the net load applied to the piles is, in fact, in the direction of soil movement. The interpretation of load development along piles subject to lateral soil movement is support by the K_o stepped blade test results, provided in Figure 7.

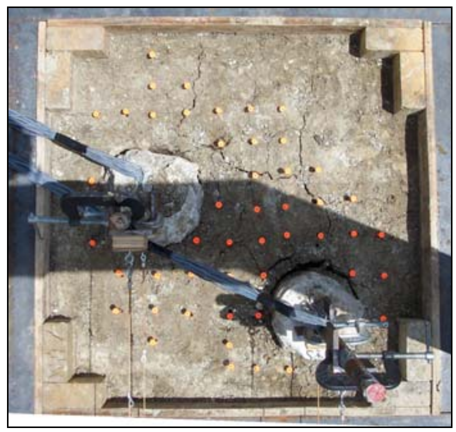


(a)

(d)



(b)



(e)

(c)

(f)

Figure 5. Pile 4 (a, b, c) and Pile 13 (d, e, f) photogrammetry pictures (a) 0 cm; (b) 5 cm; (c) 16 cm; (d) 0 cm; (e) 10 cm; (f) 14 cm

(a)

(b)

(c)

Figure 6. Load development along piles subject to lateral soil movement

(a) K_o stepped blade test; (b) gap in front of pile; (c) bulging behind pile

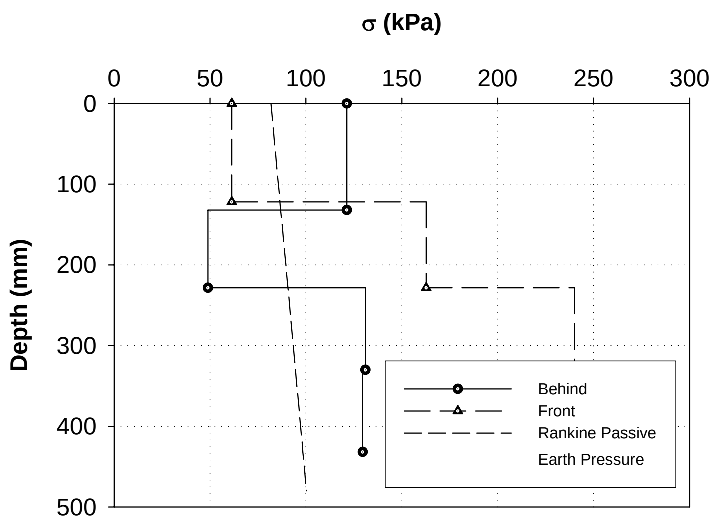


Figure 7. K_o stepped blade test results, load development along piles

Bending Moment

As the evaluation of pile response to loading is generally completed by examining the deflection, shear, and bending moment of a pile, the analysis of test data required an understanding of the relationship between steel strain and bending moment. Data interpretation (i.e., conversion of strain to bending moment) was achieved by performing moment-curvature analyses for the pile sections, with the analysis input parameters being cross-sectional configuration and material properties. For the full range of loading, from an unloaded condition to section failure, the moment-curvature relationship examines member ductility, development of plastic hinges, and redistribution of elastic moments that occur in reinforced concrete sections (Nilson 1997). The analysis additionally provides the strain distribution through pile sections, such that the measured strain of the pile reinforcement is directly related to bending moment. The relationship between pile flexural stiffness (EI) and bending moment is shown in Figure 8 to indicate the stages of pile behavior for the range of possible loading conditions. The relationship between gauge strain and bending moment is included to demonstrate the conversion of measured strain values to bending moments.

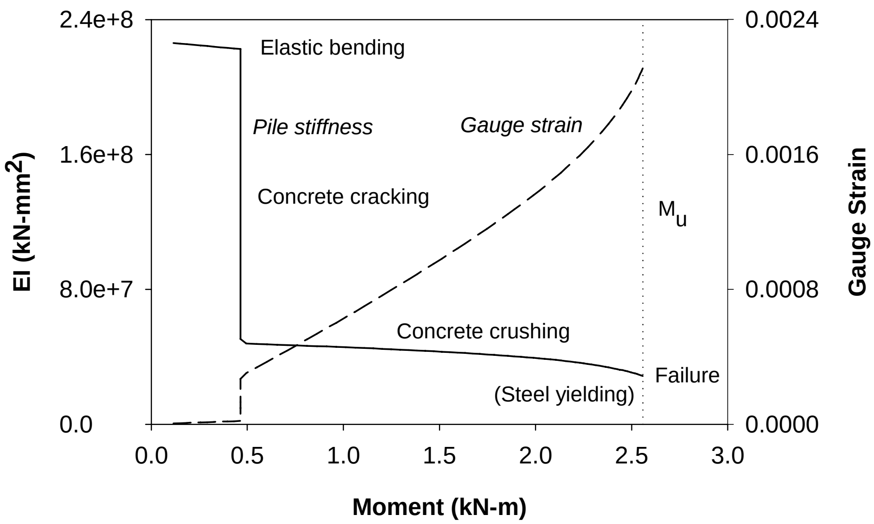


Figure 8. Flexural stiffness gauge strain-bending moment relationship

The strain of steel reinforcement was measured along the entire length of the piles during the lateral load tests. Sample strain profiles for two piles in loess at various loads are provided in Figure 9. All strain profiles provide convincing evidence that piles failed due to mobilization of bending moment capacity and support our interpretation of the behavioral stages of piles subject to lateral soil movement. The maximum measured strain for most isolated piles approximately equaled the strain corresponding to the moment capacity of the pile section. Approximately one-third of the grouped piles mobilized the full bending moment capacity. For tests with grouped piles, the loading system was particularly unstable. The moment capacity was not achieved because these tests were terminated prematurely.

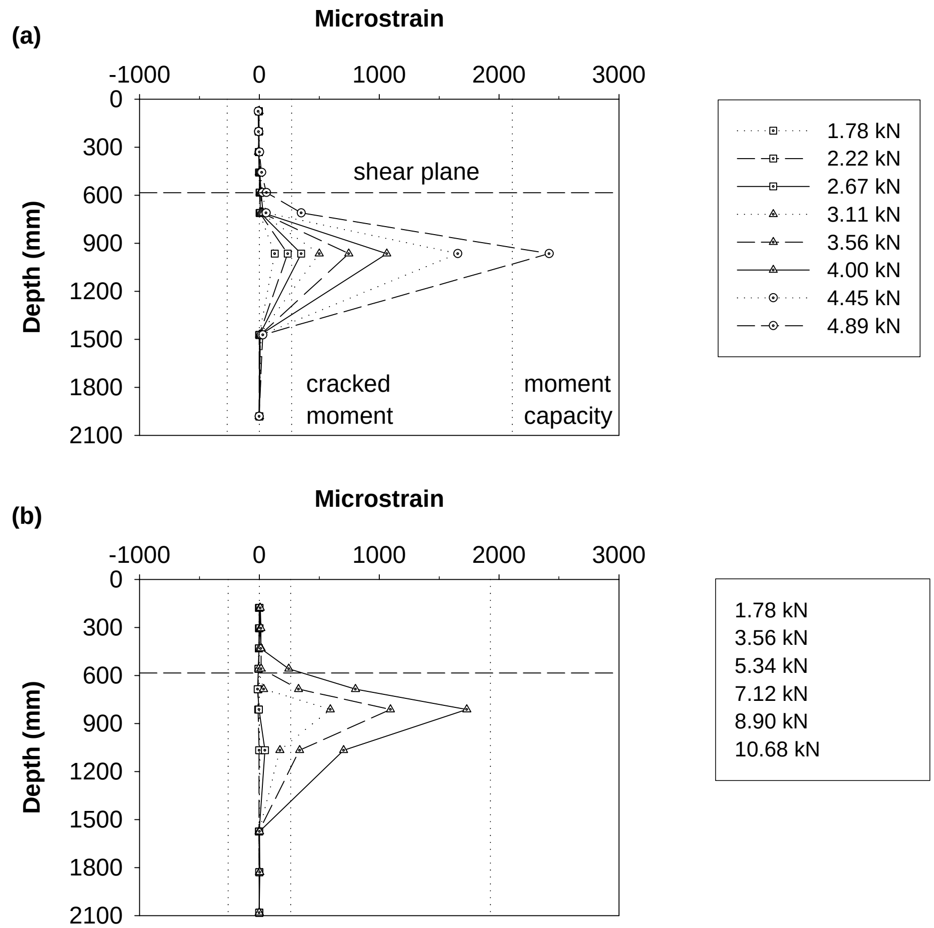


Figure 9. Strain profiles for piles in loess

(a) Pile 4; (b) Pile 11A

SUMMARY

Lateral load tests performed on small-diameter, drilled, and grouted piles indicated that slender pile elements may offer considerable resistance to the soil movement as it occurs during slope failure. The experimental testing is summarized as follows:

1. The pile concrete mixture was developed to exhibit the flow properties of CLSM and mechanical properties of SCC, as slump ranged from 20 to 24 cm and compressive strengths ranged from 27 to 32 MPa.
2. The smaller diameter (115-mm) piles offered considerable resistance to lateral soil movement, with peak loads ranging from 2.15 to 3.25 times those for unreinforced soil. The larger diameter (178-mm) piles offered additional resistance, with peak loads ranging from 3.25 to 3.90 times those of unreinforced soil.
3. Pictures taken from above the shear boxes at different stages of loading indicate stress buildup around isolated piles and the soil arching mechanism that occurs between grouped piles. Nevertheless, the peak loads from load-displacement relationships for tests performed with grouped piles ranged only from 120% to 205% of the loads for tests with isolated piles.

4. The relative displacement of shear boxes and pile heads indicate that behavioral stages of piles are subject to lateral soil movement, recognizing that pile head movement that exceeds soil movement is associated with the intermediate mode.
5. Strain measurements of pile reinforcement provide evidence of pile failure due to mobilization of bending moment capacity. The maximum measured strain for many piles approximately equaled the strain corresponding to the moment capacity of the pile section, based on moment-curvature analyses.

RECOMMENDATIONS FOR IMPLEMENTATION

The research study established small-diameter pile elements as a feasible slope stabilization alternative. Immediate recommendations for future research include the construction and monitoring of pilot projects. Implementation of slope reinforcement through pilot studies, which will help us more fully understand and verify the load transfer mechanisms of the stabilization system, is the next most important task for improving the slope remediation alternative. The outcomes of instrumented pilot projects may include the revision of existing design procedures and increased cost efficiency for use of the method by local transportation agencies. Future research may also include supplementary experimental testing to address the influences of pile orientation and truncation, and advanced numerical studies (e.g., finite element) to address the influences of interactions between adjacent piles.

ACKNOWLEDGMENTS

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Early-Age Behavior of Jointed Plain Concrete Pavement Systems

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ABSTRACT

This paper mainly focuses on the early-age behavior of concrete pavement systems under varying temperature and moisture gradients upon construction. In an effort to better understand the early-age behavior of the jointed plain concrete pavements under varying environmental conditions, a field study has been conducted on instrumented portland cement concrete slabs in Platteville, Wisconsin. The study involves on-site measurements, extensive laboratory testing, and analyses of the concrete pavement systems under temperature and moisture profiles using finite element methodology-based analytical tools. The aim of the study is to summarize the laboratory test results for concrete samples and the analysis of

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the early-age slab deflection data captured with linear variable displacement transducers (LVDTs). In the analytical modeling of the slabs, the ISLAB2000 finite element model has been used. Based on the large number of comprehensive finite element analyses, a good match has been observed with the analytical solutions and field measurements, thus capturing the early-age behavior of concrete pavement systems under temperature and moisture profiles. Such important findings are presented and discussed in the paper in relation to the early behavior of concrete pavement systems.

Key words: early-age behavior—finite element methodology—jointed plain concrete pavements—temperature effects

INTRODUCTION

Climatic factors can affect the performance of portland cement concrete (PCC) slabs in numerous ways. Temperature differences between the top and the bottom of the PCC slabs causes curling. A higher temperature at the bottom of the slab results in a positive temperature gradient and causes the corners and the free edges of the slab to curl downwards. A higher temperature at the bottom of the slab results in a negative temperature gradient and causes the corners and the free edges of the slab to curl upwards.

Westergaard provided closed form solutions for corner, edge, and interior loading conditions based on elastic foundation analysis (Westergaard 1926). Westergaard's first paper, accounting for stress and deflection analysis for slabs on grade under a temperature distribution throughout the thickness, was published in 1927 (Westergaard 1927). Bradburry (1938) extended Westergaard's solution to the case of slabs of finite size with Westergaard's assumption fully intact. Over the years, sophisticated finite element tools were developed for analyzing pavement systems with two or more layers and allowing separation between layers (Korovesis 1990; Khazanovich 1994). These tools are also suitable for temperature analysis. Analysis with a non-linear temperature gradient was subsequently adapted to the finite element model (FEM) program, ILSL2 (Khazanovich 1994; Ioannides et al. 1998). ILSL2 was modified to model several support and load transfer conditions and is now called ISLAB2000 (Khazanovich et al. 2000).

Several studies have been made to better understand the actual behavior of the slab under environmental conditions. This paper includes a study of curling in jointed PCC (JPCC) pavements. The study involves site measurement, laboratory testing, and computer analysis. The data obtained from the site study and the laboratory testing were used to model JPCC pavement behavior. In the analysis, a powerful finite element program, ISLAB2000, was utilized. Comparison between the predicted FEM results and the measured results showed that the FEM estimated the shape of the curves reasonably well.

METHODOLOGY

Field data were collected from a project on a Platteville bypass road on US Highway 151. In the project, a 25 cm JPCC pavement was placed over a 10-cm–thick open-graded base layer on top of a 7.5-cm–thick dense-graded base material. The joints were dowelled and tied. In the longitudinal joints, dowel bars were 38 mm in diameter, 460 mm in length, and spaced 305 mm apart. In the transverse joints, tie bars were 12.5 mm in diameter, 600 mm in length, and spaced 900 mm apart.

Two test slabs were instrumented. The first slab was built on October 20, 2004 at 9:30 am CST. It was 4.3 m long and 4.4 m wide. The second slab was constructed on October 27, 2004 at 16:00 pm CST. Its dimensions were 4.4 m in length 4.5 m in width.

iButton temperature sensors were used to collect temperature data. The sensors were placed at seven different locations at 50 mm, 114 mm, 146 mm, 178 mm, 190 mm, 216 mm, and 241 mm from the top of the slab; they were all placed 1 m away from the edge of the pavement. The top view of the installation is given in Figure 1. They were programmed to read every five minutes. Air temperature was also collected continuously during monitoring time.

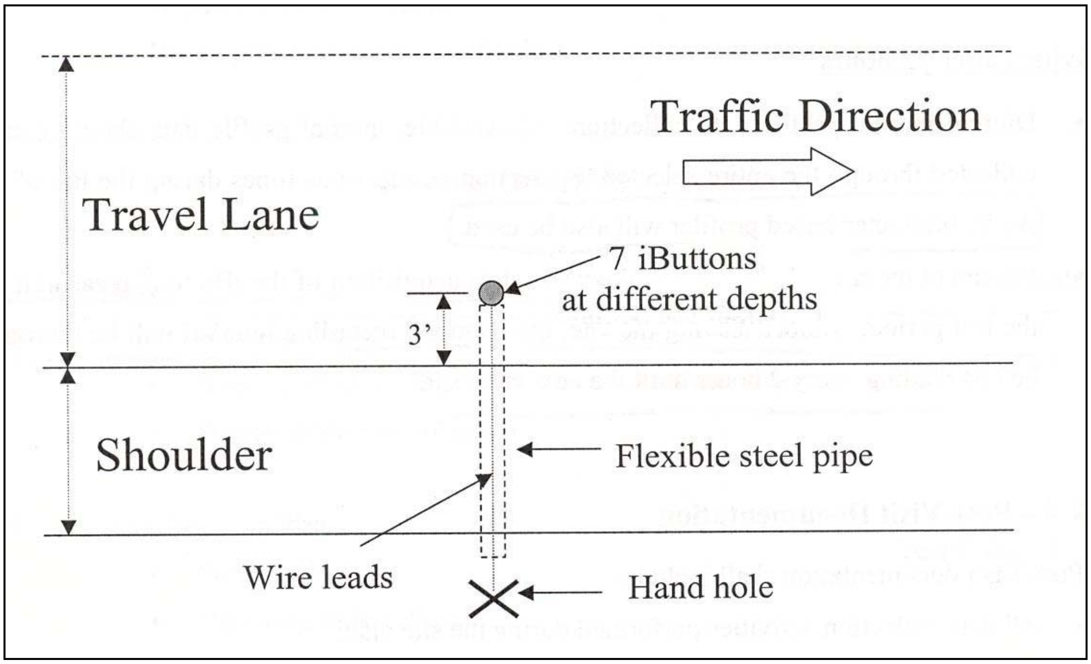

Figure 1. Installation of iButtons

For the first slab, temperature data were collected from October 20, 2004 at 9:45 am CST to October 27, 2004 at 18:25 pm CST. The temperature profile is plotted in Figure 2. For the second section, temperature data were collected from October 20, 2004 at 16:15 pm CST to October 27, 2004 at 17:35 pm CST. The data are shown in Figure 3.

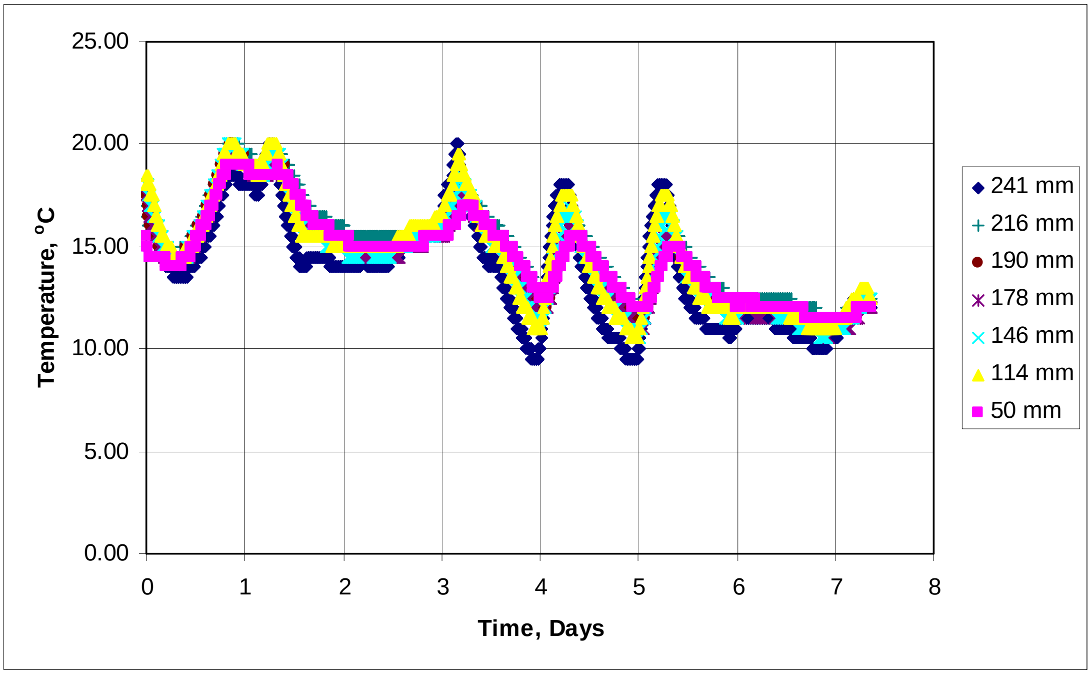

Figure 2. Temperature variation in the first section during the first seven days after pour

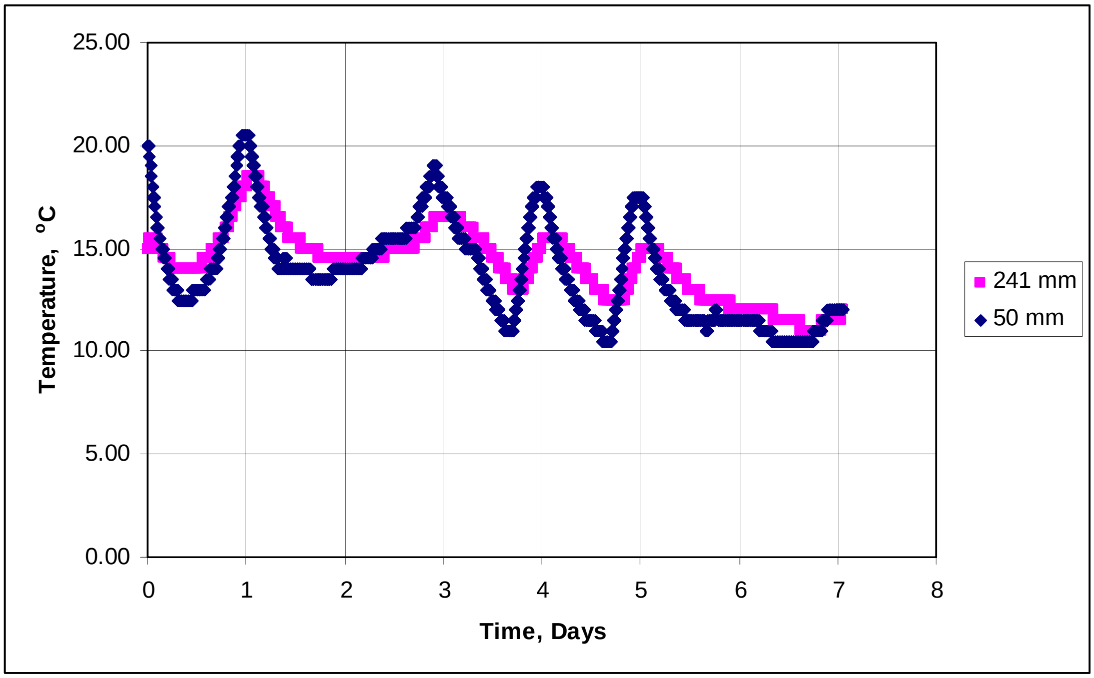

Figure 3. Temperature variation in the second section during the first seven days after pour

Deflection measurements were done using linear variable differential transformers (LVDT). The stroke length of LVDTs is ± 5.0 mm with a sensitivity of 54 mV/V/mm. Five-cm–diameter cores were drilled and steel rods were pounded in these holes without any contact with the concrete slab. LVDTs were clamped to these rods. One of the installed LVDTs from the first section can be seen in Figure 4. Eight LVDTs in the first section and seven LVDTs in the second section were used to profile the deflection of the slabs. In the first slab, deflections were measured from October 21, 2004 at 5:20 pm CST to October 27, 2004 at 5:40 pm CST, and in the second slab measurements were collected from October 22, 2004 at 3:30 pm CST to October 27, 2004 at 5:20 pm CST, every 10 minutes. Layouts of the LVDTs in the first and in the second section are shown in Figure 5.

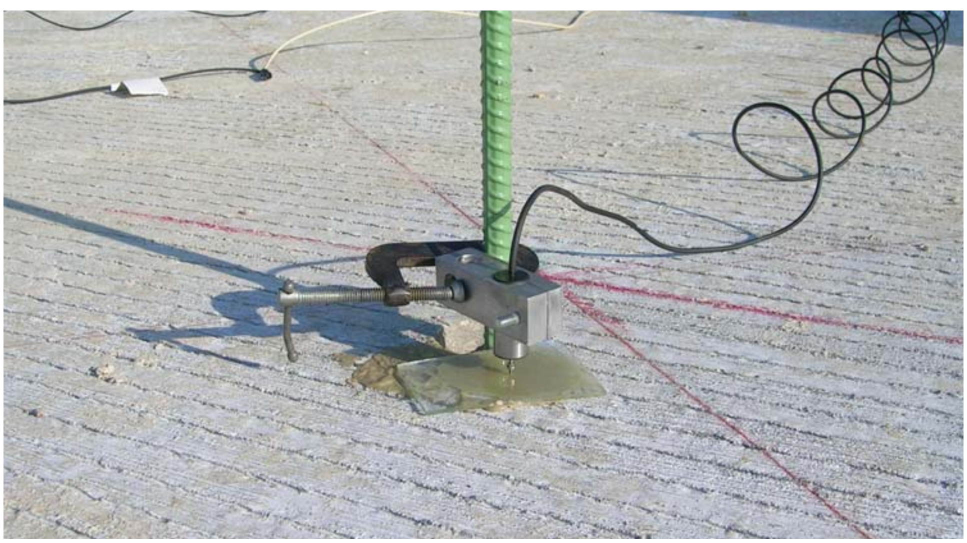

Figure 4. Deflection measurement setup from the first slab

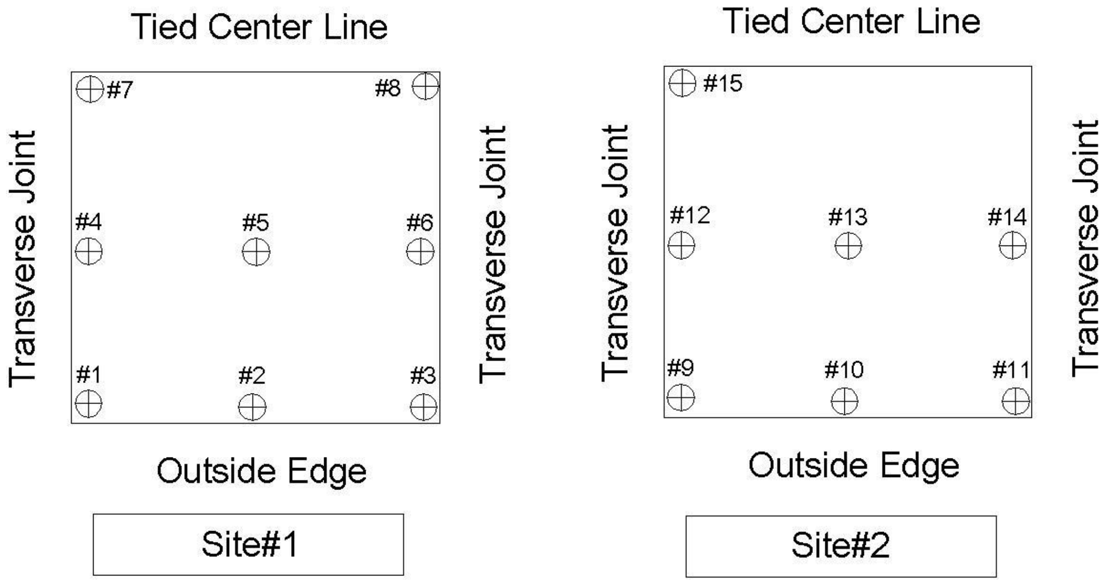

Figure 5. Layouts of the LVDTs in the first and the second site

LVDTs are sensitive devices and can be easily affected by environmental factors. The measurements from three LVDTs (#4, #5, and #7) in the first slab and one LVDT (#15) in the second slab were discarded and not used in the analysis, since it was concluded that the measurements were unreliable.

The tests for the determination of the elastic modulus and the coefficient of thermal expansion were carried out at the Iowa State University Mobile Laboratory and the Iowa DOT, respectively. PCC elastic modulus tests were conducted at interim ages of 12 hours, 1 day, 2 days, 4 days, 7 days, 28 days, and 56 days of age, according to ASTM C 469. The coefficient of thermal expansion of PCC was conducted at 56 days of age, according to AASHTO TP-60.

ANALYSES

In the first slab, the maximum temperature difference was observed on October 25, 2004 from 14:00 pm CST to 14:20 pm CST, with a top-bottom temperature difference of 9 ^{o}C. The minimum temperature difference was observed on October 24, 2004 from 7:30 am CST to 7:50 am CST, with a top-bottom temperature difference of -5 ^{o}C. The temperature of the top of the slab was assumed to be the air temperature. Temperature profiles of the maximum and minimum gradients in the first slab are plotted in Figure 6.

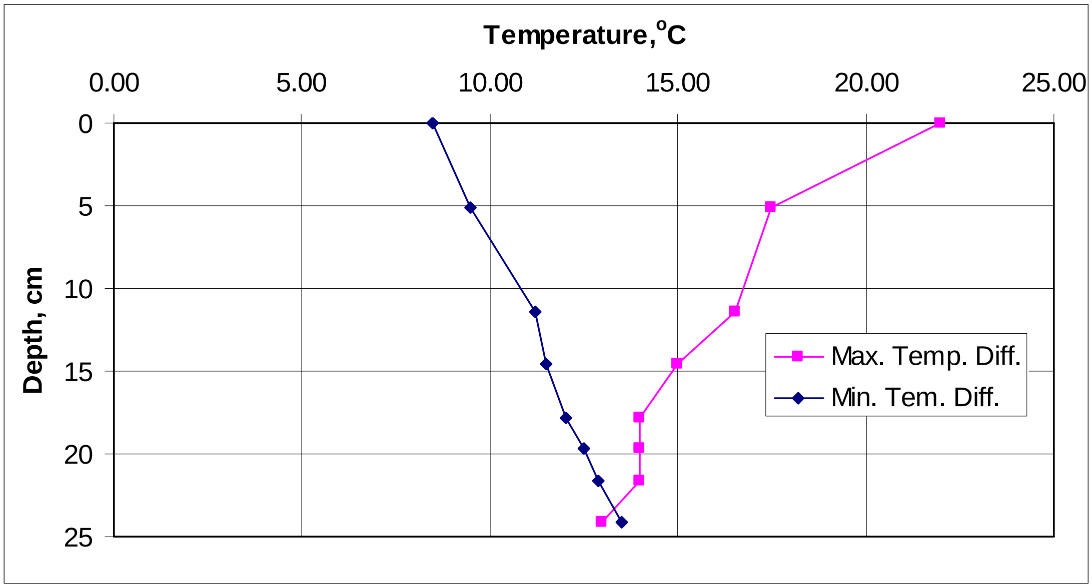

Figure 6. Measured temperature distribution through JPCP thickness at peak gradients in 1st slab

In the second slab, the maximum temperature difference was observed on October 25, 2004 from 14:10 pm CST to 14:20 pm CST, with a top-bottom temperature difference of 8.5 ^{o}C. The minimum temperature difference was observed on October 24, 2004 from 7:20 am CST to 7:40 am CST, with a top-bottom temperature difference of -5.5 ^{o}C. The temperature of the top of the slab was assumed to be the air temperature. Temperature profiles of the maximum and minimum gradients of the second slab are plotted in Figure 7.

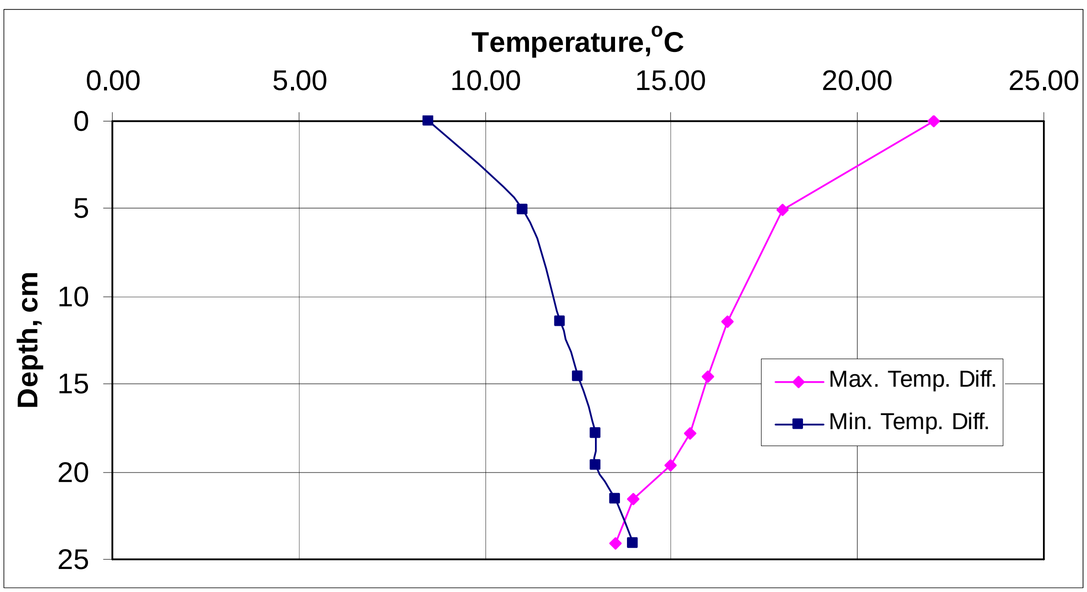

Figure 7. Measured temperature distribution through JPCC thickness at peak gradients in 2nd slab

Material properties determined from the laboratory tests were used in modeling with the FEM program, ISLAB2000. The maximum and minimum temperature difference conditions were modeled in the two sections. Laboratory tests were used to interpolate the modulus of elasticity at the peak gradient time. A modulus value of 20.01 GPa was used for the concrete. The coefficient of thermal expansion, from the laboratory tests of specimens cast in the field, was determined to be 10.4x10^{-6}/^{o}C. For the subgrade, a k-value of 54.2 Kpa/mm was assumed, and dowel bars and tie bars are specified for the joint parameters. For the first site, longitudinal free edge transverse joint measurements and modeling results for the maximum and minimum temperature difference cases are plotted in Figures 8–11.

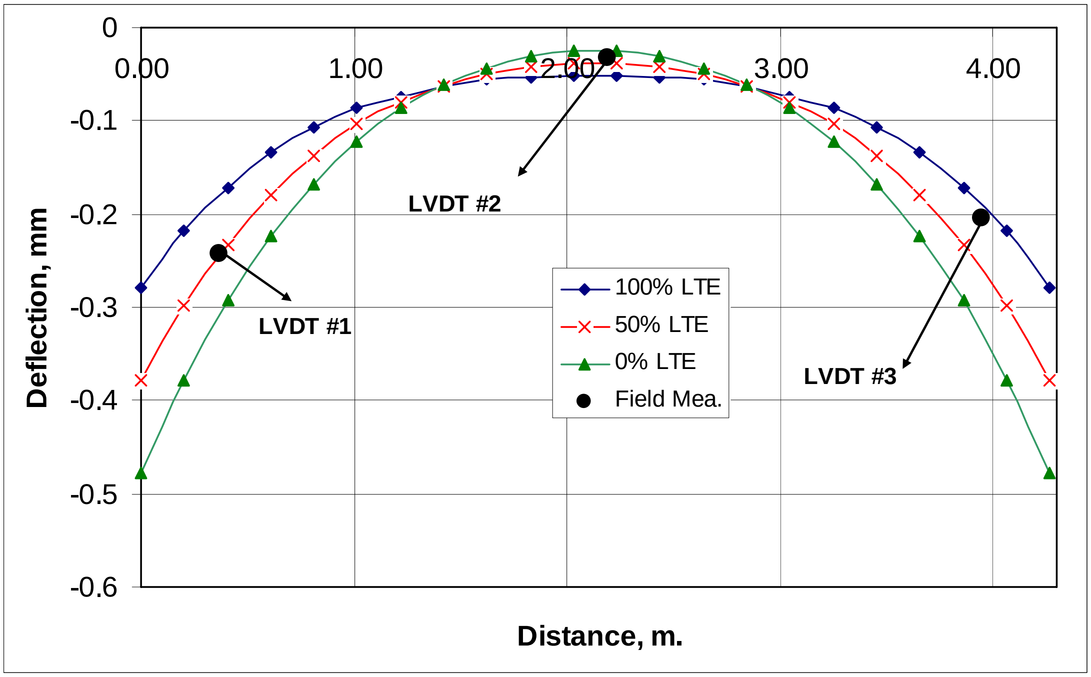

Figure 8. Longitudinal free edge measurements for the curl down case using LVDTs #1, #2, and #3 and the modeling results for the first site

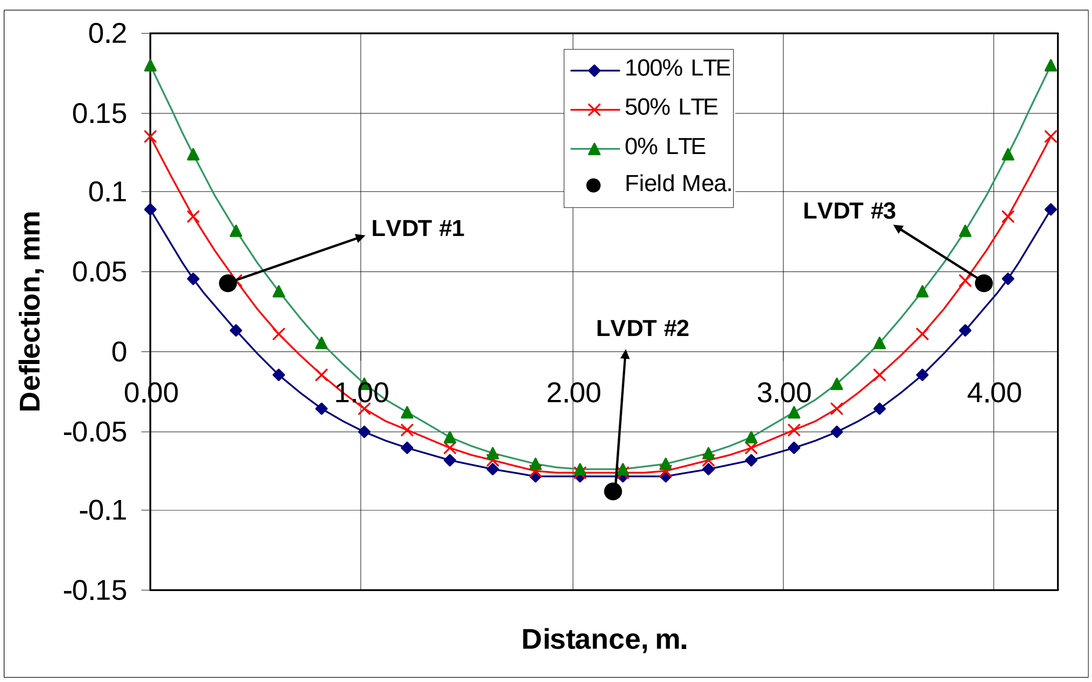

Figure 9. Longitudinal free edge measurements for the curl up case using LVDTs #1, #2, and #3 and the modeling results for the first site

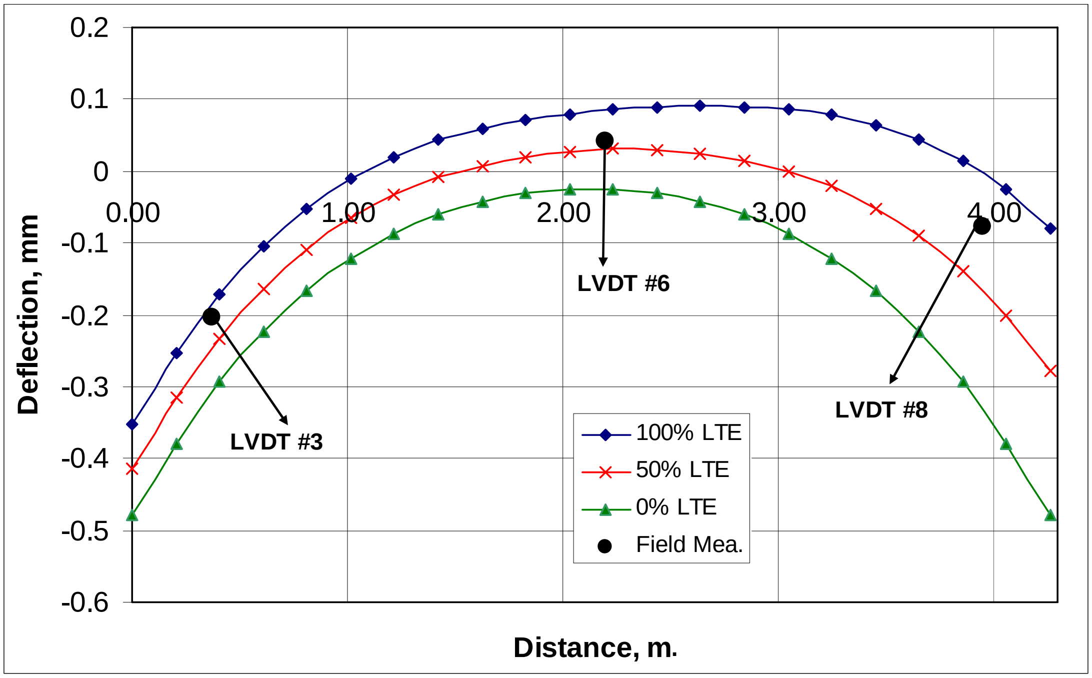

Figure 10. Transverse joint edge measurements for the curl down case using LVDTs #8, #6, and #3 and the modeling results for the first site

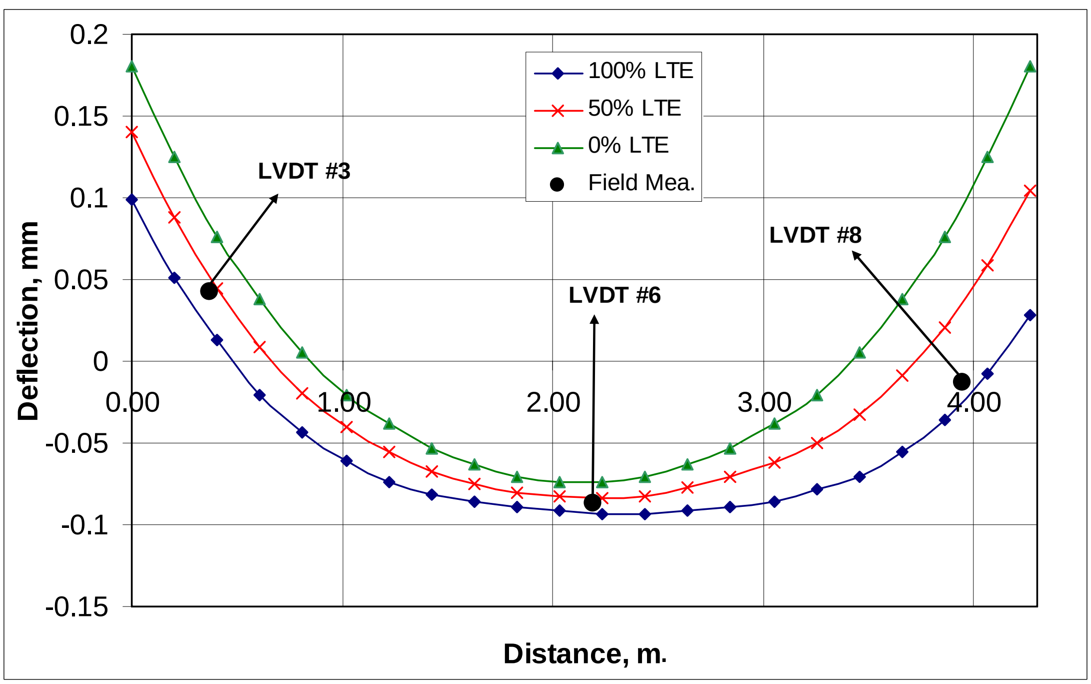

Figure 11. Transverse joint edge measurements for the curl up case using LVDTs #8, #6, and #3 and the modeling results for the first site

For the second site, longitudinal free edge centerline measurements and the modeling results for the maximum and minimum temperature difference cases are plotted in Figures 12–15.

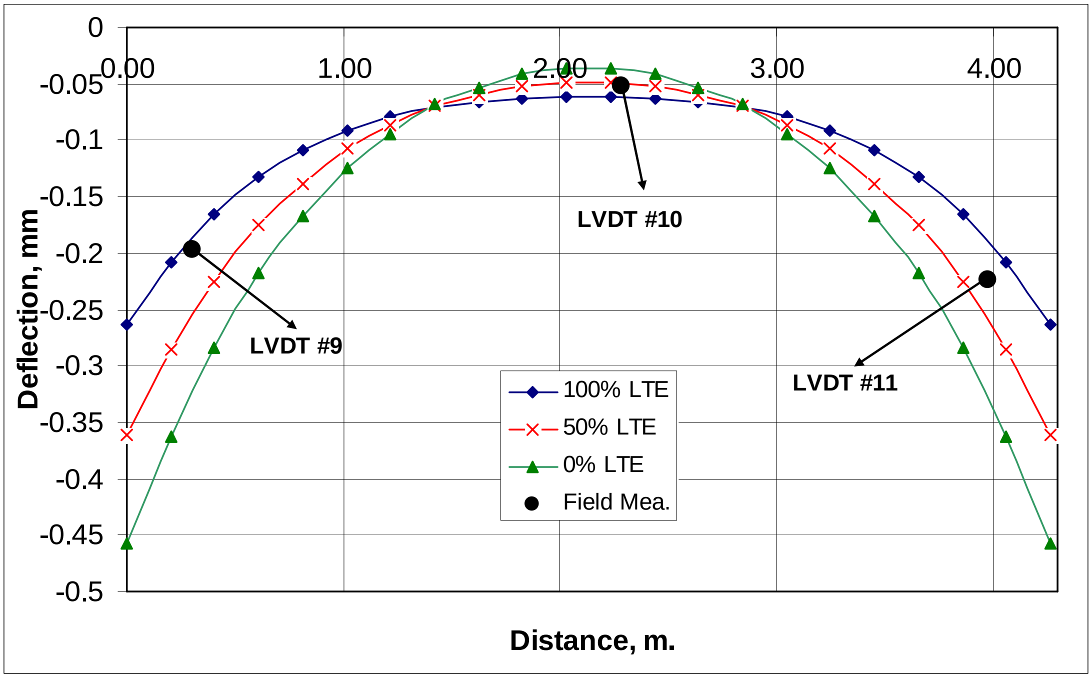

Figure 12. Free edge measurements for the curl down case using LVDTs #9, #10, and #11 and the modeling results for the second site

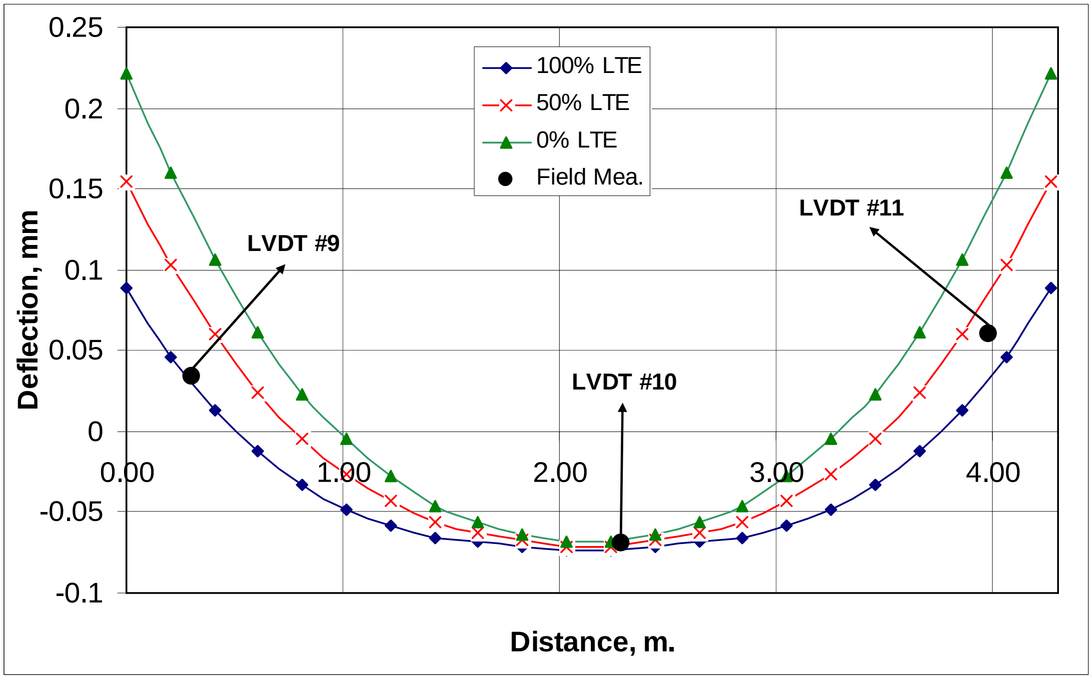

Figure 13. Free edge measurements for the curl down case using LVDTs #9, #10, and #11 and the modeling results for the second site

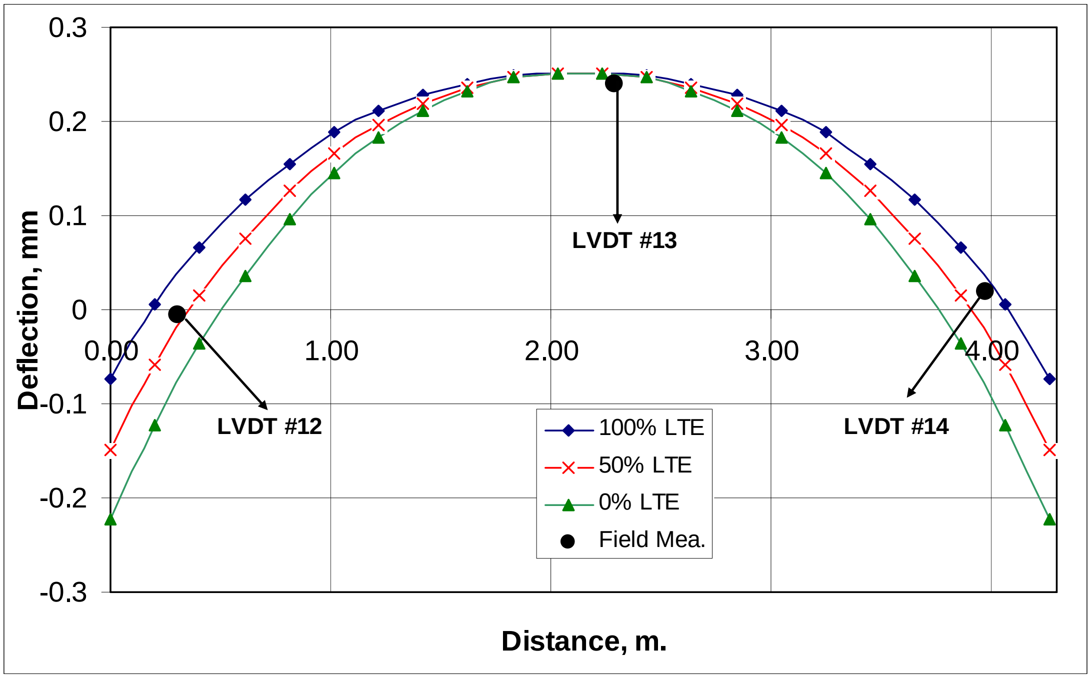

Figure 14. Center line measurements for the curl down case using LVDTs #12, #13, and #14 and the modeling results for the second site

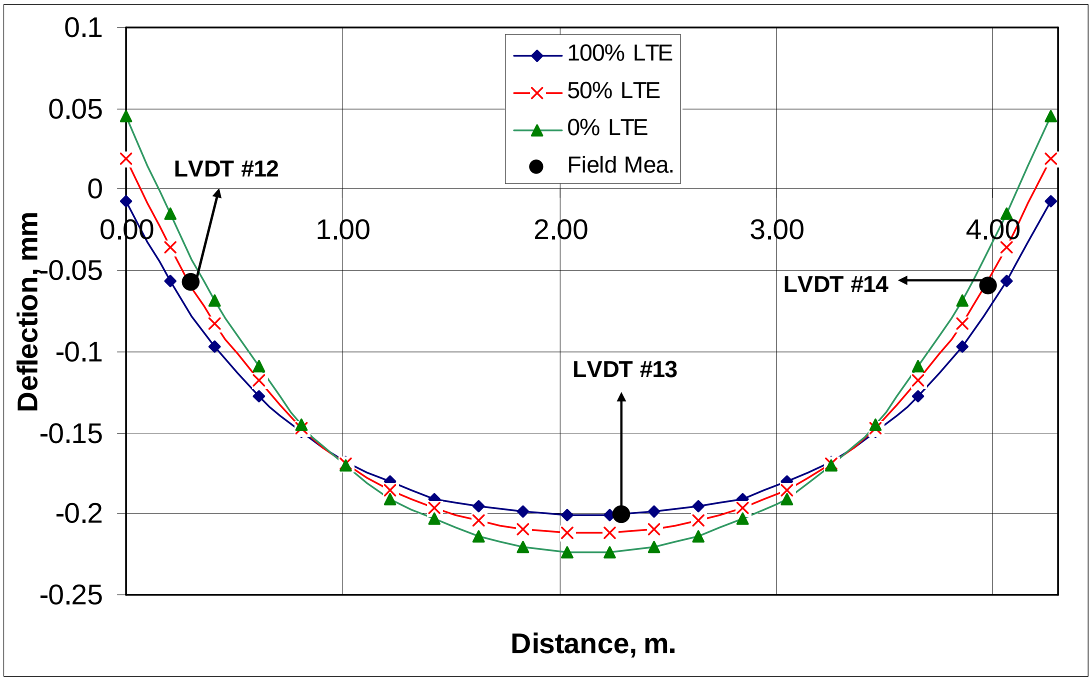

Figure 15. Center line measurements for the curl up case using LVDTs #12, #13, and #14 and the modeling results for the second site.

A comparison between the predicted FEM results and measured results showed that the FEM model estimated the shape of the curves reasonably well. Measured displacement values were usually between the 100% load transfer efficiency (LTE) curves and 50% LTE curves. In the transverse joint edge measurements, free edge measurements (LVDT #3) are more than the tied corner measurement, which is to be expected. Free edge and center line measurements are almost symmetrical, due to the different restraining conditions, such as different load transfer efficiency levels in the joints and different base modulus values. Perfect symmetrical conditions in the measurements cannot be expected. Moreover, a discrepancy between the measured and predicted values might occur because of the moisture difference between the top and bottom of the slab, which is not accounted for in the study. However, according to the national weather station, the average humidity value on the casting day and the monitoring time averages are 85% and 80.67%, respectively. The authors believe that the moisture difference between the top and the bottom of the slab did not play a big role during the monitoring period.

CONCLUSION

The early-age behavior of jointed plain concrete pavements under varying temperature profiles has been the focus of this study. Pavement deflection data collected from instrumented slabs at a Platteville, Wisconsin bypass road have been analyzed. Extensive concrete testing was conducted using the Iowa State University Mobile Concrete Laboratory and the concrete testing laboratories located at Iowa State University and the Iowa DOT. Using the laboratory and field-collected data, the early-age behavior of the jointed plain concrete pavements was modeled by the ISLAB2000 finite element model. The comparison of the field data and the FEM results showed that the early age behavior of the jointed plain concrete pavements under varying temperature profiles can properly be modeled.

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Rapid Field Testing Techniques for Determining Soil Density and Water Content

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ABSTRACT

Proper control of soil water content and soil compaction during earthwork construction operations is critical in achieving adequate performance of structural fills. Conventional methods (e.g., sand cone, rubber balloon, nuclear density gauge) are not always reliable and may require more time than is available to expedite construction. To improve upon the current approach, new rapid and reliable field testing technologies are needed. As part of this effort, this paper describes an evaluation of relatively inexpensive equipment that uses time domain reflectometry (TDR) and capacitance measurement techniques to determine soil density and water content in the field. Comparisons were made to the conventional nuclear and drive core methods for four different soil types and were used to estimate the accuracy and precision of each device and grounds for their successful use. The devices used were an IMKO TRIME TDR, Campbell Scientific DMM600 Duff Moisture Meter, and Humboldt nuclear moisture-density gauge. Gravimetric sampling via drive cores was used as the reference test method. Results show that TDR measurements of volumetric soil moisture are accurate without soil-specific calibration and, given a priori knowledge of gravimetric moisture content, could be used to estimate soil dry unit weight. The Duff Moisture Meter was used in conjunction with a modified drive core developed by the author to measure both volumetric moisture content and the total unit weight of a soil sample. For moisture content determination relative to oven-dried moisture content, the nuclear and TDR methods were less erratic overall than the DMM600. No method compared favorably to drive core dry unit weight; the nuclear method had the lowest RMSE and SEP values, but also the lowest R^2 values overall compared to the TDR and DMM600 methods. The average time to perform each test was two to three minutes for the DMM600, less than one minute for the TDR (two replications), and two to five minutes for the nuclear moisture-density gauge (two replications).

Key words: capacitance— nuclear density gauge— soil moisture—time domain reflectometry

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INTRODUCTION

Soil moisture content and unit weight measurements are key elements to any soil compaction quality control program. The nuclear moisture-density gauge has been the standard tool in geotechnical engineering in recent years. However, regulatory burden, recurring costs, and safety concerns have made alternative techniques to the nuclear gauge very attractive. In 1980, Topp et al. popularized time domain reflectometry (TDR) as a method to measure soil water content using soil electromagnetic properties. Since that time, electronic methods of soil characterization have seen much development (Veenstra et al. 2005). The TDR method has been used extensively in long-term data acquisition experiments, where the inability to leave a nuclear type instrument unattended was prohibitive. In the past few years, new commercial offerings developed specifically for geotechnical engineering applications have been released. The objective of this study was to compare TDR, capacitance, nuclear moisture-density gauge, and drive core methods of moisture and density measurement.

BACKGROUND AND DEFINITIONS

Soil is considered to be a three phase system consisting of soil solid particles, water, and air, as illustrated in Figure 1.

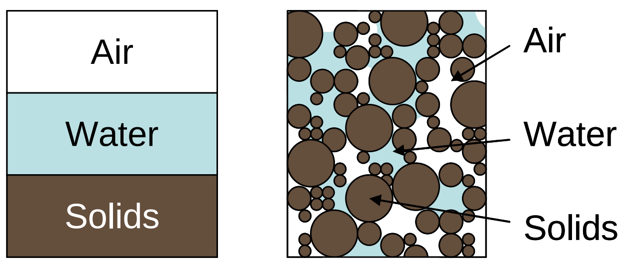

Figure 1. Soil phase diagram (left) and generalized illustration of a soil cross-section (right)

The following definitions are used throughout this paper:

Gravimetric soil water content is defined as

$$\theta_g = \frac{\text{Mass of Water}}{\text{Mass of Solids}} \tag{1}$$

Volumetric soil water content is defined as

$$\theta_v = \frac{\text{Volume of Water}}{\text{Total Volume}} \tag{2}$$

Dry unit weight is defined as

$$\gamma_{dry} = \frac{\text{Weight of Solids}}{\text{Total Volume}} \tag{3}$$

To convert between gravimetric and volumetric water content

$$\frac{\theta_v}{\theta_g} = \frac{\gamma_{dry}}{\gamma_{water}} \tag{4}$$

Where $\gamma_{water} = 9.8\ kN/m^3$.

METHODS AND MATERIALS

Equipment

Time Domain Reflectometry

Time domain reflectometry (TDR) is an electronic technique that may be used to measure the moisture content of soil (Topp et al. 1980). The method works by applying a voltage signal to a transmission line (metal rods) inserted into the soil. The time required for the voltage signal to travel from the source to the end of the transmission line and back again is determined principally by the moisture content. The TDR instrument used in this study was an IMKO TRIME®-EZ probe with coated rods 16 cm long by 6 mm IN diameter (Figure 2). A comprehensive review of TDR theory and techniques is given by Robinson et al. (2003), Ferre and Topp (2002), and O'Connor and Dowding (1999).

Figure 2. IMKO TRIME-EZ probe (left) and inserted into soil (right) with TRIME Data Pilot

DMM600 Moisture Meter

The DMM600, a capacitance-based dielectric moisture sensor, is manufactured by Campbell Scientific, Inc., (Logan, Utah) and is traditionally marketed for use in the forestry service. The DMM600 was designed to measure the moisture content of the upper layer of soil covering the forest floor, known as duff, a highly organic soil containing detritus, vegetation, etc. Moisture measurement consists of placing duff loosely into the sample chamber then compressing the material to a preset pressure against a sensor plate containing the waveguides (see Figure 3) used to measure the dielectric constant of the material. In this study, a modified procedure was developed in which a special soil cutter containing an intact soil sample is placed into the DMM600 chamber. The DMM600 provides the moisture content while the cutter is used to determine the sample density from the known volume and mass as measured on a portable electronic balance.

Figure 3. DMM600 components: (a) sensor plate/waveguides; (b) electronics housing; (c) sample chamber; (d) sample chamber cap, including compression knob (right) and compression plate; (e) special soil cutter tool (0.75 inches high by 2.88 inches in diameter)

Nuclear density gauge

The nuclear density gauge used in this study was manufactured by Humboldt Manufacturing, Inc., (Model 5001) (see Figure 4). For this study, the nuclear tests averaged a measurement over the top six to eight inches of the soil. Moisture content measurement was performed according to ASTM D3017 and density measurement according to ASTM D2922.

Figure 4. Humboldt Model 5001 nuclear density gauge

Drive core

Drive core and/or bag samples were collected at each test location to determine water contents using the oven method. The soil density was determined from the drive core volume. The nominal dimensions of drive cores used in this study were three inches in diameter by three inches in height (see Figure 5). The drive core samples were taken in the top two to five inches of the soil.

Figure 5. Drive core sampling device

Materials

The names and engineering properties of soils evaluated in this study are provided in Table 1.

Table 1. Schedule of testing materials

Soil name	Kickapoo topsoil (1)	Kickapoo fill clay (2)	Edwards till (3)	Kickapoo sand (4)
Texture	Silty clay loam	Silty clay loam	Loam	Sand/Loamy sand
USCS	ML Silt	CL Lean clay	CL Sandy lean clay	SW-SM Well graded sand with silts
AASHTO (GI)	A-6 (13)	A-7-5 (25)	A-6 (7)	A-3
F_{200} (%)	96.9	97.7	67.8	10.0
LL (PI)	38 (12)	47 (22)	29 (15)	NP
$w_{natural}$ (%)	30.1	29.9	6.3	9.8
Standard Proctor:	---	---	---	---
γd, max (kN/m^3)	15.79	16.02	18.61	17.99
w_{opt} (%)	19	20	12	9
Modified Proctor:	---	---	---	---
γd, max (kN/m^3)	17.36	18.14	20.74	18.69
w_{opt} (%)	17	12	8	9
Cation Exchange Capacity (%)	22.5	17.6	14.2	---
Organic Matter (%)	2.8	1.8	3.1	---

Experimental Setup

Test strips were established in the native Edwards till of the test site and the strip bases were stabilized with liberal compaction. Test soils (Kickapoo Topsoil, Kickapoo Fill Clay, Edwards Till, and Kickapoo Sand) were placed in the strips and mixed in situ to achieve relatively uniform, homogeneous soil conditions, inclusive of moisture content. Prior to construction, achievement of appropriate moisture content was verified by drying select soil samples in a microwave. The moisture content was accepted once the moisture content from spot tests was within 2% of the desired moisture for each test strip. Water and/or wet soil were added to test strips containing soil too dry for testing. Soil too wet for testing was air-dried and occasionally mixed.

For testing, 10 test points were established for each test strip at 1.52-m intervals. Density and moisture values for the uncompacted fill were determined with a nuclear density gauge. Tests were performed for one pass, two passes, four passes, and eight passes of the roller. Following the eighth roller pass, drive cores were excavated for a direct measurement of density and moisture.

Average moisture contents and dry densities of the soil measurements are provided in Table 2, as determined by drive core. These moisture contents represent typical values encountered in earthwork construction operations.

Table 2. Nominal moisture contents and dry densities of soils tested after eight roller passes, determined by drive core

	Strip 1			Strip 2			Strip 3		
Soil Name	**θ_g (%) (cov*)**	**γ_{dry} (kN/m^3) (cov)**	**N†**	**θ_g (%)**	**γ_{dry} (kN/m^3)**	**N**	**θ_g (%)**	**γ_{dry} (kN/m^3)**	**N**
Kickapoo Topsoil	23.2 (2.8)	15.43 (1.6)	18	16.0 (3.8)	15.16 (4.1)	10	18.8 (4.1)	15.65 (3.2)	34
Kickapoo Fill Clay	23.9 (3.1)	15.57 (1.5)	24	15.8 (9.7)	16.04 (4.4)	10	20.0 (8.9)	16.20 (1.9)	10
Edwards Till	7.9 (4.2)	15.61 (1.9)	19/4	16.4 (5.4)	17.41 (2.2)	18	10.9 (8.3)	16.97 (6.4)	18
Kickapoo Sand	5.6 (8.2)	16.97 (1.9)	20	8.8 (5.6)	17.78 (3.1)	20	---	---	---

* coefficient of variation † number of samples

RESULTS AND DISCUSSION

Tests were performed to compare the relative accuracies of TDR, DMM600, and the nuclear density gauge with respect to drive core results for soil moisture and density.

Determination of Moisture Content by Time Domain Reflectometry

The TDR system was used to determine the volumetric moisture content at each test point and then was converted to gravimetric moisture content using equation (4) and the soil density, as determined by drive core. The TDR-determined moisture content was then linearly regressed against oven-determined moisture contents from the drive core samples, hereafter referred to as the drive core moisture. The results for topsoil, till, and sand are shown in Figure 6. The clay soil is not included because insufficient TDR data points were collected; this was because the clay lift thicknesses were less than the length of the TDR probe. The average time to perform each test was less than one minute (two replications).

TDR moisture determination showed strong correlation relative to drive core moisture measurements (oven drying method). The soil density must be known to calculate gravimetric moisture content, and therefore error in soil density measurement will propagate into the calculated gravimetric moisture content. For this reason, the drive core was taken as close to the TDR measurement site as reasonable.

Unlike the nuclear density gauge, which may use man-made materials (e.g., metal, plastic) for calibration, TDR requires calibration in soil. In this study, it was not practical to perform a material-specific calibration prior to field testing, so the factory calibration was used and the calibration performed after the data was collected.

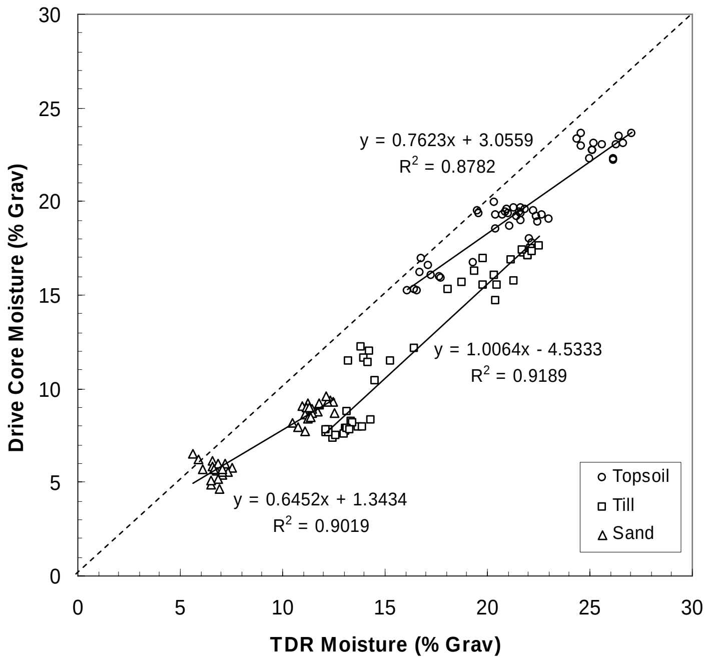

Figure 6. TDR vs. drive core moisture content for topsoil, till, and sand soils

Determination of Moisture Content by DMM600

The DMM600 frequency output is plotted against the drive core volumetric moisture content. A second-order polynomial fit is used per the manufacturer's calibration instructions. The DMM600 volumetric moisture content is then converted to gravimetric moisture content using equation (4) with soil density as determined by drive core. The DMM600 moisture content is then plotted against the drive core moisture content, as shown in Figure 7.

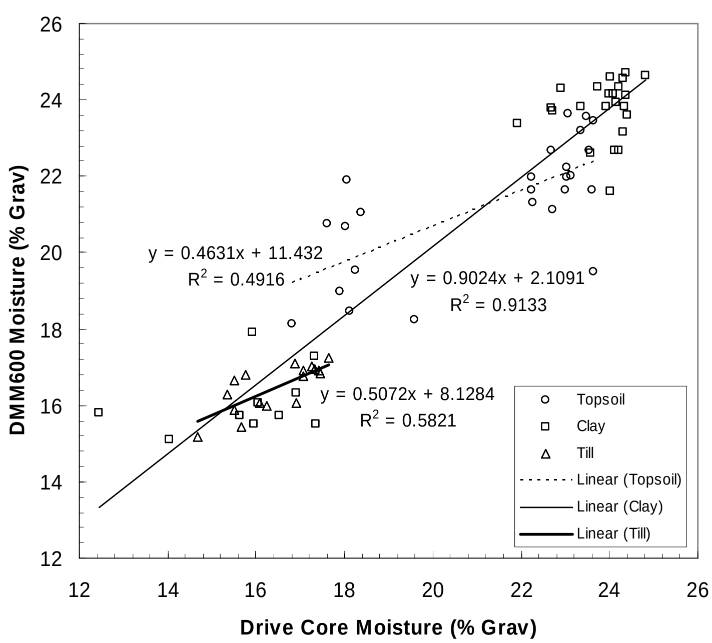

Figure 7. DMM600 moisture content vs drive core moisture content for topsoil, clay, and till soils

DMM600 results were not as well correlated as the TDR and nuclear gauge results and were relatively poor compared to a laboratory study using the same methodology (Veenstra et al. 2005). Compared to laboratory-prepared samples, field soils were much coarser and less structurally homogeneous, resulting in large gaps in the sample surface and prohibiting full contact between the soil sample and the sensor plate. Also, large stones in the sample would considerably lower the measured permittivity, resulting in anomalously low moisture content. These conditions were especially true for the till, which was very stiff and crumbly at a low moisture content. It was not possible to use the DMM600 with the sand because the sand would not stay within the soil cutter. The average time to perform each test was about three minutes.

Determination of Moisture Content by Nuclear Moisture-Density Gauge

Nuclear gauge moisture content is plotted against drive core moisture content in Figure 8–9. Nuclear gauge measurements had good correlation with drive core moisture measurements, comparable to TDR moisture measurements. The accuracy of moisture content determination using a nuclear gauge is directly proportional to the measurement time, with longer measurement times producing higher accuracy. The measurement time in this study was 15 seconds; the total time for two replications is about 2 to 5 minutes.

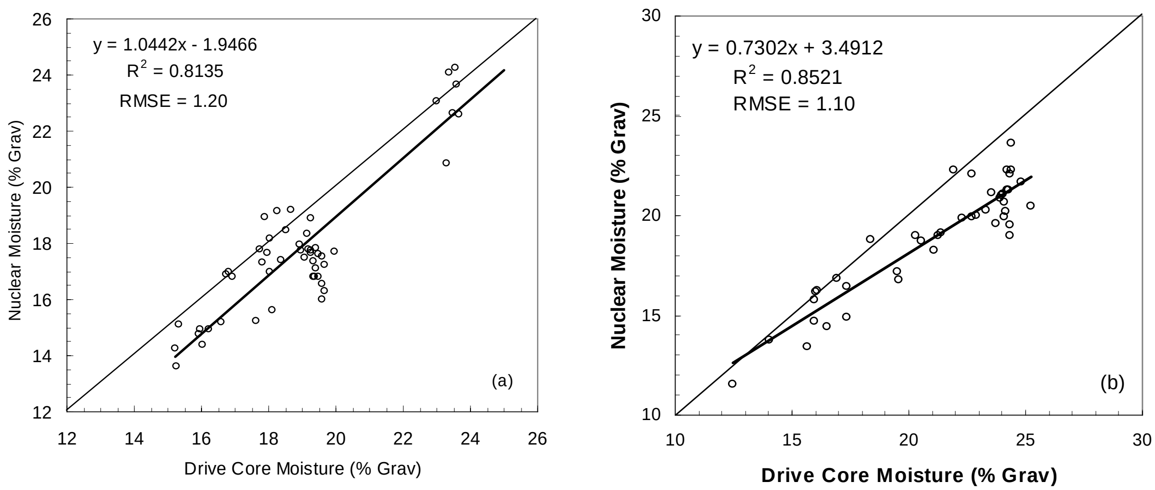

Figure 8. Nuclear moisture gauge vs. drive core moisture for topsoil (a) and clay (b)

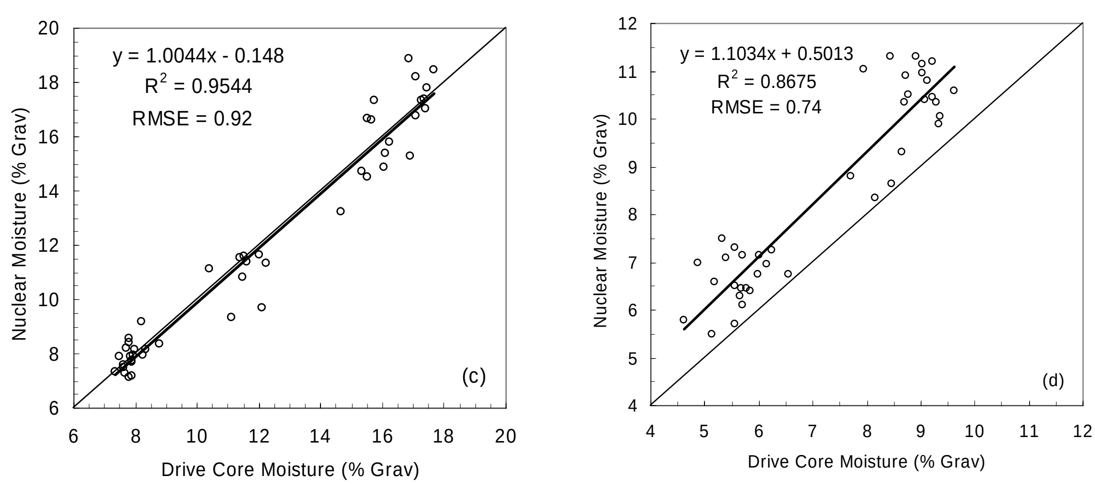

Figure 9. Nuclear moisture gauge vs. drive core moisture: till (c) and sand (d)

Determination of Dry Unit Weight by Time Domain Reflectometry

Knowing the gravimetric moisture content and assuming that it stays constant throughout the compaction process, the volumetric moisture content determined by TDR will reflect changes in soil density. The TDR-determined dry unit weight is calculated using Equation (5), where $\theta_{v\,(tdr)}$ is the TDR-determined volumetric moisture content.

$$\gamma_{dry\ (TDR)} = \frac{\theta_{v_{(TDR)}} \cdot \gamma_{water}}{\theta_{g_{(Nuc)}}} \tag{5}$$

The calculated TDR dry unit weight is plotted against nuclear gauge-determined dry unit weight in Figure 10. TDR dry unit weight is compared to nuclear gauge density because this allows more data points over a wider range than if compared solely to drive dry unit weight. Insufficient data points were collected to calculate the TDR dry unit weight for clay because the lift thickness was often too shallow for TDR measurement.

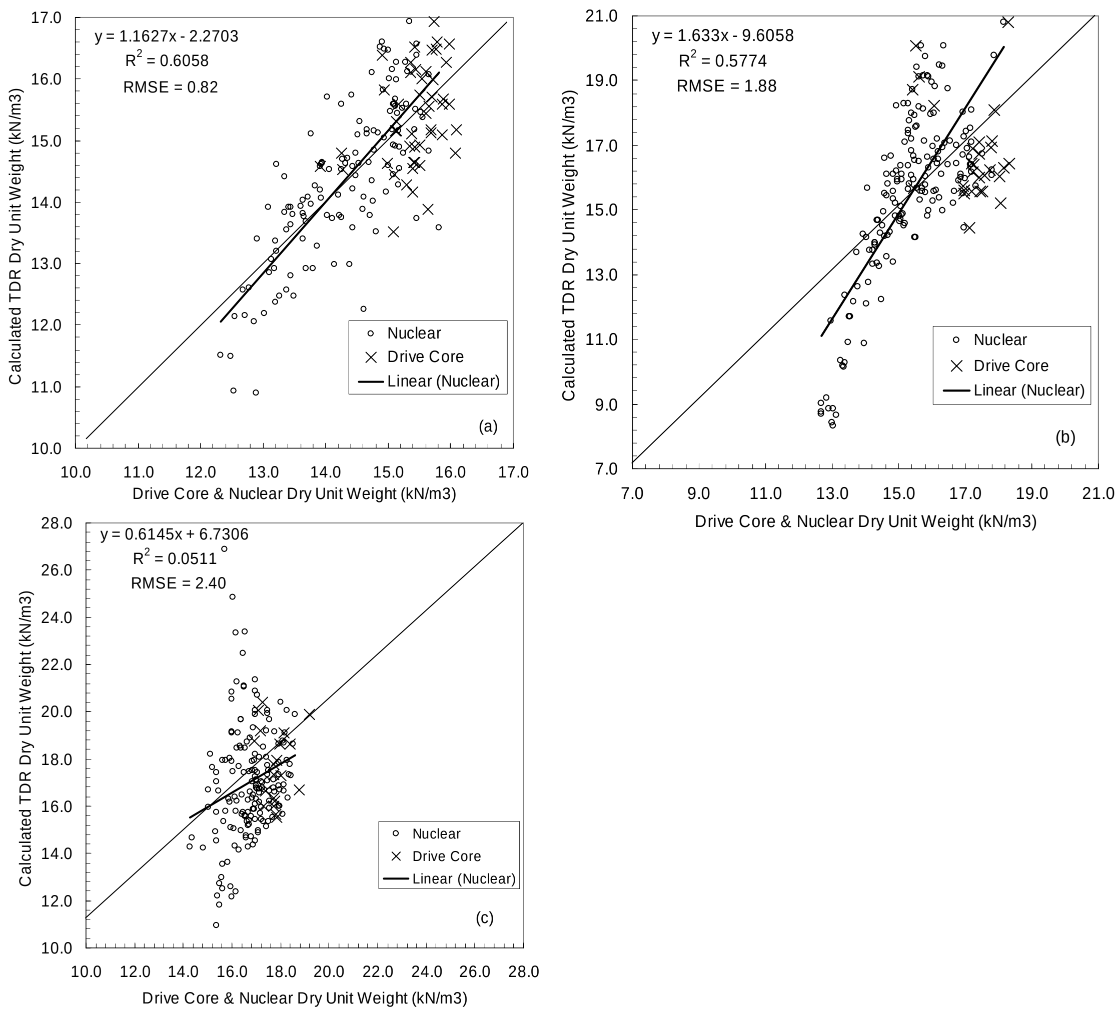

Figure 10. Drive core and nuclear-determined dry unit weight vs. TDR dry unit weight for topsoil (a), till (b), sand (c); linear regression based on nuclear data

A major difference between the TDR and nuclear methods of unit weight determination is sample volume. A nuclear density gauge has a sample volume of about 6,200 cm^3 depending on source depth (Humboldt Mfg. Co. 2004); the TDR measurement volume is estimated to be less than 200 cm^3. Thus, TDR is more sensitive to small-scale variation in soil unit weight than the nuclear density gauge.

Determination of Dry Unit Weight by DMM600

The DMM600 dry unit weight is determined using the mass of the soil, the soil cutter volume, and the corresponding drive core moisture content. The DMM600 dry unit weight is plotted against the dry unit weight determined by drive core, as in Figure 11. The sand was not tested because it failed to stay within either the drive cores or the soil cutter.

The measurement volume of the drive core is 320 cm^3, while that of the soil cutter is only 80.1 cm^3; thus, the soil cutter is more sensitive to spatial variation in soil density than the drive core. Also, trimming the soil cutter samples was problematic because the draw knife would often pull material out of the soil cutter. Because of the difference in measurement volume, soil unit weight determined by the soil cutter is more sensitive to errors in soil mass measurement than the drive core. An error of 1 gm (e.g., due to soil falling out of the soil cutter or drive core) soil mass will result in an error of only 0.03 kN/m^3 unit weight for the drive core but 0.13 kN/m^3 for the soil cutter.

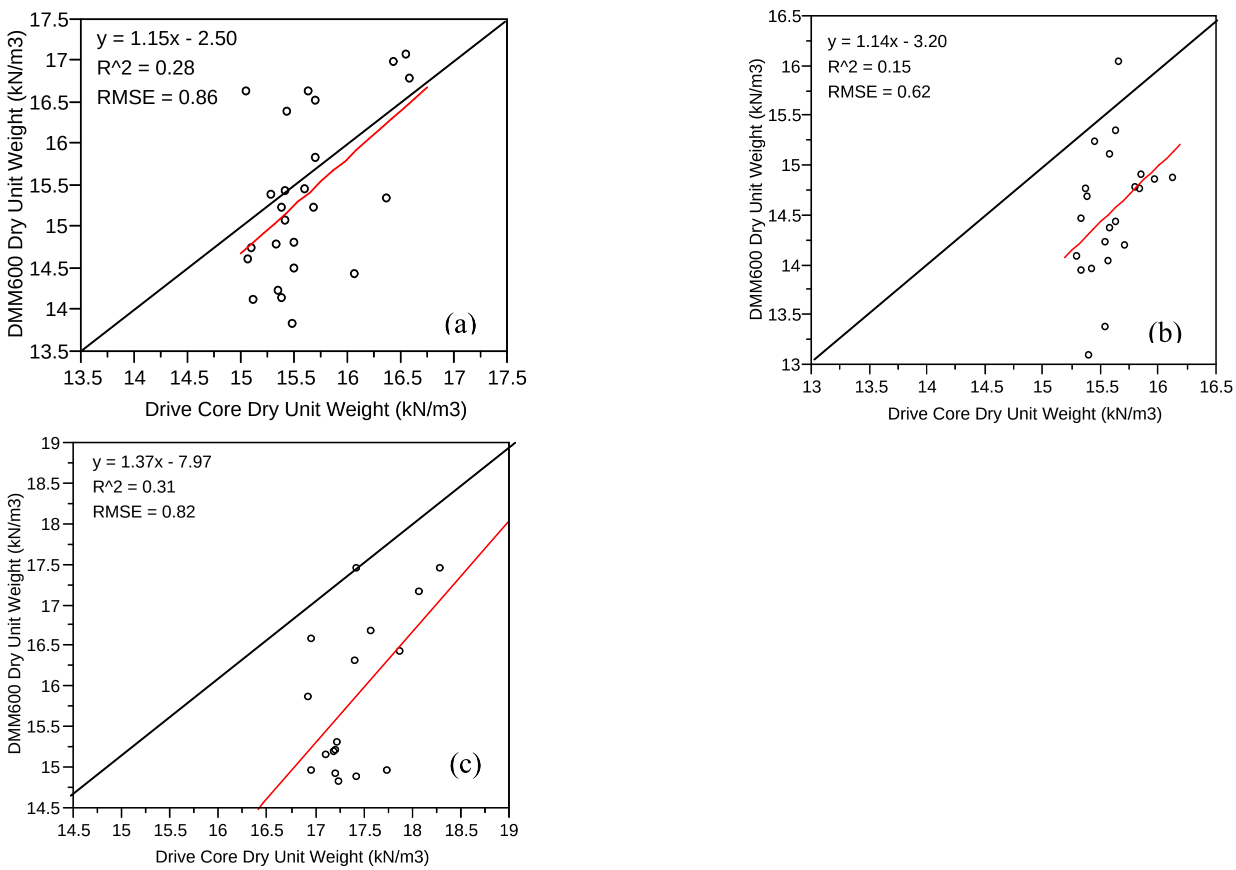

Figure 11. Drive core dry unit weight vs. DMM600 dry unit weight for topsoil (a), clay (b), till (c)

Determination of Dry Unit Weight by Nuclear Moisture-Density Gauge

The dry unit weight as determined by nuclear density gauge is plotted against drive core determined dry unit weight in Figures 12–13. There is very poor correlation between nuclear density gauge and drive-core–determined dry unit weight. Again, to account for this poor correlation, it is important to consider the difference in measurement volume of the nuclear gauge and drive core. To increase the accuracy of nuclear measurements, a longer measurement time could be used.

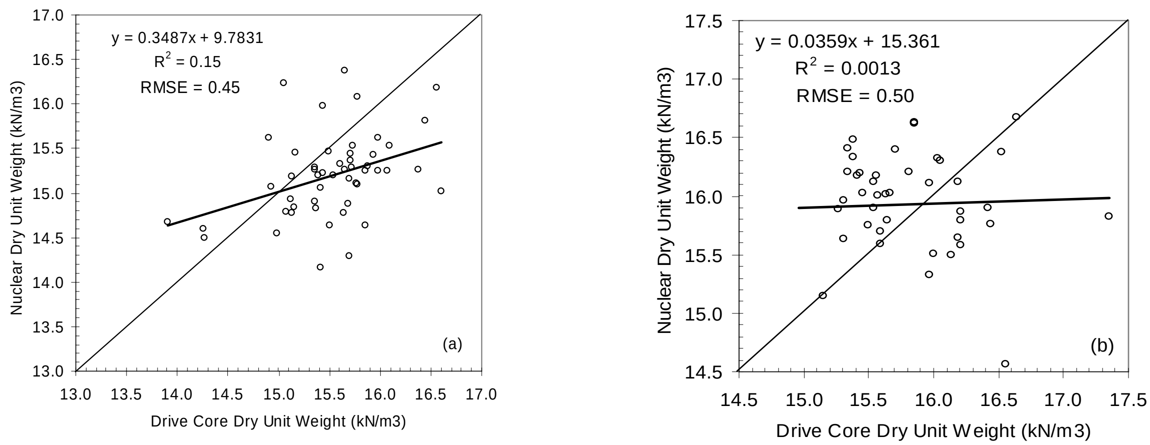

Figure 12. Nuclear density gauge vs. drive core dry unit weight for topsoil (a) and clay (b)

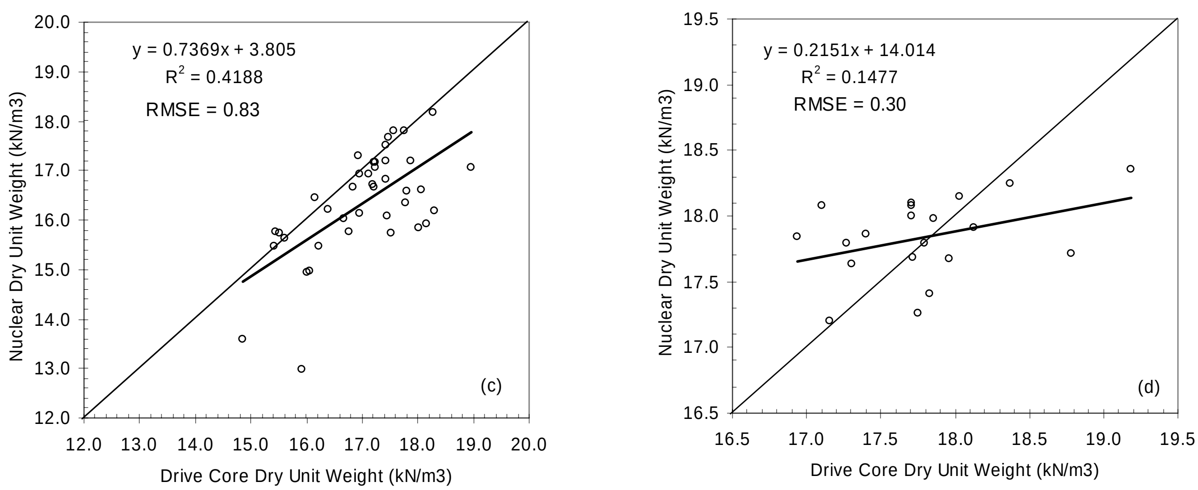

Figure 13. Nuclear density gauge vs. drive core dry unit weight for till (c) and sand (d)

Summary of Results

A summary of results is provided in Table 3 for moisture content and Table 4 for dry unit weight. For each method and soil type, the coefficient of determination (R^2), root mean square error (rmse), and standard error of performance (SEP, i.e., the standard deviation of the error) are provided relative to drive-core–determined values; also, the number of samples, N, is provided. The 95% confidence interval (CI_{95}) for a particular method and soil type may be calculated using Equation 6.

$$CI_{95} = \pm 2 \cdot \mathrm{SEP} \tag{6}$$

Table 3. Summary of moisture content results for each method relative to drive core values

Method	Topsoil				Clay				Till				Sand			
	R^2	rmse	SEP	N	R^2	rmse	SEP	N	R^2	rmse	SEP	N	R^2	rmse	SEP	N
Nuclear	0.8	1.2	1.2	52	0.9	1.1	1.4	44	0.9	0.9	0.9	55	0.9	0.7	0.8	40
TDR	0.8	0.9	1.2	48	--	--	--	--	0.9	1.1	1.1	44	0.9	0.5	1.0	39
DMM	0.5	1.2	1.8	25	0.9	1.9	1.1	33	0.6	0.4	0.6	17	--	--	--	--

Table 4. Summary of dry unit weight results for each method relative to drive core values

Method	Topsoil				Clay				Till				Sand			
	R^2	rmse	SEP	N	R^2	rmse	SEP	N	R^2	rmse	SEP	N	R^2	rmse	SEP	N
Nuclear	0.2	0.4	0.6	51	0.0	0.5	0.7	43	0.4	0.8	1.1	40	0.1	0.3	0.5	20
TDR	0.6	0.8	0.8	123	--	--	--	--	0.6	1.9	1.9	168	0.1	2.4	2.2	165
DMM	0.3	0.9	0.8	25	0.2	0.6	0.6	22	0.3	0.8	0.8	17	--	--	--	--

SUMMARY AND CONCLUSION

Moisture measurements using the nuclear moisture-density gauge and TDR compared favorably to oven-dried samples. The DMM600 did not compare as well, but it is wrong to conclude that the capacitance method is inferior to the TDR or nuclear gauge methods; rather, the measurement technique employed in this study was probably unsuitable. No technique compared favorably to the drive-core–determined unit weight; however, it is apparent that the scale of measurement volume has a significant influence on unit weight determination. The most rapid technique was TDR (~ one minute), then nuclear density gauge (~ two to five minutes), and finally the DMM600 (~ three minutes).

Electromagnetic techniques such as TDR provide sufficient accuracy compared to oven-dried samples to be used in place of the nuclear moisture-density gauge. However, unlike the nuclear moisture-density gauge, the TDR cannot simultaneously measure unit weight without additional measurements. A major disadvantage of TDR to nuclear moisture-density gauge is the need for soil-specific calibration.

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Traveler Information Systems to Support Transportation Operations (Invited Presentation)

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ABSTRACT

This project is a pooled fund consortium of 15 states and provinces working together to advance the deployment of innovative technology, validate and improve ITS standards, and promote the use of travel information systems to support transportation operations. The result of the Pooled Fund is the Condition Acquisition Reporting System (CARS) and related modules that currently serve as the backbone of traveler information systems for many states. This collaboration represents a best practice because new states joining the consortium benefit from the years of experience and the monetary investments of member states. Similarly, new states bring an outside perspective and insight to assist existing member states in solving problems. As an example, Rhode Island was able to roll out a statewide condition reporting system, a statewide 511 phone system and a statewide travel information web site for less than $300,000 deployment costs, and meet their rapid rollout schedule.

This traveler information system was developed because State Departments of Transportation needed a cost effective solution supported by emerging ITS standards to enable manual and automated event reporting that could serve as a central repository of all travel-related events statewide. This central system serves as the data content for travel information systems such as 511, DMS signs, and Internet dissemination. The nature of this issue and the ability to share costs through common development led to the natural concept of a Pooled Fund.

The CARS consortium benefits member agencies and the traveling public as follows:

- State DOTs benefit from the experience gained working with other state agencies to collectively solve problems and learn from others experiences.
- Travelers benefit from the delivery of real-time traveler information through 511, web, highway advisory radio, DMS etc. For contiguous states such as Minnesota and Iowa, travelers experience universal information across state borders.
- The information provides real time traveler information on road surface conditions, road work, and incidents that will allow drivers to make better travel decisions.
- The collaboration reduces barriers to deployment and helps maintain advanced technology within a rapidly changing cross- cutting operational environment. Decisions reached by the group literally bring the experiences of thousands of operators currently using the system to help understand (from a real-world perspective) how to improve the systems.

Additional information is available from each state or at the website www.carsprogram.org.

Key words: ITS standards—transportation operations—traveler information system

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Improving Surface Transportation Safety and Effectiveness through Modern Weather Technologies and Information

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ABSTRACT

Roadway safety and cost data estimate that approximately 7,346 fatalities per year, 713,537 injuries, $42 billion in economic costs, and over 544 million hours in delays can be attributed to weather-related accidents and weather events. Since the National Oceanic and Atmospheric Administration and the Federal Highway Administration released W*eather Information for Surface Transportation: A National Needs Assessment Report*, a great deal of progress has been made in bringing new weather products and services to the surface transportation community. Examples include new technology and information capabilities such as 511 systems; new forecasts and visualizations of weather's impact on roadways in national and local media outlets and on the internet; increased surface weather research and development activities by federal and state government, universities, and others; and an increased congressional interest in surface transportation research and development. Additionally, a consensus has formed among the federal meteorological community's leadership that an integrated approach is needed, supported by the surface transportation and meteorological stakeholders, to continue to improve weather information for surface transportation and effectively meet all surface transportation user needs. This paper describes such an integrated approach designed to support the weather information needs of the surface transportation community. It also provides examples of work already underway, and activities that the federal meteorological community is doing to develop a vision to guide an integrated federal surface transportation weather research and development program.

Key words: accidents—surface transportation weather research—weather safety

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Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

ORGANIZATION OF THE FEDERAL METEOROLOGICAL COMMUNITY

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

OFCM carries out its mission through the Federal Meteorological Coordinating Infrastructure, as depicted in Figure 1. Overall policy guidance is provided by the Federal Committee for Meteorological Services and Supporting Research (FCMSSR). Fifteen federal departments and agencies are currently engaged in meteorological activities and participate in the OFCM’s coordination and cooperation 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 federal agencies engaged in meteorological activities or that need meteorological services. OFCM assesses the adequacy of the total federal meteorology program, as well as reviews current and proposed programs, to identify opportunities for improved efficiency, reliability, and cost avoidance through coordinated actions and integrated programs. OFCM also provides analyses, summaries, and evaluations that provide a factual basis for 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 the areas of natural disaster reduction (hurricanes and post-storm data acquisition) and weather information for surface transportation.

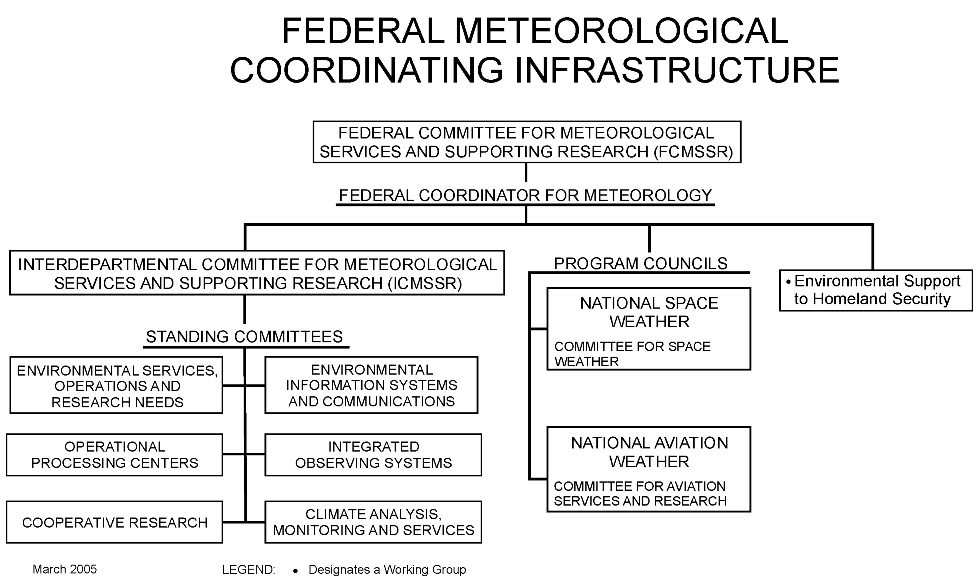

Figure 1. Federal meteorological coordination infrastructure

Based on the OFCM-published *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 weather information for surface transportation (WIST) requirements, research and development (R and D) needs, and services. 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 accomplish the WIST coordination and synchronization.

IMPACT OF WEATHER ON SURFACE TRANSPORTATION SYSTEMS

It is common knowledge that weather has a significant impact on the nation's surface transportation systems. Looking at the roadway piece alone indicates that safety and cost data estimates caused by weather-related accidents and weather events are approximately 7,346 fatalities per year, 713,537 injuries, $42 billion in economic costs, and over 544 million hours in delays. Note that the approximately 7,000 deaths per year is an astounding number that has escaped the general public's attention. Recently, U.S. Transportation Secretary Norman Mineta called the problem of highway traffic deaths a "national epidemic." He went on to indicate that if Americans saw the number of highway traffic deaths as a disease, then they would demand a cure. The much-publicized Baltimore water taxi accident in March 2004 and the October 2004 severe weather event of a fast-moving line of late-afternoon thunderstorms plowing into Interstate 95 traffic north of Baltimore, which caused 11 separate accidents, sent about 50 people to hospitals, and caused widespread traffic disruption in the heavily traveled corridor, further highlight the devastating impact that weather can have on the nation's surface transportation systems.

INTEGRATED APPROACH TO MEET SURFACE TRANSPORTATION WIST USER NEEDS

Major Themes for an Integrated WIST Program

A consensus has formed that there are at least four key components that an integrated WIST program should contain to achieve success in reducing the numbers of weather-related surface transportation deaths, injuries, and delays. Broadly, they are (1) data collection and analysis, (2) research and development, (3) transition R and D to operations, (4) and education and outreach. These themes are already being reflected in both the National Oceanic and Atmospheric Administration's (NOAA) Surface Weather Program and the Federal Highway Administration's (FHWA) Clarus Initiative.

Data Collection and Analysis

In the area of data collection and analysis, federal agencies need to buy and operate sensors that incorporate the latest available and tested technologies to ensure the highest data quality possible is achieved. An example is the FHWA's multi-year Clarus Initiative, which is an initiative to develop and demonstrate an integrated surface transportation weather observing, forecasting, and data management system, and to establish a partnership to create a Nationwide Surface Transportation Weather Observing and Forecasting System.

Additionally, there is a need to obtain and include private-sector sensor data into the federal data base for everyone to use. Such real-time data is needed to support the national deployment of 511 capabilities and support such R and D prototype activities as wireless connectivity that allows vehicles to collect and share data to avoid congestion and potential weather-related accidents.

Another facet of data collection and analysis is the establishment of federal standards. There is a need to standardize surface weather observation formats, accuracy requirements, and data security operations, wherever possible, or to develop some kind of data conversion applications to ensure surface transportation users have the quality of data when and where they need it. Common technologies and standards for access to and assimilation of sensor/measurement data are needed to maximize the data available to all users.

Research and Development

Ongoing R and D efforts must be leveraged wherever possible, such as applying boundary layer and other R and D findings from the aviation weather R and D programs. Some examples can include the Integrated Terminal Weather System, which ties in terminal Doppler weather radar data, the convective weather forecast product, and the low-level wind shear alert system; turbulence measuring and prediction systems; marine stratus forecast system; and weather support to a deicing decision making system.

Unique surface transportation problems must also be solved as a part of a coordinated surface weather R and D program. A few examples are black ice on roadways; turbulent vortices; sun glint and glare; spatial distribution of pavement conditions (temperature, frost, etc.); rainfall spatial variability; and crosswind, headwind, and turbulence.

Moreover, coordination among the WIST R and D programs and weather providers must be expanded. There are good starting points on which to build: the FHWA, American Meteorological Society (AMS), and the Intelligent Transportation Society of America partnering efforts; the ongoing FHWA outreach efforts as they implement their Corporate Master Plan for Research and Deployment of Technology and Innovation; and the National Research Council’s Board on Atmospheric Sciences and Climate report, Where the Weather Meets the Road: A Research Agenda for Improving Road Weather Services, released in January 2004. All of these provide many ideas for a coordinated approach.

Formation of strategic partnerships is needed, such as the recently formed NOAA-FHWA collaboration and the NASA development of decision support system enhancement arrangements with other agencies.

Since 2002, several universities and agencies have worked to expand WIST R and D efforts. Such efforts must be encouraged by providing a clear federal vision for efforts to improve WIST information for surface transportation users. Some places where this kind of R and D and coursework are being offered are the University of North Dakota’s Surface Transportation Weather Research Center, formed in 2004; Iowa State University’s Center for Transportation Research and Education; Montana State University’s Western Transportation Institute; Virginia Tech’s Transportation Institute; and the University Corporation for Atmospheric Research, to name a few.

Transition R and D to Operations

Several activities related to transitioning R and D to operations should be pursued vigorously. First, advanced development or demonstration projects should be supported to facilitate and evaluate the transition of successful research results into operations. Examples of such projects include roadway weather information systems testing and development, the winter maintenance decision support system (MDSS), the development of 511 technologies and operational concepts, and the use of the nationwide differential global positioning system data to collect precipitable water vapor measurements.

Second, R and D findings (e.g., the improved assimilation of sensor/measurement data) need to be integrated into next-generation weather research and forecasting (WRF) models to provide better guidance/advisory products and data sets, and into other programs such as the FHWA's Clarus project.

Meanwhile, many unmet WIST needs could be satisfied within five years through applied R and D of technology applications. One such area is the development of the national 511 system. The system has already received over 30 million calls since inception; it now averages just over 1.25 million calls a month. Over 20 states are involved, with 28% of the United States having access. A telling statistic comes from a 2004 Virginia survey of 511 users: 49% of those surveyed indicated they changed their travel plans due to information obtained from the Virginia 511 system. It would appear that progress is being made in reaching the surface transportation user and influencing their travel plans.

Education and Outreach

To facilitate continued education activities, national WIST priorities must be determined that help academic institutions determine coursework and degrees to offer in WIST-related areas. One example of such outreach taking shape is the FHWA's Turner-Fairbank Highway Research Center's collaboration with the George Mason University in Fairfax, Virginia, to educate future transportation engineers.

There also needs to be outreach to all stakeholders (including the media) on improvements in WIST services such as 511 capabilities; improved weather forecasts and warnings and how to get them; and increased access to real-time observation data, such as RWIS. Many steps have already being taken to accomplish these tasks, and there are new services and products already available. Many TV and radio stations are now providing weather forecasts focused on surface travel weather impacts, as well as possible airport delays. Additional services, such as the web-based Intellicast's "Drivecast" and The Weather Channel's "Interstate Forecast" are helping travelers more effectively plan their highway trips. There is also the beginning of efforts (e.g., by Verizon, Clear Channel) to bring increased media content to cell phones, such as TV broadcast information, which should eventually provide another method to supply surface transportation users with improved weather information access.

NOAA weather radio (NWR) broadcasts are also available in the cars of several automobile manufacturers (BMW, Mercedes, Range Rover, and Saab), which equip their cars with radios capable of receiving NWR broadcasts. Finally, where possible, with help from such partners as the AMS, the federal meteorological community needs to ensure that public versus private sector responsibilities are clearly understood.

CONCLUSION

Based on projections of U.S. population growth and the limited expansion of our highway system, the need for improved surface 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 was outlined in the 2002 WIST report. However, to continue to move forward effectively, an integrated approach is needed to improve our weather information for surface transportation 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 that vision and approach, and it will take the input and support of the surface transportation and meteorology communities to achieve success.

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Low-Volume Road Abutment Design Standards

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ABSTRACT

Although several superstructure design methodologies have been developed for low-volume road bridges by the Iowa State University Bridge Engineering Center, no standard abutment designs had been developed. Thus, there was need for an easy-to-use design methodology, generic abutment construction drawings, and other design aids for the more common substructure systems used in Iowa.

A survey of the Iowa county engineers determined that while most counties use similar types of abutments, only 17% use some type of standard abutment designs or plans. In consultation with the Project Advisory Committee, a design methodology was developed for single-span stub abutments supported on steel or timber piles for bridge spans ranging from 20 to 90 ft and roadway widths of 24 and 30 ft. Using the foundation design template provided, other roadway widths can also be designed. The backwall height was limited to between 6 and 12 ft, while both cohesive and cohesionless soil types were considered. Depending upon the combination of variables for a specific site, tiebacks may be required; the design of tiebacks is also included.

Various design aids, for example charts for determining dead and live gravity loads based on the roadway width, span length, and superstructure type, were developed for the design of the stub abutments. A foundation design template was developed in which the engineer can check a substructure design by inputting basic bridge site information. Information for estimating pile friction and end bearing for different combinations of soils and pile types published by the Iowa DOT were also included. Generic

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standard abutment plans were developed to enable engineers to detail county bridge substructures more efficiently.

In addition to briefly describing the substructure design methodology developed in this project, two example problems with different combinations of soil type, backwall height, and pile type, plus a construction drawing example, will be presented to show the versatility and applicability of the materials developed.

Key words: abutment standards—bridge abutments—low-volume bridges

INTRODUCTION

Various superstructure design methodologies have been developed by the Iowa State University Bridge Engineering Center. However, to date no standard abutment designs or design methodologies have been developed. Obviously, with a set of abutment standards and the various superstructures previously developed, a county engineer could design a complete bridge for a given site. Thus there was a need to establish an easy-to-use design methodology in addition to generating generic abutment standards for the more common systems used in Iowa counties.

OBJECTIVE AND SCOPE

The objective of this project was to develop a series of standard abutment designs, a simple design methodology, and a series of design aids for the more commonly used substructure systems. Based on the results of a survey of Iowa counties and the recommendations of the Project Advisory Committee (PAC), a simple design methodology and a series of standard abutment design aids were developed. The design aids include the following: (1) graphs for estimating dead and live load abutment reactions, (2) a summary of estimated allowable pile end and friction bearing values based on the Iowa DOT Foundation Soil Information Chart (Iowa DOT FSIC 1994), (3) a generic foundation design template (FDT), and (4) a set of generic standard abutment plans. When used correctly, these tools will assist Iowa county engineers in the design and construction of low-volume road (LVR) bridge abutments.

The assumptions incorporated in the developed design methodology and corresponding design aids are similar to those made for a stub abutment system. The applicability of the design aids are limited to span lengths ranging from 20 to 90 ft and are intended for roadway widths of 24 and 30 ft (however, abutments for other roadway widths can be designed with the FDT). Superstructure systems other than the beam-in-slab bridge (BISB), railroad flat car (RRFC), pre-cast double tee (PCDT), glued-laminated girders (glulam), prestressed concrete (PSC), quad-tee, and slab bridge systems are the only superstructure system included in the LVR bridge abutment design aids. However, the general design methodology can be applied to the design of substructures for other superstructure systems.

INPUT FROM IOWA ENGINEERS

Local engineers were actively involved in the development stage (i.e., providing information, guidelines, and recommendations to the research team) to assist in meeting the project objectives. This included information on the design of the most common abutment systems, construction practices, and county capabilities. This information was collected through a survey sent to the Iowa counties, from the recommendations of the PAC, and from personal contacts with county engineers.

Based on the collected information, it was decided that standard abutment designs should include roadway widths of 24 and 30 ft with span lengths ranging from 20 to 90 ft. It was also decided that the standard abutment designs should accommodate different superstructure types, such as the RRFC, BISB, PCDT, PSC, quad tees, glulam timber girders, and slab bridges. Additionally, since 6 to 12 ft is a common range for abutment backwall heights in Iowa, designs were limited to this range. Since most Iowa counties primarily use steel and timber piles, only these two materials were investigated for use in the abutment designs.

It was also evident that integral abutment systems used in Iowa counties are based on the standard designs available through the Iowa DOT (1987). Thus, it was decided that the focus of this research project would be non-integral or stub abutments. As shown in Figure 1, a typical Iowa county stub abutment consists of a single vertical row of either steel or timber piles. The pile cap typically consists of either steel channels connected to the piles or a cast-in-place reinforced concrete cap (not shown). Also shown in this figure is a tie back system, which will not be required in numerous abutment systems.

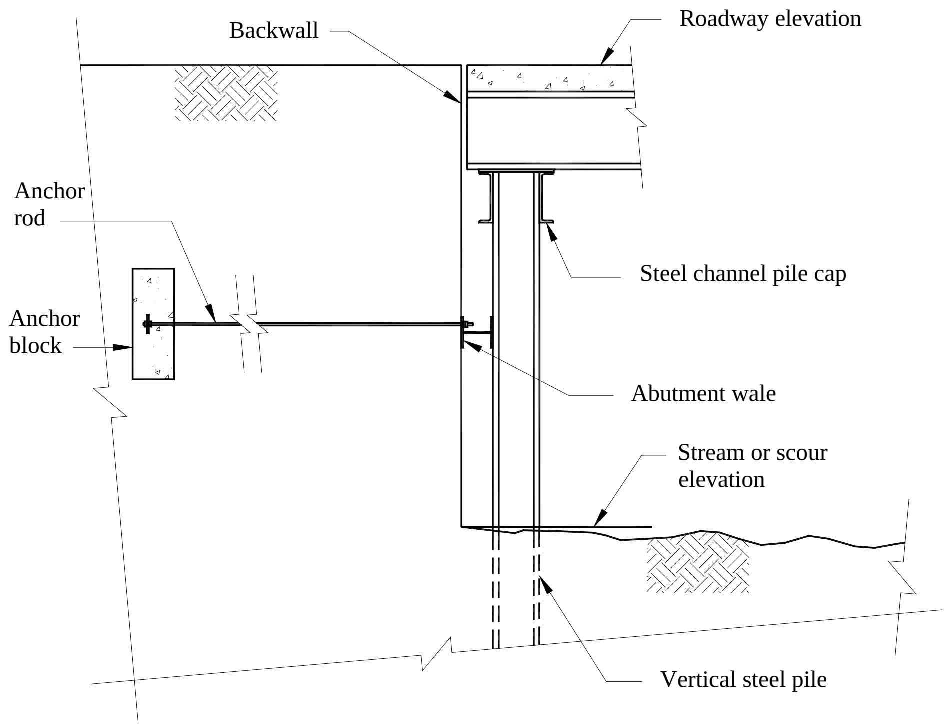

Figure 1. Typical Iowa county stub abutment with a steel channel pile cap

LATERAL LOAD ANALYSIS

Two different lateral load analysis methods were investigated in this project. The first lateral load analysis method, commonly known as the p-y method, utilizes a series of non-linear, horizontal springs to represent the soil reaction imparted on the pile when subjected to lateral loads. The springs have non-linear stiffness properties similar to the surrounding soil, which creates a statically indeterminate, non-linear system (Bowles 1996).

The second lateral load analysis method, developed by Broms (1964), considers a sufficiently long pile, fixed at a calculated depth below ground. By assuming a point of fixity, the pile can be analyzed as a cantilever structure with external loadings and appropriate boundary conditions. The calculated depth to fixity is a function of soil properties, pile width, lateral loadings and pile head boundary conditions. The pile moment and deflection can be determined using basic structural analysis techniques.

A comparison of the two lateral load analysis techniques revealed advantages for both methods. The non-linear method can be used for more complex soil conditions such as a non-homogenous soil profile and more accurate soil reaction distribution. However, advanced geotechnical software is needed to perform this analysis. In the linear method, the assumed soil pressure distributions used to determine the depth to fixity and soil reactions were developed in the 1960s. Also, the linear method does not account for the redistribution of pile loads below the assumed point of fixity. However, once the shape of the soil reaction is established, pile deflection and moment along the length of the pile above the point of fixity can easily be determined and incorporated into commonly available spreadsheet software.

Although the non-linear and linear methods use different assumptions and modeling techniques, they produce comparable maximum pile bending moments for different soil types and practical lateral load cases for LVR bridge abutments. The linear method is more conservative for stiff cohesive soils and cohesionless soils by up to 15% depending on soil type and lateral load magnitude. However, the linear method is less conservative for soft cohesive soils by approximately 3% to 20% depending on the magnitude of the lateral load. Given the assumptions used for the development of this design methodology, general similarity in results when compared to the non-linear method, and reduced computational requirements, the linear method, presented by Broms (1964), was used in this investigation for the development of LVR bridge abutment design methodology.

SUMMARY OF DESIGN METHODOLOGY

Once a lateral load analysis method was selected, a design methodology was developed for LVR bridge abutments. This included determination of substructure loads, performing of the structural analysis, foundation capacity calculations, and checking of design requirements for the pile and anchor systems. Additional miscellaneous substructure elements such as the pile cap, abutment wall, and backwall also need to be investigated; however, designing these elements was beyond the scope of this project. A graphical representation of the design methodology summarized herein is shown in Figure 2.

Design Loads

The first step in the design methodology is determining the substructure configuration such as number of piles, pile section properties, and general bridge geometry. This permits the calculation of bridge gravity and lateral loads. Lateral loadings are imparted to the bridge substructure by active and passive soil pressures on the backwall in addition to superstructure lateral forces transmitted through bridge bearings.

Conservative dead load abutment reactions for the PCDT, PSC, quad-tee, glulam, and slab bridge systems are shown in Figure 3 for a 24-ft roadway width. Similarly, charts for conservative dead load reactions for a 30-ft roadway and gravity live loads for two traffic lanes were also developed, but are not included in this paper. These estimates are based on published standard bridge designs for the respective superstructure systems and include the self-weight of both the superstructure and substructure. The maximum simple-span live load abutment reaction for one traffic lane occurs when the back axle of an American Association of State Highway and Transportation Officials (AASHTO 1996) HS20-44 design truck is placed directly over the centerline of the piles, with the front and middle axles on the bridge.

Substructure systems commonly used by Iowa counties require the piles to resist lateral loads in addition to gravity loads. The Iowa DOT Bridge Design Manual (Iowa DOT BDM 2004) provides guidance for the soil pressure distributions to be used in the design of bridge substructures. Other lateral bridge loadings, such as longitudinal wind forces, transverse wind forces, and a longitudinal braking force, are also included in the Iowa DOT BDM. The longitudinal braking force, transverse wind load on the superstructure, and transverse wind load on the bridge live load are included in the design methodology for this project. Longitudinal wind forces were investigated and found to be negligible for LVR bridge abutments and are therefore not included. Load groups cited in the Iowa DOT BDM are used to determine maximum loading effects for various combinations of gravity and lateral loadings.

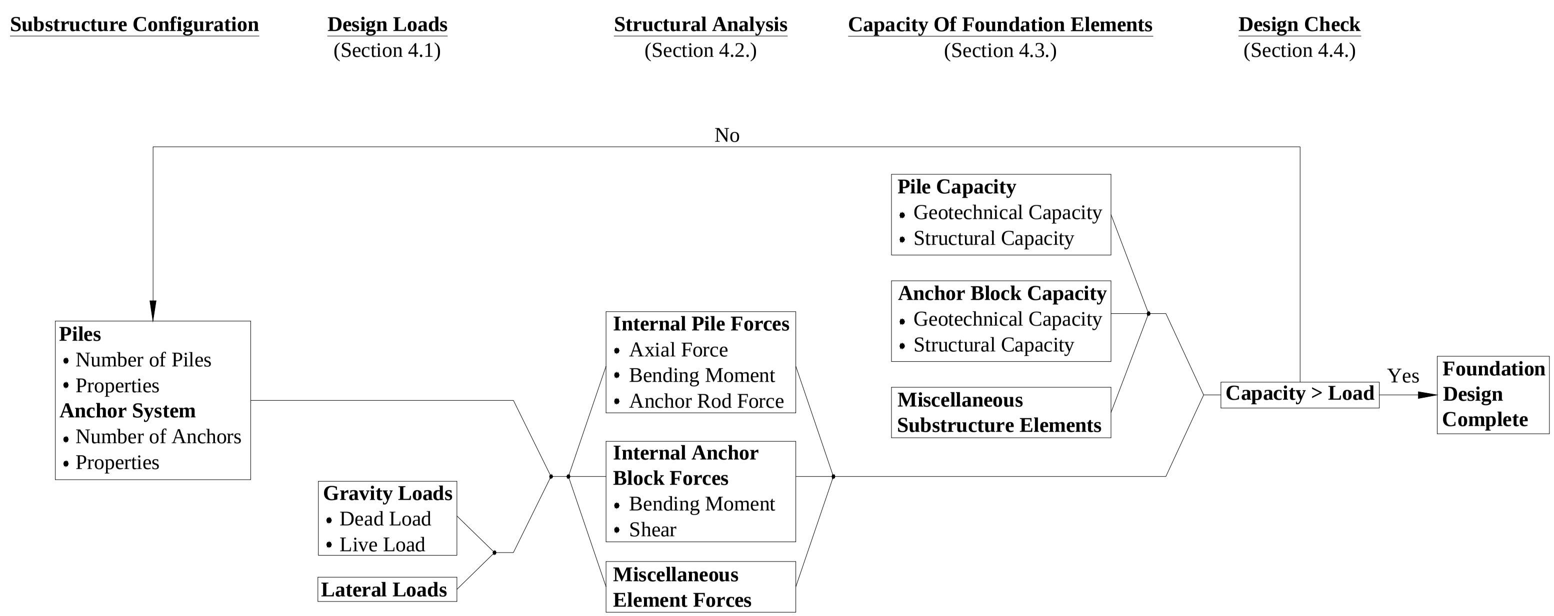

Figure 2. Graphical representation of the design methodology for a LVR bridge abutment

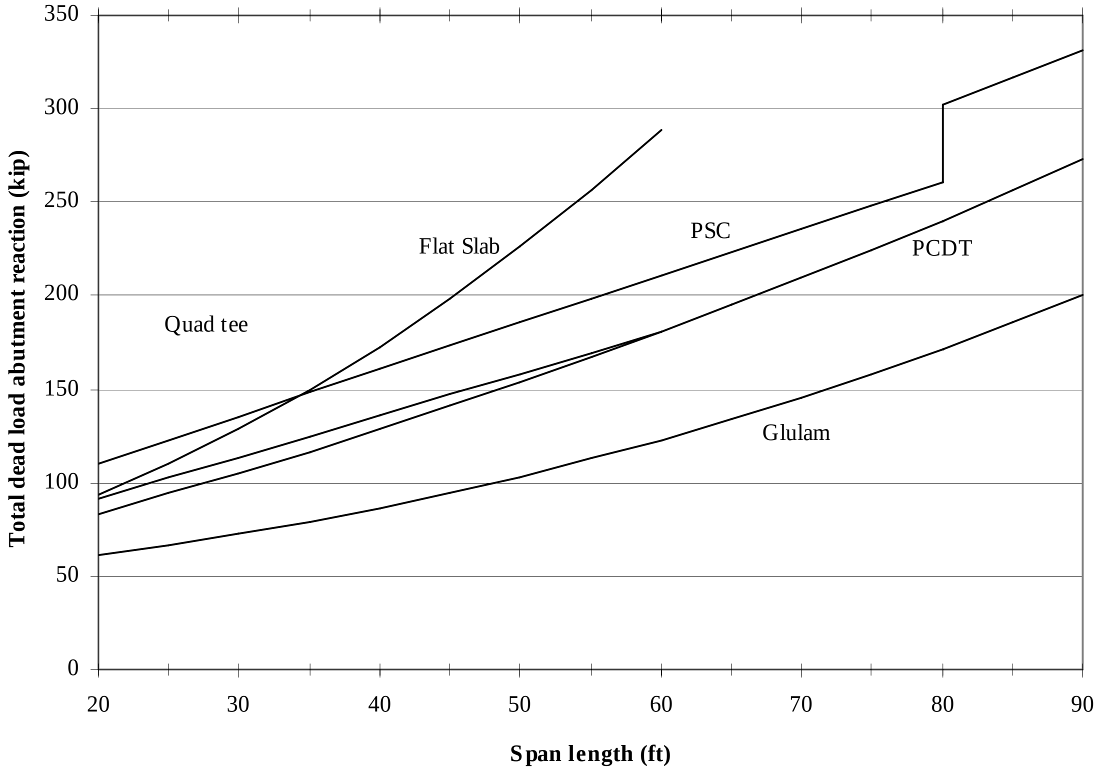

Figure 3. Estimated dead load abutment reactions for a 24-ft roadway width

Structural Analysis

Once the substructure loads have been determined, the structural analysis of the foundation system can be performed to determine internal pile forces. This includes pile axial force and bending moment, anchor rod axial force, and internal anchor block shears and bending moments.

The total abutment reaction, which is the sum of dead and live load abutment reactions, is used to determine individual axial pile forces. The axial pile loads (i.e., the load each pile much resist) are a function of the total number of piles and their spacing plus the superstructure bearing points. Different combinations of pile and superstructure bearings point configurations will produce various maximum axial pile forces within a given pile group. Therefore, a nominal axial pile factor was developed for all superstructure systems and included in this design methodology to account for the different axial forces that can develop. The design axial pile force is equal to the total abutment reaction divided by the number of piles times the nominal axial pile factor.

As previously described, the lateral load analysis technique used in this design methodology considers the pile fixed at a calculated depth below ground. After the depth of fixity is determined, the pile is analyzed as a cantilever structure. A lateral restraint system can be used to reduce lateral loading on the piles. The lateral restraint system incorporated in the design methodology was a buried reinforced concrete anchor block connected to the substructure with tension rods and a positive connection between the superstructure and substructure.

If a lateral restraint system is not utilized, the system is statically determinant and the maximum pile bending moment and deflection are easily determined using statics. Incorporation of a lateral restraint system creates a statically indeterminate system. The structural analysis methodology in this project used an iterative, consistent deformation approach, in which the displacement of the lateral restraint system is equal to the displacement of the pile at the connection point. Once the anchor rod force per pile has been determined, internal anchor block bending moments and shear forces can also be calculated.

The anchor block is analyzed as a continuous beam with simple supports that correspond to the anchor rod locations. The net soil reaction imparted on the anchor block to resist lateral substructure loads is represented by a uniformly distributed load equal to the anchor rod force per pile, multiplied by the number of piles, and divided by the total length of the anchor block.

Capacity of Foundation Elements

Guidelines specified in the Iowa DOT BDM, AASHTO, and the National Design Specification Manual for Wood Construction (NDS Manua1 2001) were all used to determine the capacities of various foundation elements. For this design methodology, foundation piles are classified as end bearing piles, friction bearing piles, or combined friction and end bearing piles. The Iowa DOT FSIC provides estimated end bearing and friction bearing values for various pile types, sizes, and foundation materials and soil types. The Iowa DOT BDM states that piles are to be designed using allowable stress design methods. All equations used for determining the design capacity of steel piles are from Part C (Service Load Design Method) of AASHTO, Section 10. Piles for typical Iowa county LVR bridge abutments are required to resist both axial and bending forces. Therefore, interaction equations for steel piles subjected to combined loading are used.

The design capacity of a timber pile is determined using the guidelines specified by AASHTO and the NDS Manual. The timber material properties vary significantly with the species type, member size and shape, loading conditions, and surrounding environmental conditions. Therefore, timber modification factors taken from AASHTO Section 13 are used to account for these variables. As recommended by AASHTO, the interaction equation defined by the NDS Manual is used to verify the structural adequacy of timber piles subjected to combined axial and bending loads.

The structural capacity of the anchor system and passive resistance of the surrounding soil must also be determined. The lateral capacity of the anchor system is related to the mobilized soil pressure that acts on the vertical faces of the anchor block. The magnitude of the soil pressure is a function of surrounding soil properties and the depth of the anchor block with respect to the roadway surface (Bowles 1996). Once the lateral capacity of the anchor system has been calculated, the structural capacity of the anchor block must be determined using reinforced concrete design practices as described in Section 8 of AASHTO.

Design Requirements

Once the foundation systems' capacities have been determined, the foundation system's adequacy must be verified. In general, this consists of verifying that capacities of the individual elements are greater than the effects of the applied loads. For design bearing requirements in general, the capacity must be greater than the axial pile load. Due to the presence of combined bending and axial loads, the structural capacity of a pile cannot be determined directly. Rather, interaction requirements are used to compare ratios of applied to allowable stresses due to combined bending and axial loads. Other design requirements include pile deflection, axial piles stress, anchor rod stress, maximum pile length (timber piles only), etc.

The capacity of the anchor system must also be verified. The maximum lateral capacity of soil surrounding the anchor block (per pile) must be greater than the required anchor force per pile. To satisfy structural design requirements, the internal anchor block shear and bending forces must be less than the structural capacity of the anchor block determined from AASHTO reinforced concrete design guidelines.

DESIGN AIDS

In addition to the development of a design methodology, several design aids were also created, including (1) graphs for estimating dead and live load abutment reactions, (2) estimated pile end bearing and friction bearing values, (3) a FDT, and (4) a set of generic standard abutment plans.

Foundation Design Template

The FDT is an Excel spreadsheet that is used to verify the design of a foundation system. At most for a given site, the engineer will need two worksheets. These include pile design and anchor design worksheets (PDW and ADW, respectively). Use of the ADW may not be necessary depending on the bridge site. The engineer has the option to use a unique PDW for each combination of pile type (steel or timber) and soil type (cohesive or cohesionless). In the case where a subsurface bridge site investigation reveals a non-uniform soil profile consisting of both cohesive and cohesionless soils, properties of the upper-level soil should be used to determine which PDW should be used. Examples of the PDW are presented in Figures 4 and 5, and the ADW can be seen in Figure 6.

General Bridge Input	1	Span length	40.00	ft		
	2	Roadway width	24.00	ft		
	3	Location of exterior pile relative to the edge of the roadway	0.92	ft		
		Maximum number of piles	9	piles on	2.77	ft centers
		Minimum number of piles	4	piles on	7.39	ft centers
	4	Number of piles	7			
	5	Backwall height	6.00	ft		
	6	Estimated scour depth	2.00	ft		
	7	Superstructure system	PCDT			
		Estimated dead load abutment reaction	128.6	kip per abutment (default value)		
	8	Dead load abutment reaction for this analysis	128.6	kip per abutment		
		Estimated live load abutment reaction	110.0	kip per abutment (default value)		
	9	Live load abutment reaction for this analysis	110.0	kip per abutment		
Foundation Material Input	10	Soil SPT blow count (N)	20			
		Correlated soil friction angle (□)	33.3	degrees		
	11	Soil friction angle for this analysis	33.3	degrees		
	12	Estimated friction bearing value for depths less than 30 ft	0.7	tons per ft		
	13	Estimated friction bearing value for depths greater than 30 ft	0.7	tons per ft		
Pile Input	14	Timber species	southern pine			
	15	Tabulated timber bending stress	1,750	psi		
	16	Tabulated timber compressive stress	1,100	psi		
	17	Tabulated timber modulus of elasticity	1,600,000	psi		
	18	Pile butt diameter	13.0	in.		
	19	Pile tip diameter	10.0	in.		
Lateral Restraint Input	20	Superstructure bearing elevation	3.58	ft		
	21	Type of lateral restraint system	buried concrete anchor block			
	22	Anchor rod steel yield stress	60	ksi		
	23	Total number of anchor rods per abutment	5	per abutment		
	24	Anchor rod diameter	0.75	in.		
	25	Height of anchor block	3.00	ft		
	26	Bottom elevation of anchor block	1.08	ft		
		Anchor block lateral capacity	9.7	kip per pile		
		Computed anchor force per pile	7.5	kip per pile		
		Minimum anchor rod length	13.47	ft		
	27	Anchor rod length	15.00	ft		

Figure 4. Input section of PDW

Design Checks	1	Axial pile load	$\boxed{P \le P_{ALLOWABLE}}$	48.0	kip	OK
	2	Pile length	$\boxed{\text{Length} \le 55\text{ ft}}$	37	ft	OK
	3	Pile bearing capacity	$\boxed{\text{Axial Pile Load} \le \text{Capacity}}$	sufficient if pile is embedded at least		34 ft
	4	Interaction equation validation	$\boxed{\frac{1}{(1 - f_C / F'_e)} > 1.0}$	1.04		OK
	5	Combined loading interaction requirement $\boxed{\left(\frac{f_C}{F'_C}\right)^2 + \frac{f_{bx}}{F'_b\left(1 - \frac{f_C}{F'_{ex}}\right)} + \frac{f_{by}}{F'_b\left(1 - \frac{f_C}{F'_{ey}} - \left(\frac{f_{bx}}{F'_{bE}}\right)^2\right)} \le 1.0}$		0.75		OK
	6	Buried anchor block location	$\boxed{\text{Anchor rod length} \ge \text{minimum}}$	15.00	ksi	OK
	7	Anchor rod stress	$\boxed{\sigma \le 0.55\ F_Y}$	23.9	ksi	OK
	8	Anchor block capacity	$\boxed{\text{Total Anchor Force} \le \text{Capacity}}$	9.7	kip per pile	OK
	9	Maximum displacement	$\boxed{\delta_{MAX} \le 1.5\text{ in.}}$	0.37	in.	OK

Foundation Summary	1	Roadway width		24.00	ft
	2	Span length		40.00	ft
	3	Distance between superstructure bearings and roadway grade		2.42	ft
	4	Backwall height		6.00	ft
	5	Dead load abutment reaction		128.6	kip per abutment
	6	Live load abutment reaction		110.0	kip per abutment
	7	Number of piles		7	
	8	Total axial pile load		24.0	tons
	9	Pile spacing		3.69	ft
	10	Pile size			
			Butt diameter	13.0	in.
			Tip diameter	10.0	in.
	11	Pile material properties			
			Timber species	southern pine	
			Tabulated timber compressive stress	1,100	psi
			Tabulated timber bending stress	1,750	psi
			Tabulated timber modulus of elasticity	1,600,000	psi
	12	Minimum total pile length		37	ft

Figure 5. The Design checks and foundation summary sections of the PDW

Input Information	1	Anchor block length	27.00	ft
	2	Distance from end of anchor block to exterior anchor rod	1.50	ft
	3	Concrete compressive strength	3.0	ksi
	4	Yield strength of reinforcing steel	60	ksi
		Tension steel area required	0.28	in^2
	5	Number of reinforcing bars on one vertical anchor block face	3	bars
	6	Tension steel bar size	4	#
		Minimum tension steel area	0.60	in^2
		Are stirrups required?	Yes	
	7	Shear stirrup bar size number	3	#
	8	Number of stirrup legs per section	2	
		Maximum stirrup spacing	4.69	in.
	9	Stirrup spacing for this analysis	4.50	in.

Design Checks	1	Design flexural capacity	$\boxed{M_U < \phi M_N}$	24.78	ft-kips	OK	{AASHTO 8.16.3.2}
	2	Reinforcement ratio	$\boxed{\rho < 0.75\rho_b}$	0.0018		OK	{AASHTO 8.16.3.2.2}
	3	Minimum reinforcement				OK	{AASHTO 8.17}
	4	Design shear capacity	$\boxed{V_U < \phi V_N}$	54.8	kip	OK	{AASHTO 8.16.6.1.1}

Anchor System Summary	1	Number of anchor rods	5			
	2	Anchor rod steel yield stress	60	ksi		
	3	Anchor rod diameter	0.750	in.		
	4	Anchor rod length	15.00	ft		
	5	Anchor rod spacing	6.00	ft		
	6	Vertical distance between bottom of anchor block and roadway grade	4.92	ft		
	7	Anchor block length	27.00	ft		
	8	Anchor block height	3.0	ft		
	9	Anchor block width	12.0	in.		
	10	Concrete compressive strength	3.0	ksi		
	11	Details of reinforcement on one vertical anchor block face	3	# 4 bars		
	12	Details for stirrups	# 3	bars on	4.50	in. centers

Figure 6. The input, design checks, and summary sections of the ADW

In the PDW, the engineer will input basic bridge parameters such as span length, roadway width, backwall height, the number of piles, pile section and material properties, the soil standard penetration test (SPT) blow count, and lateral restraint usage. These values are used in the structural analysis of the system; several design checks required by the Iowa DOT BDM, NDS Manual, and AASHTO are completed in the PDW.

Also included in the PDW are various design checks for the given foundation system. This includes but is not limited to the allowable axial pile stress, bearing capacity, combined loading interaction equations, and anchor block capacity. Finally, the PDW provides a summary of the overall bridge geometry and foundation configuration as entered by the engineer.

The ADW is only required if a buried concrete anchor is selected in the PDW. The ADW is the same for all combinations of piles and soil types. If applicable, the ADW is used only after all design requirements have been satisfied in the PDW. Additional information, such as the reinforced concrete anchor block material properties and anchor rod details, are also required. This additional information is used to calculate internal anchor block shears and moments, determine the structural capacity, and check anchorage system design requirements.

Several computer models were developed, using structural analysis software for the previously described lateral substructure loadings, to verify internal forces and deflections computed by the FDT for the various foundation elements. These models consisted of both structurally indeterminate and determinate systems (i.e., with and without an anchor, respectively). Additionally, computer models were developed to verify the internal pile forces and deflections computed by the FDT if there was a positive connection between the superstructure and substructure.

Standard Abutment Plans

In addition to the FDT, a complete set of generic standard abutment plans were developed. The standard abutment plans can be used by Iowa county engineers to produce the necessary drawings for the more common LVR bridge abutments systems. Using various superstructures and associated standard plans previously developed by the BEC, the engineer can generate a complete set of bridge plans. Note that by modifying the bearing surface of the standard abutment systems provided, essentially any type of bridge superstructure system can be supported.

In order for the engineer to produce a finished set of abutment plans, necessary details such as bridge geometry, member size designations (i.e., W, C, and HP shapes), and material properties must be inserted in spaces provided. The FDT provides many necessary details for the standard abutment sheets in the summary sections shown in Figures 5 and 6.

The standard abutment plans are composed of three different types of sheets. The first type consists of two general sheets that will be used for all bridge abutments and that are both included in the final set of construction documents. These include a cover sheet and a general bridge plan and elevation layout sheet. The second type of sheet provides general information and instructions relating to the scope and use of the standard abutment plans and is not included in the final set of construction documents. This sheet also includes a feasibility flow chart to help the engineer determine whether the standard abutment plans and FDT are appropriate for a given bridge site. The third type of sheet consists of 16 construction sheets with different combinations of pile caps, backwall systems, anchor systems, and pile types. For example, if a bridge site requires steel H-piles with an anchor, a steel channel pile cap, and sheet pile backwall, the sheet shown in Figure 7 should be used. If both bridge abutments use the same combination substructure components, the same sheet can be used twice with different dimensions, if necessary.

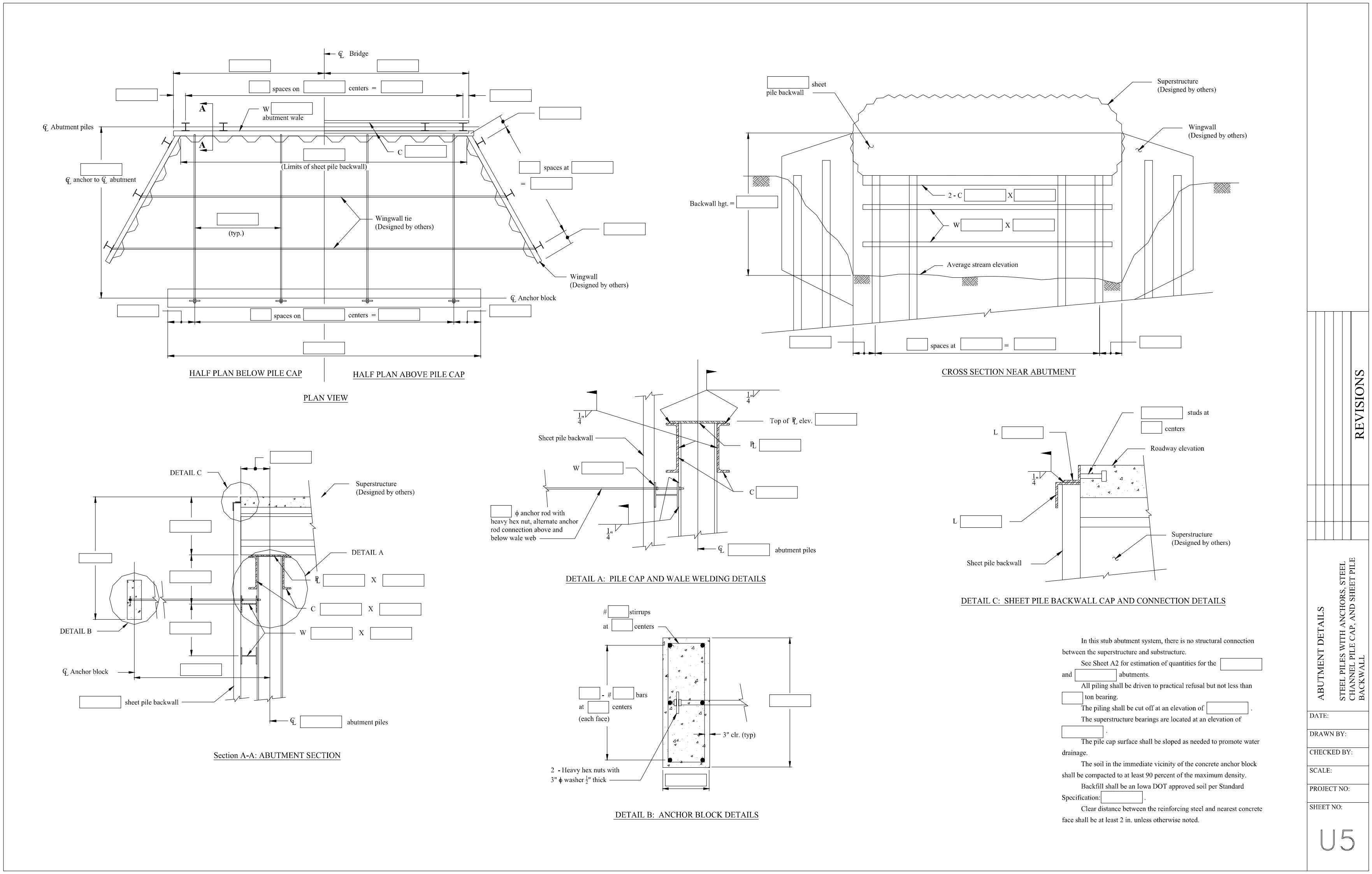

Figure 7. Standard abutment detail sheet for an anchored steel pile abutment with a steel channel pile cap and sheet pile backwall

VERIFICATION OF THE FOUDATION DESIGN TEMPLATE

Sample calculations for two examples were developed to verify and demonstrate the versatility of the FDT. These calculations are not provided here, but are available in Volume 3 of the final project report (2004). As mentioned, the FDT can be used for various roadway widths, pile types, soil types, and backwall heights. For both examples, the required input variables for the FDT are presented below. The PDW and ADW for the first example can be seen in Figures 4 through 6.

Example 1. In the first example, the FDT is used to verify the design of a timber pile abutment with a reinforced concrete anchor. The superstructure is a PCDT system with a span length and roadway width of 40 and 24 ft, respectively. In this case, bridge dead and live loads are provided by the FDT. Figure 3 can also be used to determine the gravity dead load manually. Seven timber piles with a 13-in. butt diameter are embedded in a soil best described in the Iowa DOT FSIC as gravelly sand with an average SPT blow count of 21. The backwall height and estimated depth of scour are 6 and 2 ft, respectively.

Example 2. In the second example, the FDT is used to verify design of a steel pile abutment. The superstructure is a PSC system with a span length and roadway width of 60 and 24 ft, respectively. Again, bridge dead load, provided by the FDT, can be obtained from Figure 3. Eight HP10 x 42 steel piles are embedded in soil best described in the Iowa DOT FSIC as a firm, glacial clay with an average SPT blow count of 11. The backwall height and estimated depth of scour, as in Example 1, are 6 and 2 ft, respectively.

SUMMARY

This research project consisted of the collection of LVR bridge abutment information, development of an abutment design methodology, and creation of design aids for Iowa county engineers, municipal engineers, etc. Information was primarily gathered by conducting a survey of the Iowa county engineers and through feedback of the PAC. The survey focused on the capabilities and practices of Iowa counties and the identification of common construction methods and trends. The PAC, composed Iowa county engineers and a representative from the Iowa DOT Office of Bridges and Structures, provided information about the scope of this project. This included roadway and span length limitations, common substructure configurations, and superstructures to be accommodated by the standard abutment designs. Additionally, members of the PAC suggested creating the flexible and easy-to-use design software.

Two different lateral load analysis methodologies were investigated before developing the foundation design methodology. This included a linear and a non-linear method. It was found that each method has certain advantages, such as the ability to model complex soil conditions and profiles, accurately represent actual interactions between the pile and surrounding soil, and ease of incorporating the analysis method into a complete design methodology. Based on relative simplicity and correlation of calculated maximum pile moments, it was decided that the linear analysis procedure presented by Broms (1964) would be most suitable for this project. The structural analysis procedure for the piles, both with and without lateral restraints, was developed using recommendations of the Iowa DOT BDM, AASTHO, and NDS Manual for steel and timber piles.

Finally, design aids that incorporate the design methodology were developed. These include gravity live and dead load charts for various span lengths and superstructure systems and the FDT and generic standard abutment plans. The FDT is used to verify the adequacy of a foundation system for a particular bridge site. The engineer inputs basic bridge and site data, and this information is used to determine the capacity of the foundation elements and to perform required design checks. The generic standard abutment plans include different standard sheets for each combination of pile type (steel or timber), anchor usage (with or without), pile cap (steel channel or concrete pile cap), and a backwall system (timber planks or vertically driven sheet piles). The standard abutment sheets can be used by engineers to produce necessary drawings for the more common LVR bridge abutments systems.

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Field Testing of an FRP Temporary Bypass Bridge

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ABSTRACT

Composite materials have progressively made their way into many aspects of engineering applications, from reinforcing new and existing structures to repair of damaged sections and protection of critical members, etc. The possibilities are endless. The increased use of composite materials is in large part due to the creation of the Federal Highway Administration's (FHWA) Innovative Bridge Research and Construction (IBRC) program, which promotes and reinforces the use of innovative materials, construction techniques, and structures in general. This work is part of the IBRC program and was initiated by the Iowa DOT to evaluate the applicability of using a fiber reinforced polymer (FRP) composite bridge for use as a temporary bridge crossing. The entire 39-ft x 27-ft composite structure is composed of a foam core wrapped with layers of FRP and infused with resin. The structure comes in two sections, complete with curb and guardrail, which constitutes the entire superstructure of the bridge, and will replace the DOT's current steel temporary bridge. The scope of this work involves laboratory and field testing of the structure for validation of design assumptions and to obtain a full understanding of the benefits of using this type of structure for both temporary and permanent bridge applications. Results and observations obtained from this work will provide a basis for the development of design standards and details for future use of composite bridge structures.

Key words: bridge—composites—FRP

PROBLEM STATEMENT

For years the Iowa DOT has used small, mobile steel bridges for use as temporary structures for various applications. These bridge systems include sections composed of steel girders with a steel slat flooring system and steel guardrail, as illustrated in Figure 1. In general, these structures have served their purpose well. However, the heavy, cumbersome sections have worn down, deteriorated, and become outdated over the years and are in need of replacement. Therefore, the Iowa DOT, through support of the Federal Highway Administration's (FHWA) Innovative Bridge Research and Construction (IBRC) program, opted to investigate the applicability of a composite bridge as a new temporary bridge system. The replacement system, a fiber reinforced polymer (FRP) composite bridge, is lighter, potentially easier to transport, and the few components that may deteriorate are readily replaceable (see Figure 2). This new composite structure consists of two sections, each with a guardrail attached to one side, built-in lifting lugs, replaceable wearing surface, and, once the two panels are joined, a suitable two-lane structure.

Figure 1. Current steel bypass bridge used by the Iowa DOT

Figure 2. Proposed bypass bridge, w/o guardrail or other hardware, to be used by the Iowa DOT

RESEARCH OBJECTIVES

The objectives of this work are threefold: (1) to validate through laboratory and field tests the design assumptions and structural adequacy of this structure for the designated purposes; (2) to evaluate the load distribution characteristics of the bridge components and system as a whole; (3) through physical load testing determine both the short- and long-term durability of the structure. In addition, from the results this research will provide information regarding the viability of this type of structure for this and other possible applications.

The scope of work includes a review of past and current tests performed on similar structures, as well as any other research regarding composite structures. Once substantial background knowledge is obtained, laboratory tests will then be conducted to safely evaluate the load carrying characteristics and capabilities of the temporary composite bridge to be utilized by the Iowa DOT. Following the laboratory evaluation, field tests will be conducted with the structure in service and under service-level loads. Upon completion of all laboratory and field tests, a final report detailing the general, structural, and serviceability performance of the structure will be completed.

DESIGN AND FABRICATION

Based on the temporary bridge design guidelines provided by the Iowa DOT, a design was developed and presented to the Iowa DOT by Hardcore Composites, Inc. Final design calculations were completed by the fabricator, Hardcore Composites, and checked by engineers at the Iowa DOT and Iowa State University. Final design included two 13.5-ft x 39-ft, 10-in. deck panels measuring approximately 3 ft in thickness, connected together to provide for two lanes of traffic. See Figure 3 for design sketches of the panels and related details. Each panel is composed of seven layers, called plies, of TV3400 FRP fabric on the bottom and top, and three plies on each vertical side. These layers of FRP provide the resistance to bending stresses in the structure. The core of each panel is composed of 600 8-in. x 16-in. x 36-in. foam bottles individually wrapped with one ply of FRP, and as a whole provide the shear resistance for the structure. Once all the bottles are installed and wrapped with FRP plies, vinyl ester resin is infused into the structure through a process called vacuum-assisted resin transfer molding (VARTM). Figures 4 thru 7 illustrate various stages of fabrication of the FRP panels.

At calculated locations, eight foam bottles in each panel were replaced with bottles outfitted with lifting hardware, as shown in Figure 8. The lifting hardware consists of an anchored steel plate with a threaded hole to accept bolt-on D-rings. Connection of the two panels is accomplished by means of 1-in.–thick by 16-in.–wide steel plates running the length of the bridge on the top and bottom of the centerline joint. The top and bottom plates are then connected with threaded rod through the FRP panels (see Figure 3c). Attachment of the guardrail is completed by means of a base plate attached to each guardrail post and a plate on the bottom of the deck; again, the two plates are connected with threaded rod through the FRP panels. The guardrail system for the temporary composite bridge is composed of all-steel components, including a w-shape rail post, steel tube rails, and a curb constructed of steel angle and gusset plates. Figure 3c illustrates a cross-section view of the guardrail system. The wearing surface of the deck panels consists of a 3/8-in. layer of abrasive epoxy covering the entire deck surface, except the areas occupied by connection plates (see Figures 7 and 9).

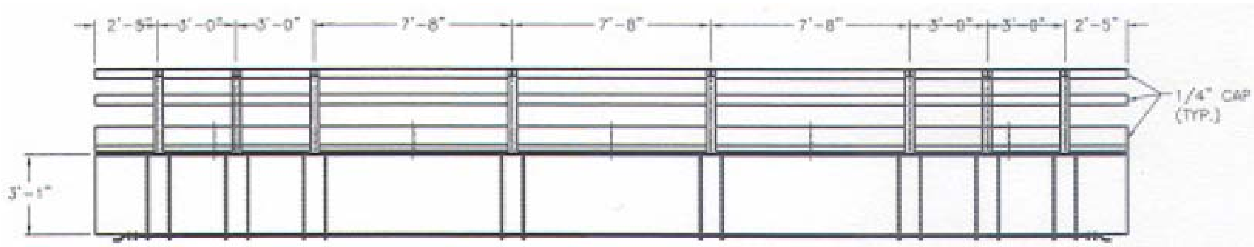

Figure 3a. Elevation view

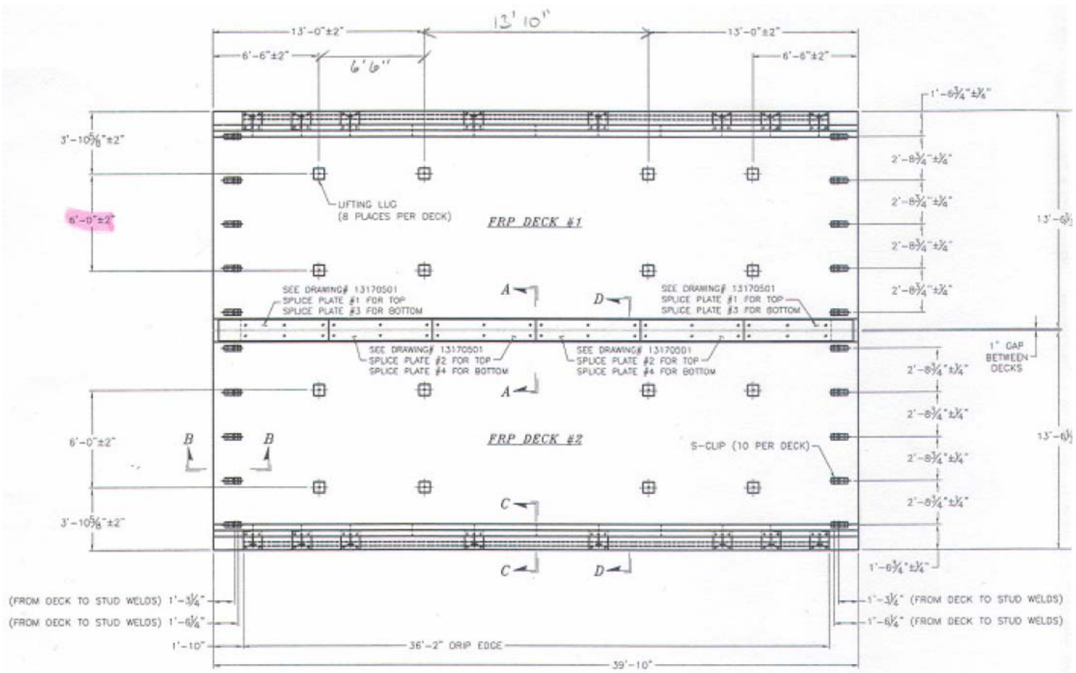

Figure 3b. Plan view

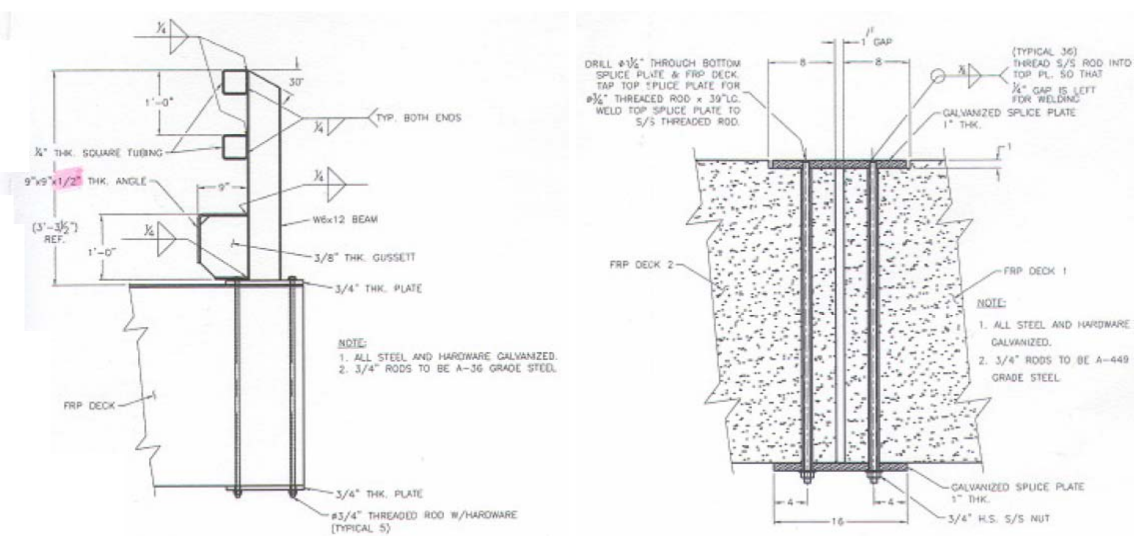

Figure 3c. Guardrail and center plate connection details

Figure 3. Iowa DOT FRP bypass bridge

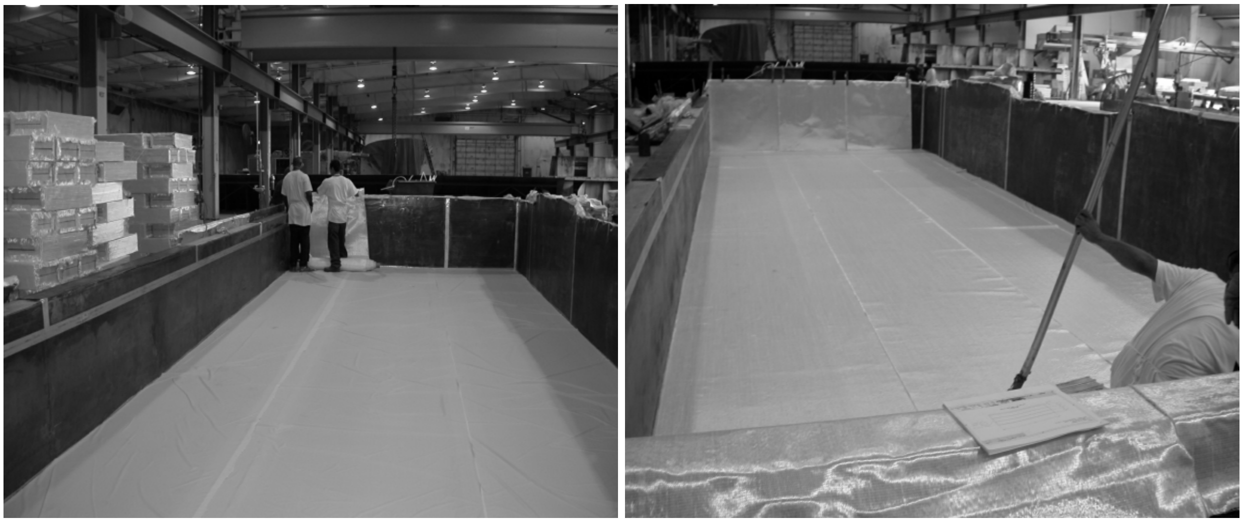

Figure 4. Installation of bottom FRP plies

Figure 5. Installation of FRP wrapped foam bottles

Figure 6. Example of VARTM process

Figure 7. Guardrail and centerline attachment locations

Figure 8. Threaded insert for installation of lifting lugs

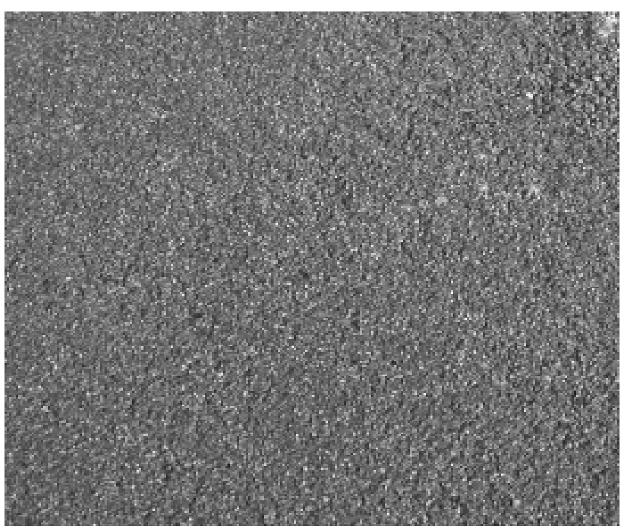

Figure 9. Epoxy wearing surface

INSTALLATION

Delivery and placement of the FRP deck panels was accomplished by means of a semi-trailer and a medium-sized truck crane. In the case of the Iowa DOT bypass bridge, the deck panels were both delivered on one flatbed semi trailer stacked one on top of the other (see Figure 10). Load testing of the deck panels was initially intended to take place in one of Iowa State University's structural engineering testing laboratories. However, due to tight tolerances between the width of the deck panels and the width of the overhead door, testing was moved to an outdoor area on Iowa DOT property.

Figure 10a. Side view

Figure 10b. Rear view

Figure 10. Delivery of Iowa DOT composite bridge

Two 30-ft–long w-shapes were placed on the ground and braced for use as temporary abutments, and a 65-ton hydraulic truck crane was rented for unloading and setting the deck panels (see Figures 11 and 12). Each of the deck panels, weighing approximately 17,000 lbs each, was unloaded and placed on the temporary abutments with 10 in. of bearing on neoprene pads at each abutment and a 1-in. gap between the deck panels, as specified in the plans. Once the panels were in the proper location, a steel fabricator was brought in to take measurements for the center connection plates and the guardrail attachment plates. Installation of the center connection plates, guardrails, and s-clips used to attach the panels to the abutments will occur immediately after fabrication of the various custom pieces and testing of an individual panel.

Figure 11. Unloading of individual deck panels

Figure 12. Temporary abutments for Iowa DOT composite bridge

RESEARCH METHODOLOGY

In an attempt to validate the design assumptions/calculations and to obtain as much information about the load distribution and serviceability characteristics of this structure, controlled outdoor laboratory tests will be conducted, in addition to a full-scale load tests, once the structure is in service. For the outdoor laboratory tests, the structure will be loaded from below using individual point loads reacting against the self-weight of the structure. These point loads of predetermined magnitude will be positioned at strategic locations such that, through the principal of superposition, stresses, strains, and deflections analogous to the design truck may be obtained. For completeness, tests will be conducted on one individual panel independently before being connected to the adjacent panel; similar tests will then be repeated on the two-panel system after being connected. Once in service, the structure will then be field tested with a fully-loaded truck to re-evaluate both the design assumptions and the results obtained from the laboratory style testing. The instrumentation layout for the in-service field load tests will duplicate that used for the laboratory style testing completed previously.

Instrumentation will include string potentiometer deflection transducers and strain gages from Bridge Diagnostics, Inc. (BDI). Displacement transducers will be installed underneath the panels for the measurement of global deflection of the individual panel and the two-panel system. In addition, deflections measured transversely across an individual panel and the two-panel system will provide an indication of the level of transverse load distribution of the FRP composite bridge. Strain gages will be strategically placed in areas and arrangements such that the following structural behaviors may be assessed: (1) validation of the transverse load distribution behavior, both from panel to panel and from bottle to bottle in an individual panel; (2) longitudinal load distribution; (3) strain distribution; (4) strain developed in the center steel connection plate; (5) type of behavior (slab vs. girder); (6) validation of design moments; and (7) strain distribution and location of the neutral axis. The BDI strain gages on the bottom of the deck will be attached directly to the structure with epoxy and will require minor preparation work. However, in order to obtain accurate strain measurements on the top of the deck, small areas of the wearing surface will be removed to allow for proper attachment of the strain gages to the panels. In addition, strain gages will be applied on the exterior side of the panels and the steel connection plate on both the top and bottom of the bridge to obtain as much information as possible about the structure. See Figure 13 for tentative instrumentation plans for testing of the Iowa DOT temporary bypass bridge.

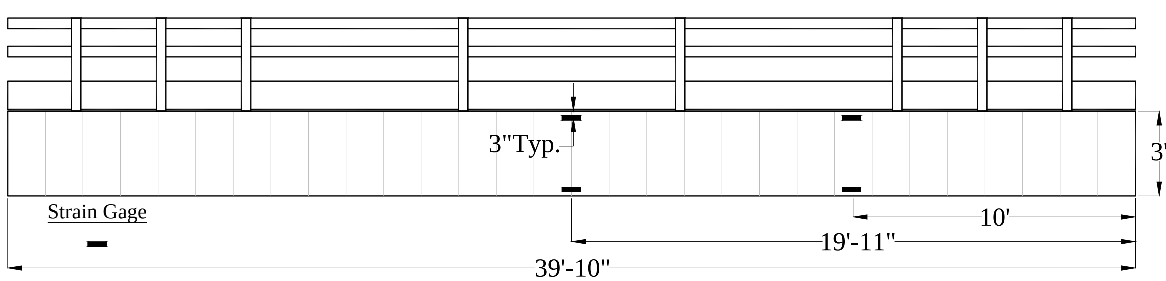

Figure 13a. Location of strain gages on side of panels

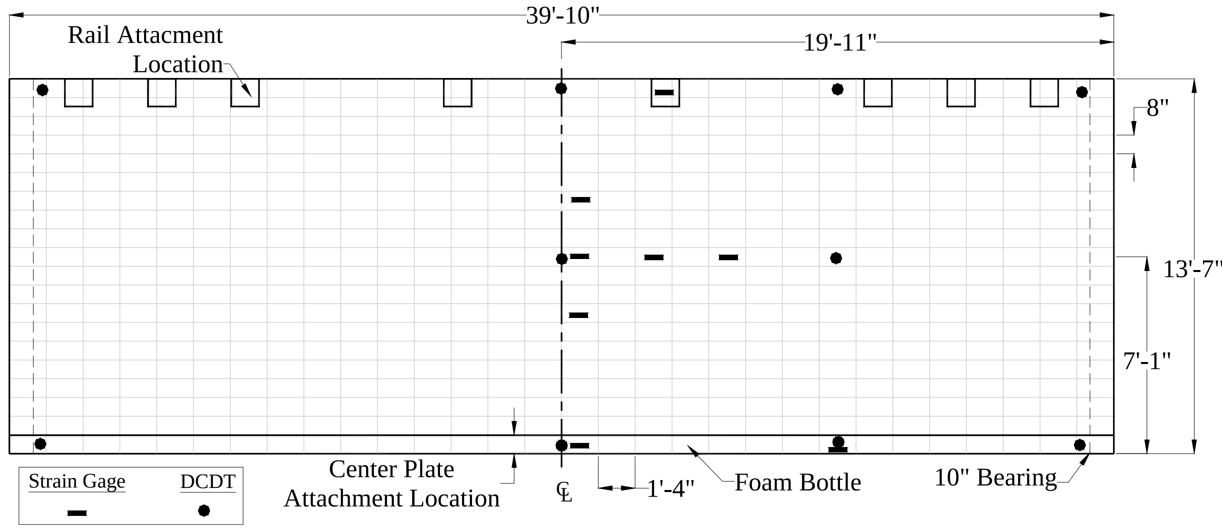

Figure 13b. Plan view of instrumentation layout for test of individual panel

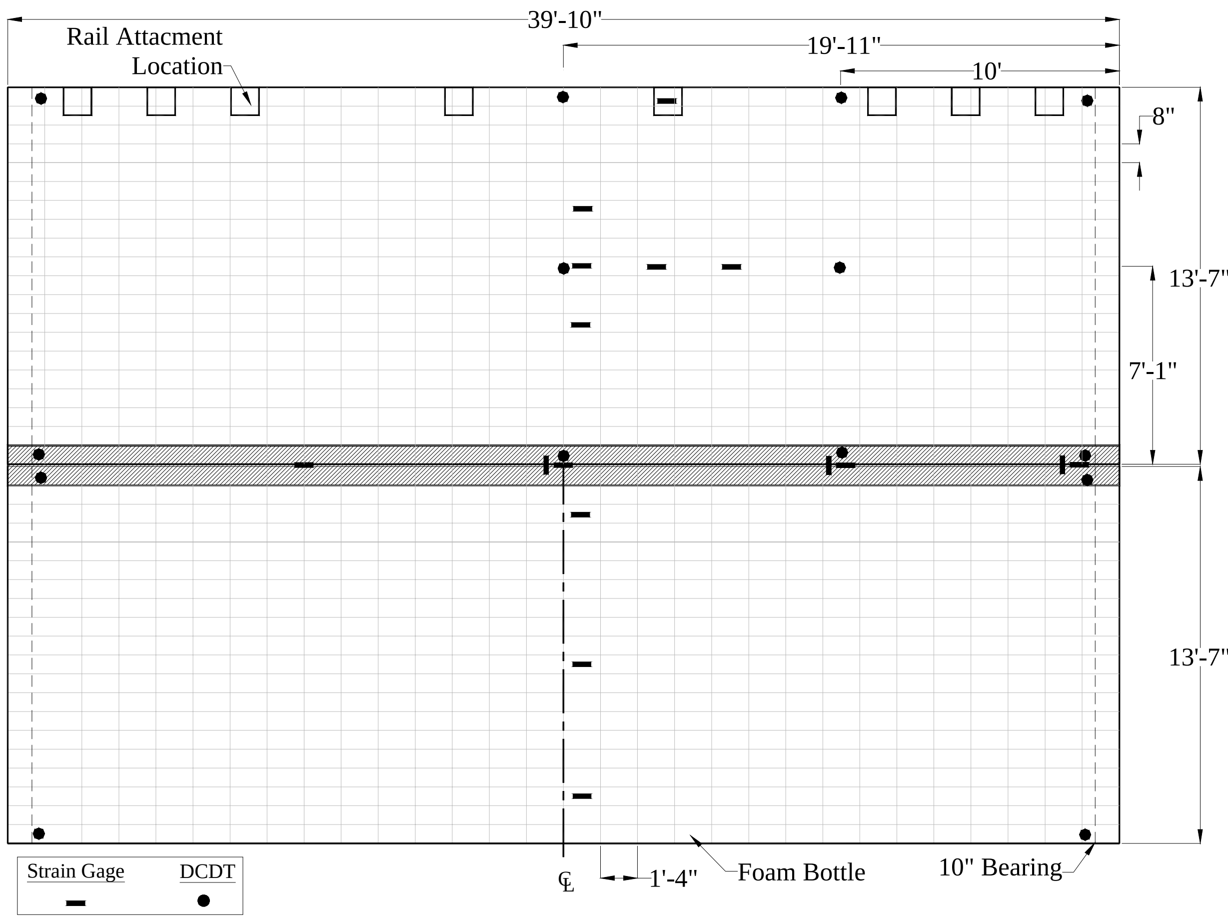

Figure 13c. Plan view of instrumentation layout for test of complete bridge system

Figure 13. Tentative instrumentation plan for testing of the Iowa DOT temporary bypass bridge

ANTICIPATED RESULTS

The test results obtained from both the outside laboratory tests and the field load testing of the bridge will be used to develop a better understanding of the structural characteristics of this type of bridge, such as load distribution, stress/strain distribution, and deflection behavior. In addition, over the life of the project, condition evaluations will be conducted to assess the durability of the structure. The combination of all the findings into a comprehensive report detailing the tips for optimizing installation procedures, expected performance of similar structures, and modes of deterioration to look for, if any, will provide a useful tool for those looking to invest in a comparable composite deck bridge. Furthermore, information obtained from this and other tests may be used to develop a rating system for this structure for the long-term use of the structure, as well as for future use and development of these structures in general.

SUMMARY

With the help of the FHWA's IBRC program, the Iowa DOT is experimenting with using a composite bridge as a temporary structure for use on various projects. The benefits of using this type of bridge include reduced weight, the entire bridge breaks down and is transportable on a single tractor-trailer semi, requires minimal setup time, and is highly resistant to deterioration. This type of technology is not new, but the actual structural behavior and response of these types of bridges is still undetermined and uncertain. The full-scale laboratory and field tests included in this work, along with other research, will both provide useful knowledge into the structural performance of composite bridges and will lead to the development of design standards, code references, and a general understanding of the uses of composite structures.

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MnROAD Portland Cement Concrete Lessons Learned

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ABSTRACT

The Minnesota Department of Transportation (Mn/DOT) constructed the Minnesota Road Research Project (MnROAD) between 1990 and 1994. The MnROAD site is located 40 miles northwest of Minneapolis/St. Paul and is an extensive pavement research facility consisting of two separate roadway segments containing 50 500-foot–long test cells. The 3.5-mile mainline test roadway, containing 31 test cells, is part of westbound interstate 94 and carries an average of 24,000 vehicles daily (14% trucks). Parallel and adjacent to the mainline is a low-volume roadway that is a 2.5-mile closed loop containing the remaining 19 test cells. LVR traffic is restricted to a MnROAD-operated 18-wheel, 5-axle tractor/trailer with two different loading configurations of 102 kips and 80 kips. Subgrade, aggregate base, and surface materials, as well as geometric design methods, vary from cell to cell. Daily information is gathered via a computerized data collection system that monitors over 4,500 mechanical and environmental sensors. All data collected is entered into the MnROAD database available to Mn/DOT and other researchers free of charge. More information can be obtained from the MnROAD web page: http://mnroad.dot.state.mn.us/research/mnresearch.asp.

The portland cement concrete (PCC) test cells at the MnROAD project have now been subject to over 10 years of traffic and environmental exposure. Despite the overall good appearance of the PCC test cells, much knowledge has been gained. This presentation will describe some of the lessons learned and give ideas about what may be studied in the future. Highlights of the presentation include the following:

- A review of the variables incorporated into the MnROAD concrete test cells
- The current surface conditions of the PCC test cells
- A review of the initial research objectives for the MnROAD PCC test cells
- A summary of significant MnROAD PCC research studies completed over the past 10 years (and how they related to the initial research objectives)
- A description of the newer PCC cells at MnROAD
- Lessons learned over the past 10 years of PCC research at MnROAD
- Future PCC test cells

Note: Contact the presenter above for more information on this topic.

Key words: MnROAD—pavement research facility—PCC test cells

MnROAD Hot Mix Asphalt Observations

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ABSTRACT

The Minnesota Department of Transportation (Mn/DOT) constructed the Minnesota Road Research Project (MnROAD) between 1990 and 1994. The MnROAD site is located 40 miles northwest of Minneapolis/St. Paul and is an extensive pavement research facility consisting of two separate roadway segments containing 50 500-foot–long test cells. The 3.5-mile mainline test roadway, containing 31 test cells, is part of westbound interstate 94 and carries an average of 24,000 vehicles daily (14% trucks). Parallel and adjacent to the mainline is a low-volume roadway that is a 2.5-mile closed loop containing the remaining 19 test cells. LVR traffic is restricted to a MnROAD-operated 18-wheel, 5-axle tractor/trailer with two different loading configurations of 102 kips and 80 kips. Subgrade, aggregate base, and surface materials, as well as geometric design methods, vary from cell to cell. Daily information is gathered via a computerized data collection system that monitors over 4,500 mechanical and environmental sensors. All data collected is entered into the MnROAD database available to Mn/DOT and other researchers free of charge. More information can be obtained from the MnROAD web page: http://mnroad.dot.state.mn.us/research/mnresearch.asp.

This presentation will review the performance of MnROAD's 14 original hot mix asphalt (HMA) test cells over the first 10 years of traffic. The presentation will concentrate on rutting, cracking (top-down and transverse), and resulting ride impacts relating to the designs developed for this initial experiment. The mainline HMA test cell design variables include binder PG grade, design method (number of blows, gyratory), HMA and base thickness, and drainage.

Note: Contact the presenter above for more information on this topic.

Key words: hot mix asphalt test cells—MnROAD—pavement research facility

Wireless, GPS-Enabled Data Processing Architecture for Highway Work Zones

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ABSTRACT

A short-range, easily deployable data communication system enables numerous highway work zone applications. The Kansas Department of Transportation is currently funding a project to develop a delay notification system in rural construction zones. The objective of the project is to estimate the expected delay until the return of the pilot car and communicate that delay to motorists waiting in the queue. Since most rural construction zones are outside existing commercially available communications, such as cellular coverage, success of the project hinges primarily on the acquisition of a low-cost, easily deployable work zone data communication system. The prototype system relies on 900-Mhz radio transceivers integrated with GPS receivers and a microprocessor. These devices mount to signs, vehicles, and relay points and are programmed according to their function in the overall system. This low-cost approach not only enables the delay notification system, but also opens the door for other value-added applications within the work-zone. The same architecture can support construction vehicle tracking, notification of blow-bys (when vehicles fail to yield to traffic control), and field construction data reporting to centralized construction management systems. This paper reviews the communication architecture developed for the delay notification project and then casts that same communication system into scenarios to support value-added applications.

Key words: global positioning system—wireless communications—work zones

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION AND BACKGROUND

The Kansas Department of Transportation (KDOT) performs routine roadway rehabilitation that requires traffic on two-lane rural highways to be narrowed to one lane. This operation involves the use of a pilot car to direct the flow of traffic. KDOT's construction policy limits driver wait time to a maximum of 15 minutes during pilot car operations. Even so, pilot car operations are a primary source of negative customer feedback for the agency. In 2002, KDOT began to investigate methods of informing the driving public of the expected wait time until the next pilot car. A preliminary study, based on surveys of the driving population conducted by KDOT and Kansas State University (KSU) in fall 2002, showed a sufficient desire from the driving public to be notified of the anticipated wait time when approaching a pilot car operation. Based on this information, KDOT sanctioned research into the pilot car notification system. Additionally, as the project was getting started in the fall of 2003, a letter was sent to the Governor of Kansas with regard to the above issue. In reply to this letter, the secretary of the Department of Transportation assured the writer that KDOT would look into the issue and address it in the best possible way.

The main objective of the subsequent research was to determine the best cost-effective method of informing drivers of delay time when approaching a pilot car operation at two-lane rural highway work zones. Several systems with similar intent have been developed as part of the intelligent transportation system (ITS) movement; however, these systems addressed high-traffic volume, high-cost, and long-term work zones (Garber and Srinivasan 1998; Dudek 1999). Rural work zones are a unique category, since such work zones have low traffic volumes, and are low cost (comparatively) and short duration. Although research on rural, pilot car-aided work zones is scant with regard to automatic traveler information systems, the work performed for urban areas has some applicability. Research conducted by the Oregon Department of Transportation revealed the following (Griffith and Lynde 2001):

- Motorists were willing to accept delays in the 5–10-minute range without expressing concerns that they would be upset or angry.
- Delays up to 15 minutes or longer were acceptable to many motorists as well, although in these cases information about length of the delay played a large role in their willingness to accept this length of time.

In 2003 and 2004, the KDOT-sponsored research effort scrutinized various alternatives to deliver wait time information to drivers in a queue waiting for a pilot car. In the summer of 2004, a system was prototyped and demonstrated to the public that made use of several mature technologies.

PILOT CAR WAIT TIME NOTIFICATION SYSTEM ARCHITECTURE

The architecture used to demonstrate the pilot car wait time notification in the fall of 2004 is illustrated in Figure 1. The pilot car was equipped with a GPS receiver, a 900-MHz serial data radio, and a laptop computer. The laptop computer received position, speed, and heading information from the GPS receiver. It ran an algorithm that determined the delay time to be displayed to the driver and then communicated that delay time via the 900-MHz radio. The sign was a standard warning approach sign equipped with an LED countdown timer display. A 900 MHz radio attached to the sign received the signals from the pilot car and displayed it accordingly.

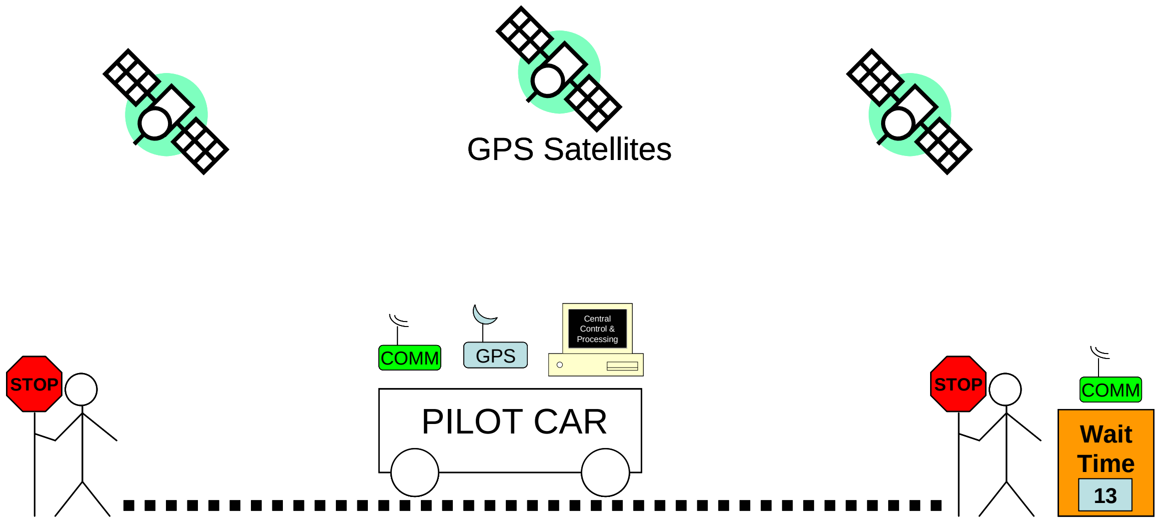

Figure 1. Initial architecture for pilot car wait time notification system, demonstrated fall 2004

A photograph of the sign system is shown in Figure 2. The demonstration took place during an overcast day along a relatively flat section of rural highway approximately four to five miles in length. The overcast conditions allowed for better visibility of the LED display. Note the large mast on the sign with an antenna mounted to the top and a radio module mounted on the mast just above the fabric. The mast was needed in the demonstration to provide connectivity with the pilot car throughout the work zone.

Figure 2. Photograph of instrumented sign during the fall 2004 demonstration

The system worked as designed during a two-hour test run in early October of 2004 at an actual work zone. The relatively flat terrain allowed the sign to receive data from the pilot car throughout the work zone. In late October, the system was demonstrated in a work zone in hillier terrain. The instrumented sign was located in a valley and quickly lost contact with the pilot car no more than a mile into the work zone. The timer algorithms in the system were not intelligent enough to estimate delay in the absence of a signal. Concerns over the weight of the sign, connectivity, and general robustness of the notification system led to a new operating concept that is schedule to be tested in 2005. The architecture for this concept is shown in Figure 3.

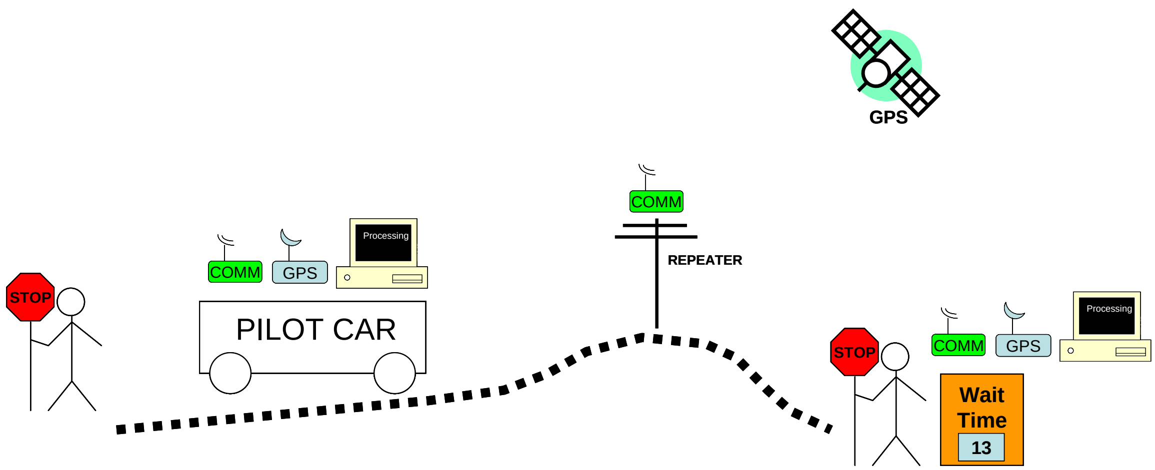

Figure 3. Target architecture for pilot car wait time notification system in 2005

Several system enhancements are included in the new architecture. The need for a communications repeater in hilly terrain was evident. It has the secondary benefit of drastically reducing the power and mass requirements on the notification sign. With a repeater nearby, a simple six- to eight-inch whip antenna can be used on the sign, reducing its height and mass and increasing its crash worthiness. The repeater can be strategically located to maximize connectivity, such as at the crest of the first hill in the direction of the work zone. The repeater can be located well away from the travel way, limiting its exposure to traffic. Power requirements of the system are dominated by the transmit cycle of the radios. If the repeaters provide most of the high-power transmit capability, the battery requirements on the signs can also be minimized.

Also notice in Figure 3 that GPS and processing capability are available at the sign as well as in the pilot car. The ultimate design will also have GPS and processing at the repeater station; however, in a limited demonstration planned for 2005, they are not needed. Other enhancements in the architecture are also targeted for 2005, but are less physically prominent. These enhancements deal with the processing algorithm for estimating delay time. In the absence of a signal from the pilot car, the sign will intelligently estimated the wait time through some type of countdown mechanism. The processing capability at the sign enables intelligent action in the absence of communication. The GPS at the sign also enables automatic configuration. If the system understands the location of all of its assets, the determination of the appropriate display time is greatly simplified. It is the combination of the communication module, location module, and processing module into a reprogrammable unit that opens the door to a wide variety of applications within the work zone.

MULTI-FUNCTION COMMUNICATION, LOCATING, AND PROCESSOR PLATFORM

Through the process of refining the delay notification system, it became evident that the core unit that provided for communications, locating, and processing was the key element in efficiently enabling and managing not only the delay notification system, but also many other work zone systems. The key to a viable delay notification system is cost. The cost includes not only the initial up-front equipment charge, but also the day-to-day cost of personnel to deploy and run the equipment. The latter is more critical in the long run. Engineering the equipment to decrease the time to deploy and operate as well as lower the qualifications needed to manage the system would provide substantial cost benefit in the long run. Unlike the suburban and urban counterparts, rural work zones are a relatively low-margin business. Contractors will resist systems, even inexpensive systems, that require extra manpower to operate. Ideally, any new system will not only provide the useful data for the traveling public, but also provide a cost advantage for the contractor. The basic communication, locating, and processing subsystem that enables the wait-time notification application can also enable other work zone applications that may reduce overall road construction costs, either through efficiency or cost avoidance.

The basic architecture of the communication, locating, and processing (CLP) subsystem is shown in Figure 4. The GPS unit provides global locating in the form of spatial coordinates of latitude, longitude, and elevation. GPS also provides a high-quality time standard as well as velocity information for mobile platforms. The processing unit is the reprogrammable brain of the CLP subsystem. It queries the GPS unit for location and processes any requests that arrive from the wireless communications. The processing section is the module that contains custom algorithms appropriate for the application. In the case of the pilot car notification system, the processing unit is programmed differently, depending on whether the unit resides on the pilot car or on the sign. In most cases, all the algorithms for a specific application can be contained within a single unit. The unit is then switched to take on the appropriate "personality," depending on its location within the system. The CLP subsystem on a sign and in the pilot car will behave differently, but share the same physical hardware architecture.

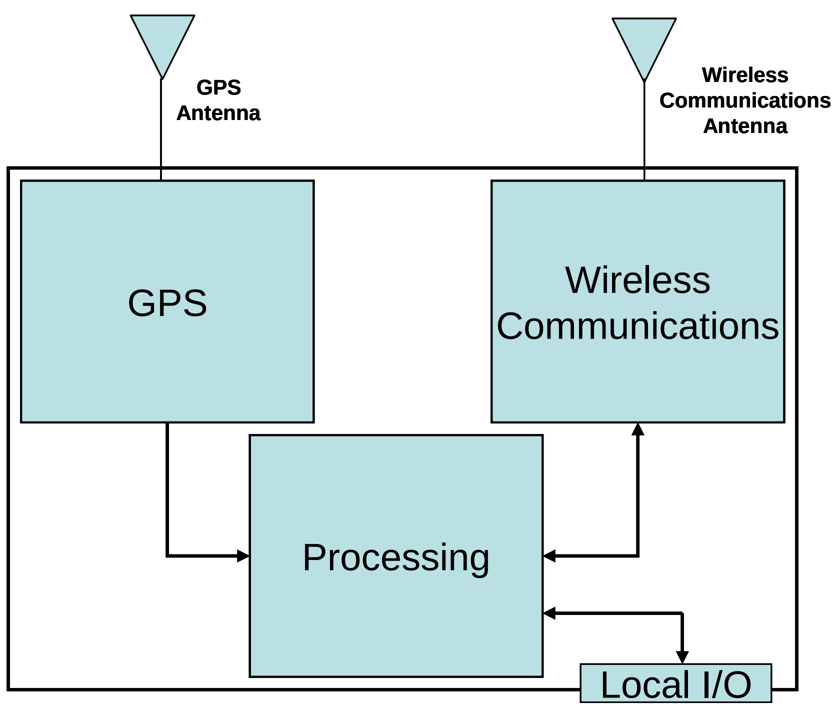

Figure 4. Subsystem architecture for communications, locating, and processing

Processing is typically accomplished with a micro-controller or PIC processor. Most of these devices are inherently capable of standard serial communications such as RS-232. A broad range of capability exists within this class of controller. On the low end are devices that simply monitor a sensor and pass the data via a serial communications channel. On the high end are devices that rival the power, flexibility, and storage capacity of desktop computers. The tradeoff in capability also comes with a tradeoff in physical dimensions, power consumption, and cost. For these reasons, it is important to size the processor appropriately for the application.

The last unit in the CLP subsystem is the wireless communications module. The wait time application utilized a 900-MHz system whose base communication standard was serial data communications. The candidates for the wireless system include both the 900-MHz and 2.4-GHz regions. The lower frequency has better signal propagation characteristics, though both are primarily line-of-sight. The lower frequency has greater tolerance for moderate foliage and for filling in between hills. The primary difference, though, is in the speed of data transmission and the resulting requirements placed on the processor. Both the 900-Mhz and 2.4-GHz bands are license-free. Although multiple protocols can be used in these bands, the 2.4-GHz band is dominated by the IEEE 802.11 standards, commonly referred to as WiFi. In this configuration, the data capacity is between 2 to 10 Mbits. WiFi is the dominant technology used to provide laptops and other computing devices with a wireless broadband internet connection in places such as airports and hotels. The 900-Mhz frequency is more typically found in voice applications and serial data applications. The 900-Mhz technology was chosen in the wait time notification system for a number of reasons. Its propagation properties were an advantage. The other modules in the system had native serial communications capabilities, so the serial nature of the 900-Mhz technology integrated well. The higher bandwidth systems also require more power as well as more processing capability, both of which were critical factors in the current application.

With the exception of the antennas for the GPS and wireless communications, the system is a self-contained unit. A common power source (either internal or external battery) services all modules. This unit is the basis for every part of the system. The sign uses it to receive location information from the pilot car, assess its own position in relation to the pilot car, and calculate and display the appropriate wait time. The LED display timer itself communicates through a serial port to the processor (local I/O). On the pilot car, the unit has no local I/O. All critical components are contained within the subsystem. The same is true if the subsystem is used for the repeater. The programming in the processor is changed appropriately, and the external power supply will need to be greater due to the increased transmit duty cycle. However, the same CLP physical subsystem remains completely intact for all three functions. A picture of the physical unit developed for the wait-time notification project is shown in Figure 5.

Several advantages exist with having the GPS unit in every module, apart from the apparent need of location information in calculating expected delay. Asset tracking becomes nearly automatic. Another module, the management module, could exist in the system. Its purpose is to keep track of all the assets. It could query all devices in the system and display them appropriately in a map of the work zone. If the CLP module is mounted on construction vehicles, these assets could be tracked as well. For the wait time notification project, note that the complete system will include multiple signs (at least one on each approach to the work zone), multiple repeaters as dictated by the terrain, and a module for the pilot car. The basic unit discussed will service all devices.

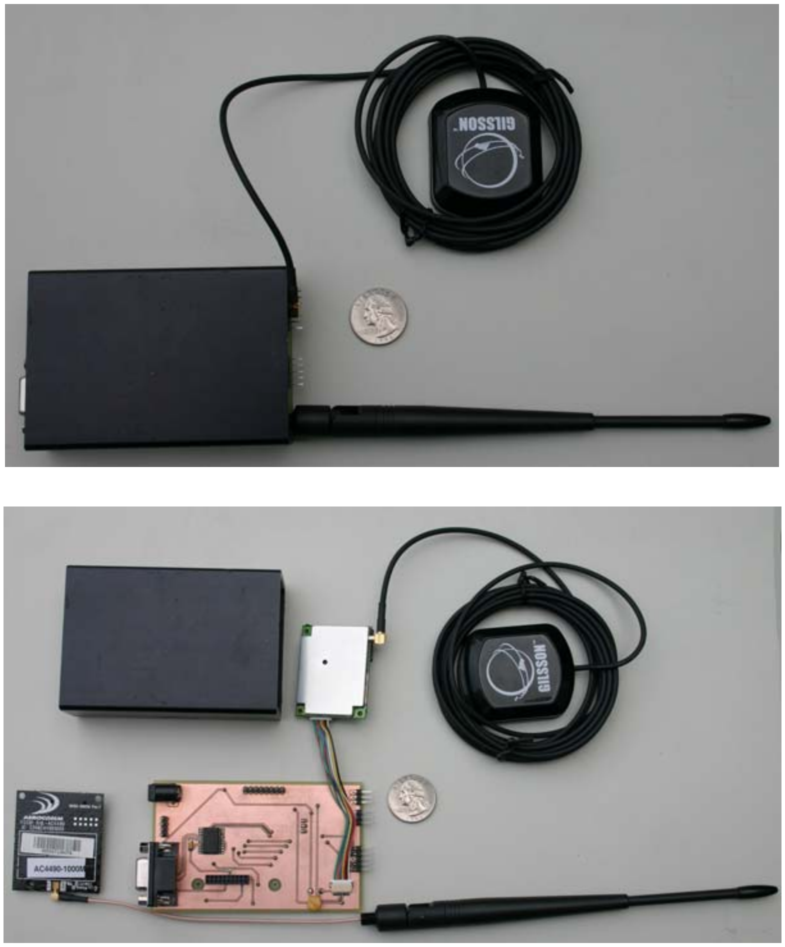

Figure 5. Fully assembled CLP subsystem built for pilot car delay notification project (top) and individual components within the unit (bottom)

ADDITIONAL WORK ZONE APPLICATIONS ENABLED BY ARCHITECTURE

Cost is the primary driver in any application. The added burden of manpower for a new public information system could quickly create undue overhead for the contractor that is passed onto the taxpayer through increased bids for construction. In addition to equipment and manpower costs, liability is also a primary driver. Any system that increases or decreases the overall safety of the system will also increase or decrease the overall liability costs proportionally. The remainder of the paper is dedicated to investigating a number of concepts in which the basic CLP module architecture could be used to enable additional work zone applications, and the impact that such applications could have on project costs.

Blow-by and Other User Action Monitoring

Work zones continue to be one of the areas of great risk both for highway workers as well as the traveling public. Many systems have been employed to alert drivers of upcoming construction zones, monitor their speed, and sound alarms signaling impending danger. Figure 6 shows a concept of a networked approach to such sensors and alarms enabled by the architecture previously discussed.

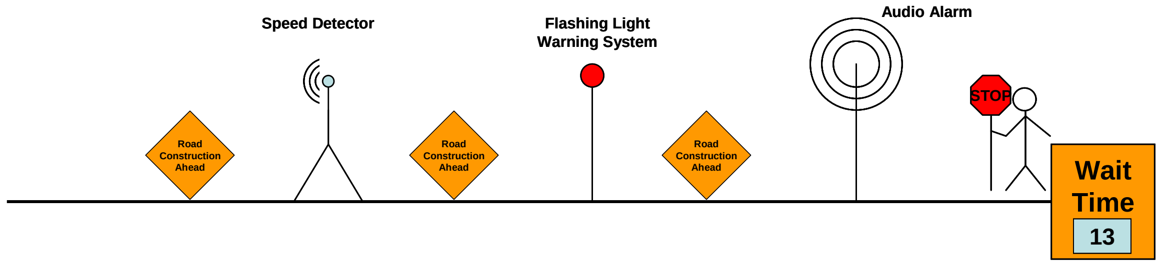

Figure 6. Inattentive driver detection and warning concept

Figure 6 depicts extra sensors and actuators on the approach to a construction zone. The speed detector, flashing light, and audio alarm each have at their base the CLP subsystem. The speed detector monitors the velocity of the approaching traffic. If the speed exceeds a certain threshold, a message is sent to the unit controlling the red flashing light. If the speed is above an even greater threshold (or another detector further into the work zone indicates that the driver has not slowed down) an additional audio alarm or other type of reactive measure may be triggered through the system. Since devices are communicating through a common wireless network, the system would be easy to deploy and configure with new sensors or actuators. Furthermore, since each device has a GPS unit, the speed detector need not be programmed with knowledge of the warning light or audio alarm. The speed detector communicates the hazard and location and the devices within the appropriate vicinity react accordingly. A central monitor can chart the location of all assets (even signs) and ensure they are deployed in agreement with governing standards.

Construction Fleet Monitoring

Knowing the exact timing of delivery can increase efficiency and safety. A CLP unit in each construction vehicle, or possibly on all construction personnel, can enhance the observation of the entire operation and thus create opportunities for increased efficiency and safety. Many vehicles have electronic interfaces for monitoring critical components, such as fuel, tire pressure, or mechanical failures. If a CLP unit is designed to interface with the vehicle, the diagnostic information becomes available centrally, allowing for better planning and deployment of resources.

Field Data Entry

Much of the construction practice is governed by standards and methods that require periodic data entry or material sampling. Direct entry of the data into electronic format on the job reduces transcription, thus reducing opportunity of error. The availability of GPS (including the exact time of data collection and location of samples) provides a method for automatic labeling. Figure 7 depicts one such scenario. The production sample (such as aggregate or asphalt) is logged electronically by an application running on a common personal data assistant (PDA). The intelligent hard hat has the CLP subsystem embedded. It allows the worker to be tracked for safety reasons, but also communicates to the PDA to provide location information to be logged with the data entry. A complete copy of the data can then be transmitted across the wireless network to a central server. Once at the server, project inspectors and owners would have instantaneous feedback concerning the quality or pace of the project.

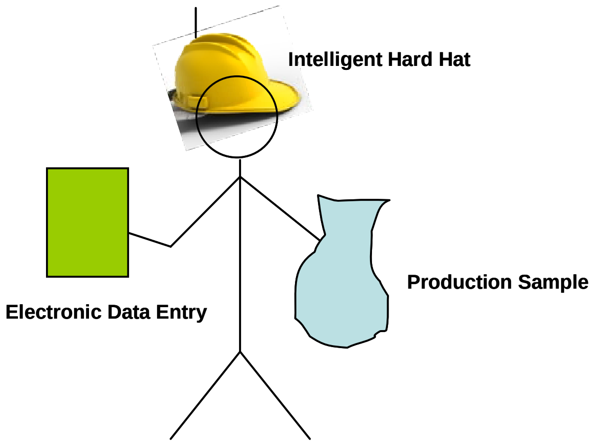

Figure 7. Field data entry concept

CONCLUSION

Wireless data communications, GPS technology, and portable processing can combine to create an opportunity to develop spatially rich applications that are easily deployable and manageable. Highway construction zones can benefit greatly by using spatially enabled applications to increase safety, deliver traveler information, and increase construction efficiency. The application currently under development at KSU for KDOT communicates the expected wait time at rural construction zones to drivers. The project utilizes a core module consisting of a GPS receiver, a 900-MHz wireless data transceiver, and a PIC processor. This module is designed to be replicated to each element in the system and forms the core of the application. The module interfaces with local devices such as the LED sign on the approach to the work area. The modules differ only by the algorithm running in the processor to perform the local task. The manpower requirements of the wait time notification system are minimized, since initialization is automated using the location data of each asset in the field. The generic framework provides a foundation to begin to automate other tasks in the work zone. Asset tracking, data collection, and inattentive driver detection are a sample of the applications that could be enabled using a similar architecture.

ACKNOWLEDGMENTS

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High-Accuracy Geometric Highway Model Derived from Multi-Track, Messy GPS Data

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ABSTRACT

GPS provides longitude, latitude, and elevation points and is already integrated into some highway data collection processes at the Kansas Department of Transportation. For instance, GPS tracks have been systematically collected on the 11,500 mile Kansas state highway system since 1997 as part of two separate data collection activities: videolog and pavement management. While not the primary purpose of the data collection activities, over 11 million GPS points have been collected to date. These individual geodetic points became the basis for a high-accuracy geometric highway model and for a high-quality base map for GIS purposes. Methods to process this data were developed and applied to the entire database of GPS points, resulting in a geometric model of the state highway system. In turn this geometric model can now be used as the basis for other applications. One of the first applications was the assessment of sub-standard stopping sight distance. The principles and mechanics of the processing methods that combine multi-year, multi-run GPS data into highly accurate spatial models are reviewed and summarized.

Key words: geometric highway model—GPS—sub-standard stopping sight distance

INTRODUCTION

The Kansas Department of Transportation (KDOT), like many state highway organizations, uses a variety of location referencing systems to specify position along a highway. These systems include stationing, county route logpoint, and many relative or descriptive means. However, inventory databases contain several million spatial coordinate triplets of longitude, latitude, and elevation collected using GPS technology, collected along the 11,500 miles of Kansas highways since 1997. An algorithm for estimating a three-dimensional spatial model of roadway geometry from the historic GPS data has been the research objective over the past few years. Although several concepts from digital signal processing, optimal estimation, and data reduction are applicable in estimating geometry from multi-run data series, the unique structure and error distribution of the historic GPS data prohibit direct application of established methods. The spatial error from successive GPS data is highly correlated. Even though the GPS error is widely published to be in the range of 1 to 5 meters, the relative accuracy of sequential GPS data is much greater, providing the opportunity to develop a high accuracy geometric model of the highway system for the entire state from which to assess geometric properties, such as stopping sight distance and passing sight distance.

This paper presents the basic principles of an algorithm to process the GPS data that is a variant of a robust least-squares technique. The algorithm takes advantage of the high correlation in GPS error in order to estimate grades and trajectories accurately. The algorithm utilizes low-order normalized piece-wise polynomials as basis functions. Both the position data and estimates of the first derivative are used in the fitting process. Robust fitting techniques are used to iteratively re-weight data points and to estimate and subtract bias errors.

The resulting spatial equations that describe the trajectory of the highway are reduced to a set of control points, that is, a sequential set of points along the route or highway spaced closely enough to capture and recreate the horizontal and vertical curvature using a spline technique. Accuracy of the model is also estimated. The accuracy metrics enable follow-on applications to determine if the model is accurate enough to obtain valid results. Accuracy metrics are calculated for absolute position error, relative position error, and error in the first derivative of the spatial model.

In order to validate the algorithm, the elevation output from the model was compared to the elevation design profile of three highway segments. The algorithm produced elevation models with elevation standard deviations of ~1.5 feet and grade standard deviations between 0.10% and 0.17%.

GPS ERROR CHARACTERISTICS

Absolute GPS error characteristics using civilian GPS receivers have been widely published. Several factors influence the size and shape of the error distribution. Among these are the number and distribution of GPS satellites, type and fidelity of GPS equipment, the use of differential correction services, atmospheric effects, activation of selective availability, jamming of the GPS signal, and local obstructions that limit the view of the sky, such as trees, canyons, and buildings. Many of the factors listed above primarily affect the absolute error in the data.

GPS data is typically taken in one-second intervals. If successive GPS data points use the same constellation of satellites, the relative error between the two data points is minimal. Assuming absolute errors of 2 m and 5 m, respectively, for horizontal and vertical error, the relative error between successive readings is easily sub-meter in both dimensions. The error correlation between successive GPS data was estimated between the 0.99- and 0.999-level in the work performed. Improvements to the GPS system in recent years, such as differential correction services and discontinuance of selective availability, have decreased the absolute error of the system, but have done little to change the relative error characteristics.

The error in the spatial model obtained from GPS data can be estimated using statistical sampling theory. The error in the estimate of the mean of a population is inversely proportional to the square root of the number of samples, as shown in Equation 1.

$$\delta_{\bar{X}} = \sqrt{\frac{\frac{1}{n}\sum_{i=1}^{n}\left(\delta_{X_i}\right)^2}{n}} \tag{1}$$

Applying this concept to the absolute positioning of highways yields the results shown in Figure 1, which shows the hypothetical reduction in the absolute position error of the three-dimensional model as a function of the number of observations, assuming that each observation has a 2-m random error in horizontal position and a 5-m random error in elevation. Assuming 2-m and 5-m absolute position accuracies of the individual GPS data run, the absolute accuracy of a spatial model is expected to reach sub-meter horizontal accuracy and ~2-m vertical accuracy by averaging a handful of GPS tracks. Equation 1 assumes independent observations. Figure 1 assumes that the data points are separated in time sufficiently to assume independent observations.

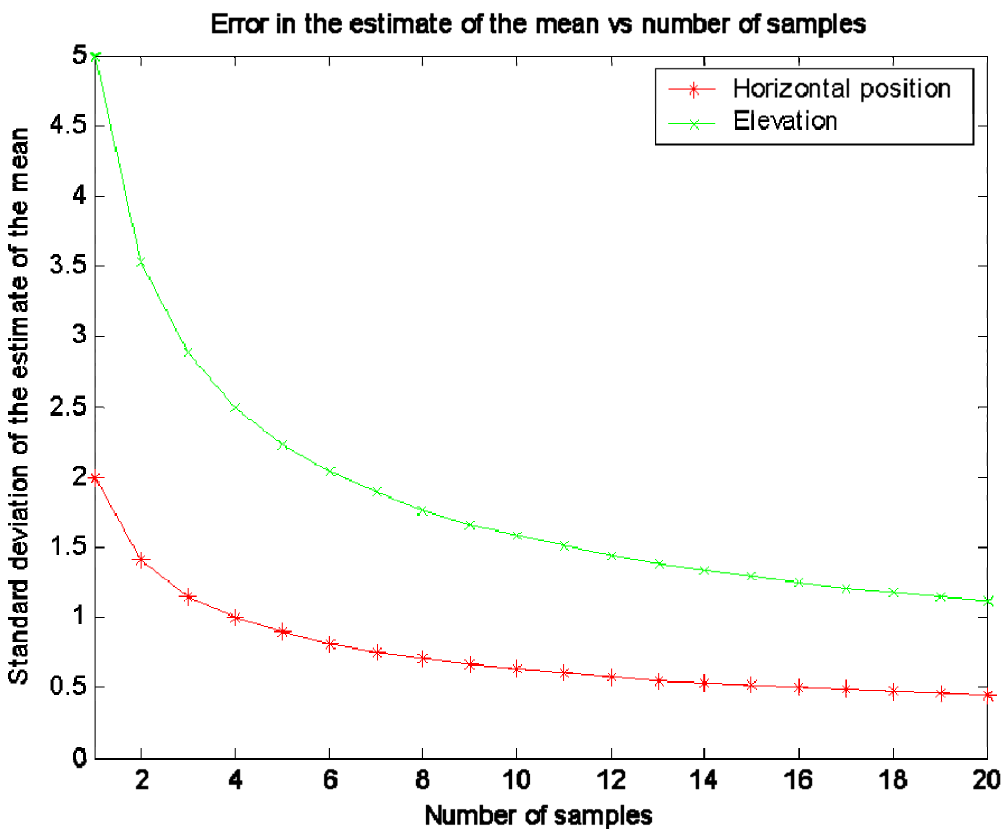

Figure 1. Hypothetical reduction in the absolute position error of the three-dimensional model as a function of the number of observations

The high correlation of GPS data error provides in essence a high quality estimate of heading in the horizontal plane and grade in the vertical plane. Additional error reduction arises because successive estimates of slope are highly independent, unlike position estimates. These GPS data attributes combined with basic knowledge of road design geometry allows for further error reduction. This is best illustrated with a simple example. An 800-foot crest vertical curve was constructed with a curvature of 247 ft per percent grade (%G). Samples of the grade were taken approximately every 12 feet to simulate seven GPS datasets. Random noise was added to the samples consistent with the error characteristics for a correlation coefficient, p = 0.99, which is approximately 2.6% grade, or 0.026 ft/ft. Since the grade of a crest vertical curve varies linearly with distance, a straight line was fit to the data using a covariance-weighted least-squares technique capable of producing one-sigma confidence bounds. Figure 2 illustrates the results. Simulated samples with a normally distributed error of 2.6% grade were fit with a first-degree polynomial (straight line) as shown. The one-sigma confidence bounds are illustrated in the dashed lines.

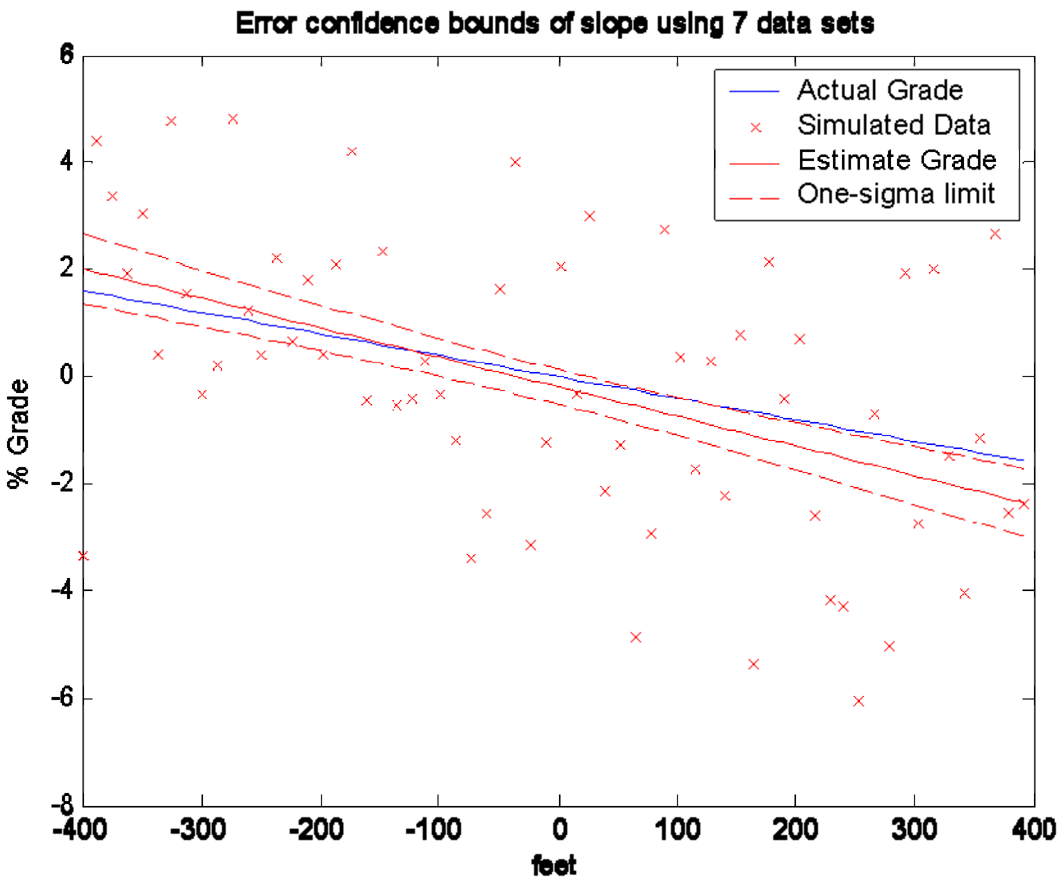

Figure 2. Slope profile for an 800 foot crest vertical curve with curvature of 247 ft/%G

Using this method, the expected error in slope from the spatial model as a function of the number of datasets can be calculated assuming typical values for GPS error and the correlation coefficient. The result of this calculation, assuming 5-m absolute vertical accuracy, a 0.99 correlation coefficient, and a simple 800-ft crest vertical curve are shown in Figure 3. Grade accuracies between 0.3% and 0.4% grade should be attainable with five or more GPS tracks. A similar analysis for heading indicates an expected one-sigma accuracy of 0.2 to 0.4 degrees after using five GPS tracks to obtain the model.

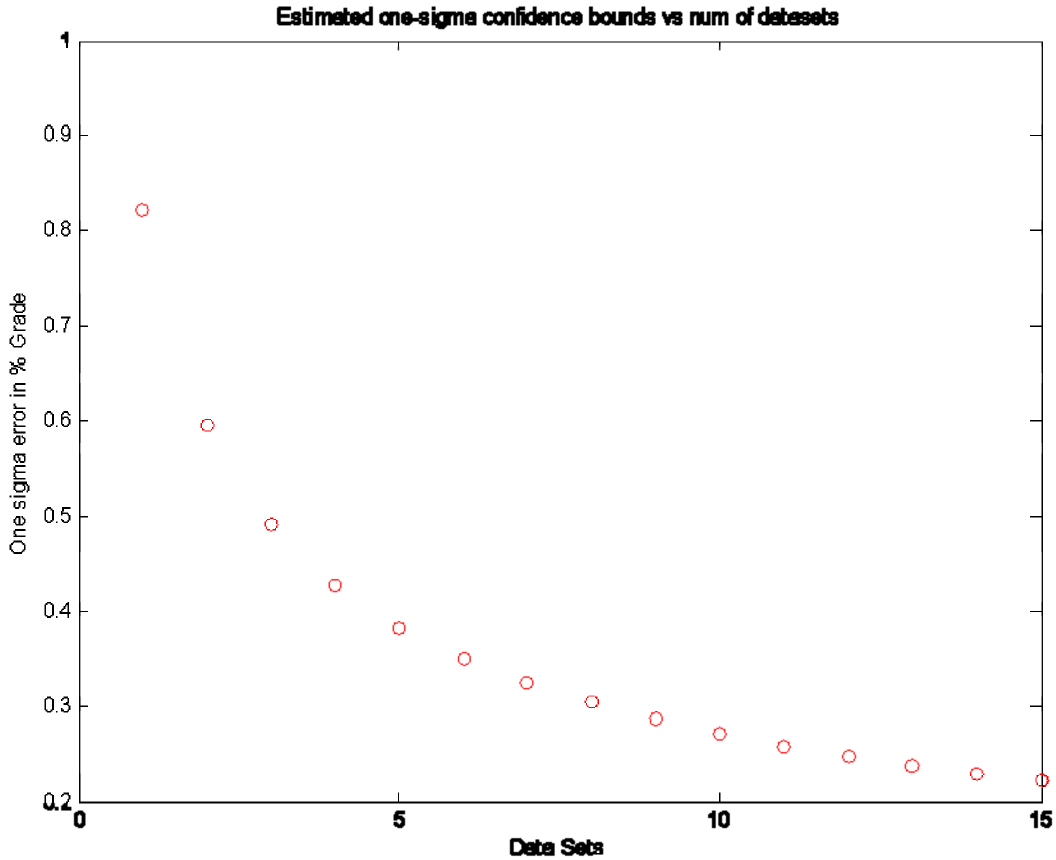

Figure 3. One-sigma confidence bounds for the estimate of slope on an 800-foot crest vertical curve

MODELING PRINCIPLES

The modeling methodology is based on a robust, iterative, linear least-squares methodology with covariance weighting. Each contiguous section of roadway is broken into 0.2- to 0.4-mile sections. Within each section, the coordinates are transformed into a normalized coordinate system. The independent axis is aligned with the general direction of the route and scaled between -1 and 1. The dependent axis is scaled accordingly in order to preserve first derivative information (heading and grade). The normalization of the data within each segment combined with the low-order polynomial basis functions provides a stable computation environment that closely resembles the Lagendre polynomials.

Slope information is estimated using a difference formula on successive GPS points. The spatial data (longitude, latitude, and elevation) and first derivative data (heading and slope) are used to estimate the parameters of the low-order polynomial on each section. Boundary constraints are introduced at the intersection of successive sections. The position, heading, grade, and curvature are maintained across the section boundaries. Lagrange multipliers provide a convenient method to enforce the boundary constraints within a linear (matrix) formulation.

The estimates of the polynomial parameters are based on least-squares minimization of the residual. The formulation of the linear problem includes weighting coefficients for each data point. The weighting coefficients are introduced in the form of a covariance matrix. The covariance matrix provides both the correct relative weight for each data point, as well as a method to encode data correlation on the off-diagonal elements. On the first iteration of the algorithm, the covariance matrix is initialized using expected values for variance and correlation. Each subsequent iteration uses the residuals from the previous fit to re-estimate the variance of each data point and the correlation between data points. This iterative approach enables detection of outliers, which are then de-weighted in subsequent calculations.

In addition to adjusting the covariance weighting matrix, the residuals are used to estimate the bias of each GPS track. The estimated bias is then subtracted from each respective track before the next iteration. The bias is estimated not only for the dependent coordinates, but also for the independent coordinate axis. Through a simplified correlation procedure, each GPS track is de-biased (or shifted) along the independent axis until it coincides with the model.

The procedure is repeated and usually converges within five to six cycles. The final solution provides not only an optimal estimate of the trajectory of the highway, but also significant information concerning the quality and consistency of the GPS data, from which an estimate of model accuracy can be derived. The covariance matrix from the final iteration provides a good estimate of the uncertainty in each data point. This covariance matrix in turn determines the confidence in the coefficients in the polynomial model, which in turn can be transformed into uncertainty estimates of slope and curvature of the final model through similarity transformations. In a similar manner, the final bias adjustments for each GPS track provide a measure of the absolute position uncertainty of the resulting model.

In order to obtain immunity from outliers, which is a commonly cited drawback of least-squares minimization, the algorithm relies primarily on percentile-based methods for determining sampling statistics. Common methods for calculating mean and variance are predicated on a well-behaved and consistent distribution of data about the mean. In the presence of outliers, which are quite common in GPS data, standard sampling statistics are greatly affected and tend to reflect characteristics of the outliers rather than capture the trend in the majority of the data. Percentile-based methods provide a robust approach to ignore data points that fall significantly outside the general bounds of the data, a situation commonly referred to as a long-tailed distribution. Robust percentile-based methods are used on each cycle to estimate the bias, variance, and correlation of each GPS track.

RESULTS

Elevation profiles obtained using the algorithm described above were compared with elevation profiles that had been manually transcribed from archived design plans on three Kansas highway sections; K-177 and K-113 in Riley County and a portion of US 283 in Hodgeman County. K-177 is a four-lane divided facility over slowly rolling hills. The highway geometry is comparable to higher class roadways, such as interstates where the effect of the local terrain is minimized by major cuts and grading. K-113 in Riley County is a two-lane structure built on rolling terrain. The road follows the local terrain more closely than higher class facilities. U-283 in Hodgeman County is a rural two-lane road built in an area of relatively flat terrain. All three highways run south to north. The GPS data obtained from the north-bound lanes are compared to the vertical design profiles. The number of equivalent full-length GPS tracks for each highway was 8, 3, and 5 for routes 177, 113, and 283, respectively. The actual numbers of tracks for each highway are greater; however, each individual GPS track may not have spanned the entire highway section being modeled. The highways were modeled with approximately 0.25-mile intervals and fifth-order polynomials.

Figure 4 depicts the elevation data for K-177 and the model obtained from the algorithm. The maximum distance between the GPS elevation traces is over 150 feet. Although the span between the traces is large, it is not wholly unexpected. Civilian GPS receivers are known to have built-in elevation biases (Wilson 2004). The data appear to have two clusters of elevation traces. This may be data from two different receivers, each with a different built-in bias. Many receivers have user-defined fields for antenna height. No data exists to determine how these user-defined fields were programmed during data collection. However, Figure 4 indicates that the relative shape of the roadway is consistently captured in the GPS data, despite the differences in absolute elevation. The reason for the large spread in elevation bias is unknown. The data appears bimodal, suggesting that possibly two different GPS receivers, each with a different bias, were used to collect the elevation data.

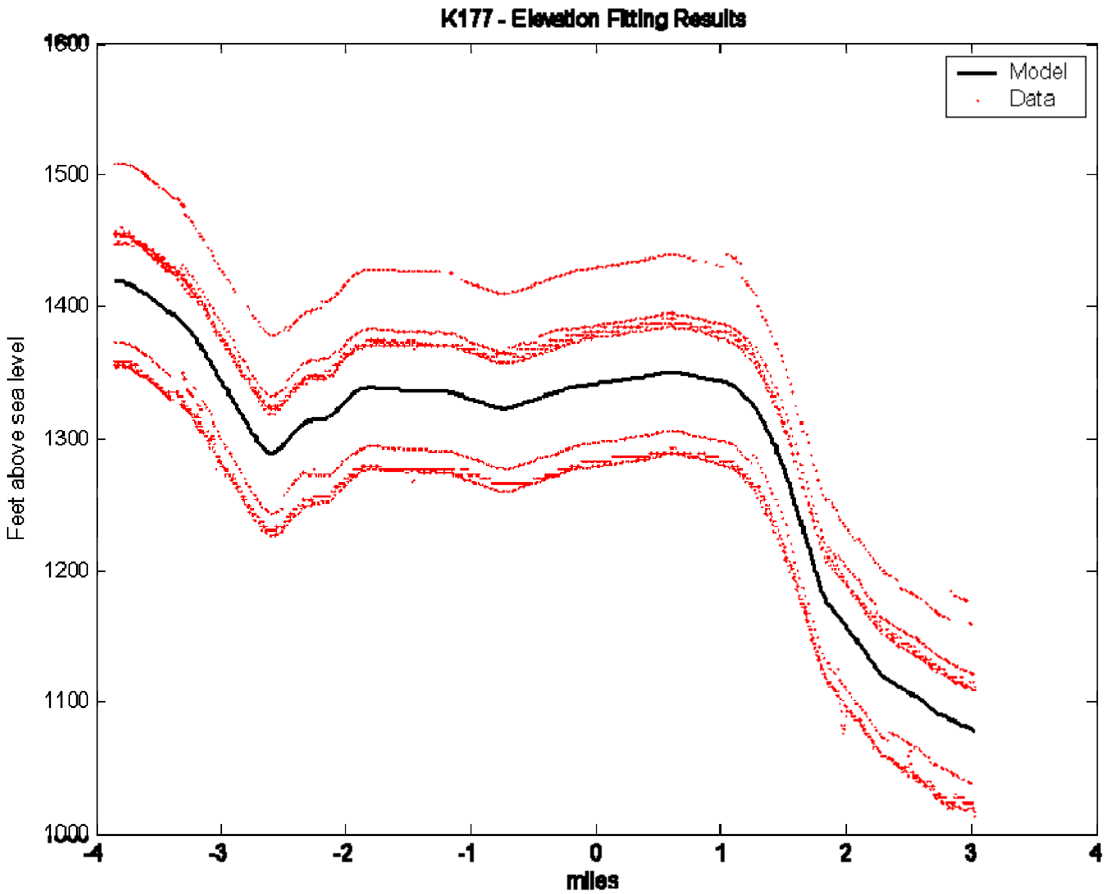

Figure 4. GPS elevation for K-177, in red, and the model obtained from the 3-D algorithm, in black

Despite the elevation biases depicted in Figure 4, a high-quality geometric model was obtained from the data and compared with the design profile, as shown in Figure 5. The red and blue traces in Figure 5 are a direct comparison of the elevation model to the design elevation profile, respectively. The difference between the model and the design profile are magnified and graphed in black on the lower portion of Figure 5, with the appropriate scale indicated on the right axis.

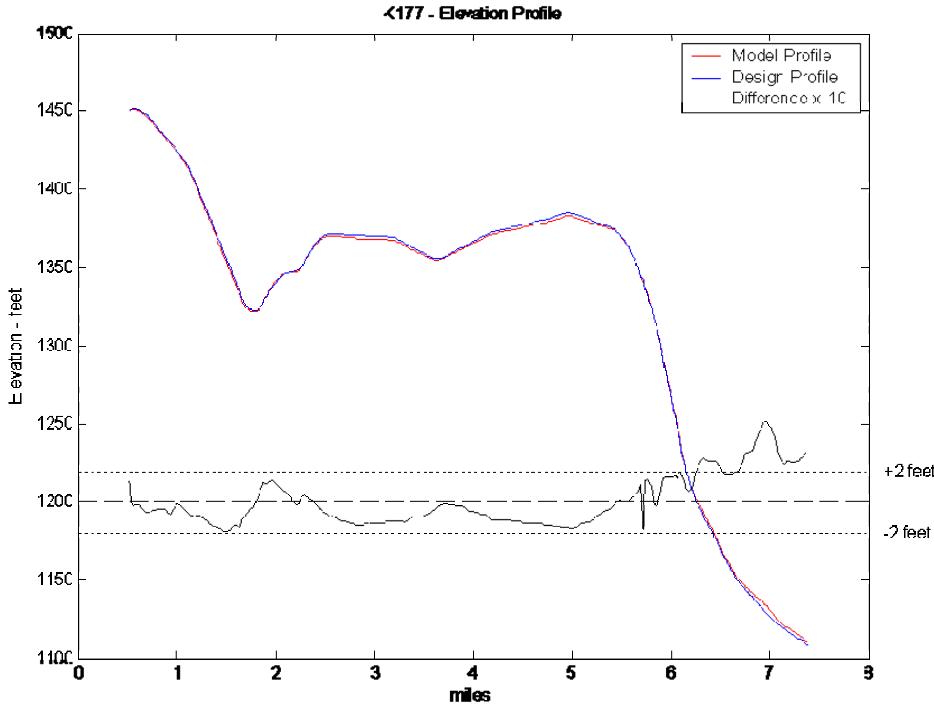

Figure 5. Elevation model for K-177

Figure 6 depicts the model grade compared to the design grade. The grade profile obtained from the 3D algorithm for K-177 is shown in red. The design grade, shown in blue, was obtained by applying a simple difference formula to the design elevation profile. Note the obvious error in the design profile at mile 5.8. The abrupt changes in the design grade at approximately 5.8 and 6.1 miles indicate transcription errors in the design grade obtained from archived plans. Recall that highways are designed primarily with parabolas and tangents (straight lines). This results in a continuous, but non-differentiable grade profile. Linear models work best on continuously differentiable curves. The abrupt changes in the rate of change of grade at the transition between parabolic and straight line segments causes the linear model to overshoot and undershoot, commonly referred to as ringing in classical control theory. This is most evident toward the end of the profile in Figure 6 between mile 6.4 and 7.5.

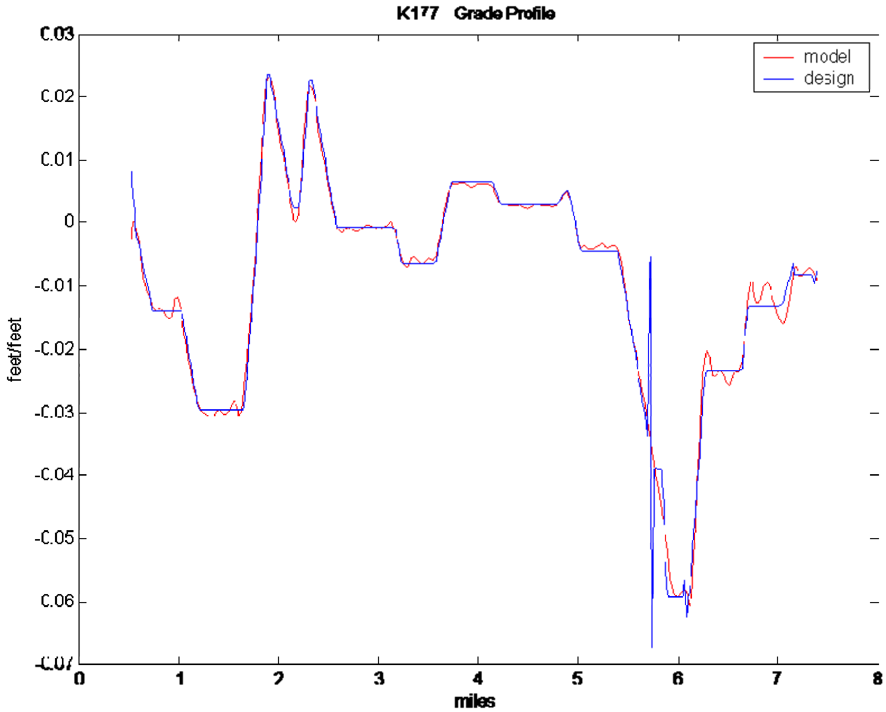

Figure 6. The grade profile obtained from the 3D algorithm for K-177

The agreement between the model grade and the design grade for the three highway sections used for verification are characterized in Table 1. The first row, labeled Standard Deviation, indicates the one-sigma error value between the model and the design profile obtained using standard sampling statistics. It is heavily influenced by outliers, such as the differences in grade due to transcription error of the grade profile. The Robust Sigma statistic label is a percentile-based method of estimating the one-sigma error boundary. It ignores the influence of outliers and is a better estimate of the agreement between the two profiles. The Model Prediction label indicates the expected one-sigma error in the model derived from the standard error of the coefficients. Note that the one-sigma error estimates in grade are less than the 0.3% to 0.4% bounds predicted from basic statistical principles and conservatively estimating the correlation in GPS error distributions. This is an indication that the correlation coefficient (a measure of the relative accuracy of GPS data) is greater than the 0.99 assumed in the preliminary calculations.

Table 1. One-sigma estimates of grade error between geometric model and design plans

	K-177	K-113	U-283
Standard Deviation	0.27	0.20	0.53
Robust Sigma	0.094	0.15	0.17
Model Prediction	0.049	0.09	0.07

(all values are one-sigma estimate of error in % grade)

CONCLUSIONS

KDOT has successfully developed procedures to use GPS data collected in existing highway inventorying systems to generate mathematical functions of highway geometry for which the accuracy can be reasonably estimated. The error in the geometric model will be reduced as the number of GPS data points grows. Subsequently, KDOT has developed a sample application built on the geometric highway models to evaluate vertical sight distance. The advantage of such an application is the ability to evaluate the entire state highway system for potential vertical site distance problems under a variety of input parameters for driver eye height and/or object height. These parameters have been changed in the recent past and, if adopted by KDOT, would have required a significant undertaking to understand the impact. Again using the geometric models, similar applications could be developed for horizontal site distance analysis, sight distance at requested access points, or, for that matter, as a means to maintain a GIS basemap. KDOT plans to maintain and continue to update the geometric models as additional data become available. Additionally, KDOT hopes to build additional applications based on the models to take advantage of now having the entire state highway system located in a three-dimensional, contiguous mathematical model with estimates of accuracy.

ACKNOWLEDGMENTS

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Geospatially Enabled, In-Vehicle Information Services Architecture: Visual Data Radio

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ABSTRACT

A GPS-augmented radio with minimal processing and display capabilities combined with XML technology and wireless communications can provide a simple yet robust method of delivering spatially enabled data to drivers at low cost with a minimal data-processing burden. This paper explores the benefits of this proposed architecture. In this architecture, the information displayed to the driver is obtained via wireless communication, such as satellite radio broadcasts or wireless internet devices, commonly referred to as WiFi. The data is filtered specifically for the vehicle's location (geospatially enabled). The traveler views only information on requested services such as restaurants, lodging, fuel, or travel advisories. The revenue format is similar to advertising in telephone directories or via radio. Service providers are charged to be included in the spatially enabled database, much in the same way they pay to appear in the yellow pages or industry trade journals. Additional subscriber fees could be charged for value-added information, much in the same way that cable charges for enhanced broadcasting. Information is displayed in a simplified map format, hence the name the visual data radio.

Key words: GIS—GPS—visual data radio

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INTRODUCTION

The information revolution is quickly changing traditional paradigms of data access. Resources available via the internet have become the primary source of reference information for the majority of the U.S. population in a little less than a decade. Quickly maturing mobile data communication technologies and standards combined with the inexpensive and effective location referencing provided through GPS will, within the next few years, enable a completely new paradigm for accessing traveler information and resources while navigating U.S. roads and highways. Products and services currently on the market provide evidence of this rapidly developing area. This paper reviews several existing commercial products and services that exhibit the supporting technologies that will form the basis of the next generation of in-vehicle information services, here called the visual data radio (VDR). The architecture upon which this concept is based comes from the culmination of several maturing technologies combined with an established business model that generates revenue by providing access to and distributing information.

The VDR architecture is diagramed in Figure 1. The VDR is not a product, but rather a conceptual method for delivering location-sensitive data in an efficient and supportable manner. All of the elements in Figure 1 are commercial products or standards. The significance of the architecture is that combining all these existing technologies enables a new method of delivering content to the mobile user. Every block in Figure 1 is a critical path or critical technology. In short, the VDR appears to the user as a display screen within the vehicle. Unlike traditional computer products, the VDR has no keyboard or confusing buttons over-laden with functionality by navigating on-screen menus. The radio is tuned to various services. For instance, the tuner may have selections for restaurants, gas stations, hotels, historic sites, recreation, etc. The display resembles a map and the vehicle itself is marked on the map. Another knob is used to control the extent of the map. The extent may be only a city block or an entire state. Information on the location of available traveler resources (restaurants, hotels, etc.) is communicated to the traveler through a data connectivity infrastructure providing ubiquitous communications. Physical communication is accomplished either through high-speed, terrestrial communications technologies, such as the popular IEEE 802.11 standard and its associated family of products, commonly referred to as WiFi, or through centralized satellite broadcasts. Using either medium, the data exchange is handled via common data structures and formats such as XML. This allows for the decentralization of the supporting application architecture and thus cost efficient management of the whole system. Push technologies made popular on the internet for filtering the broad spectrum of available news and information is embedded in the VDR to access only the requested data and display it appropriately. The GPS equipment automatically provides the vehicle with location coordinates, which in turn enable location-specific display and filtering of data. The revenue model of the whole system is analogous to existing directory advertisements for phone service as well as radio advertising. Businesses pay to have information about their establishment listed in the directory or perhaps even actively marketed through the system. The advertisement revenue funds the infrastructure elements and provides the majority of profits. Travelers pay only for the onboard electronics, which include the display, GPS, and associated communications equipment.

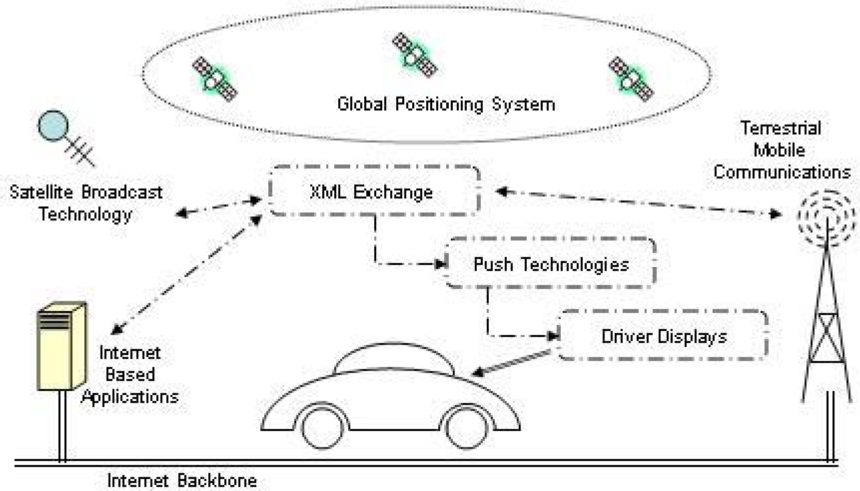

Figure 1. Supporting architecture of the visual data radio

The architectural elements of the VDR are discussed in detail in the remainder of the paper. Each element is briefly explained, and then its role in the VDR architecture is characterized. The goal of this paper is to explore the new realm of information accessibility that will result from the confluence of several quickly maturing technologies. The prognostications are based solely on the authors' observations and experience and do not reflect any vested business interest. It is the hope of the authors that the evidence presented will begin a discussion that will hasten the movement toward such systems, thus hastening the benefits.

ARCHITECTURAL ELEMENTS

The key technologies and concepts that enable the VDR are briefly explained. Specific technology examples are given as appropriate.

Communications Technologies

A robust mechanism is needed for transmitting information to a vehicle. Two such mechanisms are described below. A satellite broadcast provides the benefit of a single point of administration and global coverage. However, a true broadcast does not allow for two-way communications. The technologies associated with the IEEE 802.11 family of standards provide license-free, short-range, line-of-sight, two-way data communications. These technologies, commonly known as Wi-Fi, are quickly spreading. This technology benefits from rapid, inexpensive, standards-based, license-free deployment. The two-way communications enables capabilities and efficiencies over a pure broadcast-type mechanism. However, the power restrictions and line-of-sight nature of the medium limit the area of any single installation, requiring a network of antennas to provide region-wide connectivity.

Note that cellular coverage is not addressed in this discussion. Its properties are similar in some ways to the 802.11 technologies, but are dedicated to point-to-point voice circuits. The author does not preclude the use of cellular infrastructure, but doubts its ability to adapt quickly to the growth market, as the VDR.

802.11 Technologies

The 802.11 family of technologies has enabled a new generation of portable computing. Since 2002, the number of laptop computers sold yearly has eclipsed the sales of traditional desktop computers. This trend has been fueled in part by the Wi-Fi technology that provides high-speed connectivity, typically at 2 MBits per second or better, to computers without any physical connection. An explanation of the Wi-Fi technology or supporting standards is beyond the scope of the paper. Some important characteristics to understand is that Wi-Fi is full two-way communications, is license-free, line-of-sight and highly directional, and relatively inexpensive, costing roughly $100 for the basic equipment.

In March of 2005, a British company announced a plan to roll out a wireless network to support location-based services based on Wi-Fi technology. The company plans to use lampposts throughout a municipality to mount transmitters. As further evidence of its proliferation, in 2004 and 2005 both Texas and Iowa initiated programs to provide Wi-Fi connectivity at rest stops. The programs are intended to distribute traveler information (such as weather advisory or road closure information) to the traveling public via a wireless internet connection. Travelers with a Wi-Fi-enabled laptop or similar computing device can view a web site with pertinent traveler information. The traveler can also visit other web sites or check email (this may require a small fee, depending on the program). Both the Iowa and Texas WiFi programs cost the respective departments of transportation nothing. The vendor in each circumstance recoups cost (and obtains profit) through a combination of advertising and/or charging for general-purpose internet access.

In the near future, major travel corridors may be blanketed with Wi-Fi access, either for general-purpose internet access or for application-specific connectivity (such as the DVR). Already, 802.11 standards are used for communication-based train control along rail corridors. The Wi-Fi technology has advantages over broadcast in bandwidth and configurability. The drawbacks include limited coverage for a single antenna and the management burden of distributed assets. Low-volume rural roads may never possess significant revenue potential to draw investors to implement the Wi-Fi infrastructure.

Digital Radio Broadcasts

Traditional audio radio is being overhauled for the digital age. In 2005, the National Radio Systems Committee (NRSC) approved a digital broadcast standard. This will pave the way to offer not only improved audio quality for traditional programming of music, sports, and talk, but also data. The data is initially intended to carry such items as song titles and artist information. This would allow the user to program the radio to watch for a specific song or artist on the airwaves and then switch automatically to the appropriate station when available. Once in a digital format, the data capabilities of the radio are completely generic and could be used for a variety of purposes. In relation to the VDR concept, the data capacity of digital data radio could include information on traveler services and updates of local maps. This data is not audio or spoken information, but rather a machine-readable data structure, as discussed in the XML explanation later in the paper.

Satellite-based radio is quickly becoming popular. Currently for a subscription-based service, the audio broadcasts are digitally encoded and transmitted from satellites. The digital nature of the transmission provides for flexibility of content other than audio (music and talk), as described above.

GLOBAL POSITIONING SYSTEM

Initiated as a military application and later popularized through industry, the GPS provides accurate positioning information at a relatively low cost. The OEM circuitry that enables a GPS receiver has fallen below $40 and is being incorporated into many consumer goods, such as cell phones and watches in addition to dedicated navigation devices. The basic GPS receiver provides a three-dimensional position fix in terms of longitude, latitude, and altitude that is accurate to within 15 feet worldwide. GPS is one of the primary drivers enabling many geo-spatial information systems, such as the DVR.

With the DVR architecture, GPS provides location fixes for the traveler in an automobile. The data is encoded in standard formats and used as a filter for querying traveler services databases and cross-referencing the location of the vehicle on available road maps. GPS also enables easy construction and maintenance of traveler service databases. Although address matching is frequently used in mapping software, a direct longitude/latitude attribute for services greatly simplifies application complexity. It can also ease the burden for businesses subscribing to the advertising service. Terrestrial coordinates can be measured by the business owner and input directly. This places the burden for accuracy on the entity that will directly benefit from advertising.

Garmin, a navigation electronics corporation based in Kansas, leads the market in GPS devices in several markets. Their Street Pilot series of products provides a clue as to what DVR can provide. These products by Garmin package a GPS receiver with a mobile computer that contains mapping software and a travel service database. The instrument allows the user to view the location of the vehicle in reference to a route map and surrounding attractions. The primary difference between the DVR and the Street Pilot device is that the information in the Garmin products is static. Although its database can be updated periodically if docked with a desktop computer, these devices can only access data if it already exists in its own database. The DVR is a dynamic system that can retrieve and store data interactively through a mobile communication system. Some data items change very rarely, such as the coordinates that describe roads and highways. However, some information is very dynamic, such as road closures due to weather or amber alerts. The latter cannot be managed in a static database arrangement. It is this time-sensitive data that is most critical to traveler safety.

XML Data Structures and Information Feeds

The extensible markup language (XML) is a natural choice for delivering information that may be sensitive to one or any combination of time, location, and context. In its simplest form, XML presents data and metadata in a strictly structured yet intuitive form. Consider this possible example of a data stream that might be sent to the VDR device:


```
<DATA>
    <LATITUDE>390651N</ LATITUDE>
    <LONGITUDE>0943738W</LONGITUDE>
    <RADIUS>125</RADIUS>
    <CATEGORY>ATTRACTION</CATEGORY>
        <SUBCATEGORY>MUSEUMS</SUBCATEGORY>
        <SUBCATEGORY>FREE</SUBCATEGORY>
    <CATEGORY>CHILDREN</CATEGORY>
    <CATEGORY>ADULTS</CATEGORY>
    <MESSAGE>Come see the world famous Metropolis Museum of Natural History.
    Guided tours every 90 minutes. Open 11–9 Mon.–Sat., 11–4 Sun.</MESSAGE>
</DATA>
```

From a business model perspective, the simplicity, flexibility, and intuitiveness of XML lends itself well to direct content management by the advertising customer and furthermore could place the customer in the position of optimizing their advertising dollars based on an *a la carte* product list. Consider the following simple menu of options that could be in a level-of-service contract:

RADIUS

- 25 Included
- 50 add $2.50/mo.
- 100 add $3.00/mo.
- 125 add $3.50/mo.
- MORE please contact us…

MAJOR CATEGORY

- 2 Included
- 3 – 5 Add $1.00/mo. per each.

SUBCATEGORY

- 2 per MAJOR CATEGORY Included

MESSAGE

- 125 Characters Included
- 126 – 150 Characters Add $.15/mo. per each character.

The XML structure is necessary to allow electronic parsing and filtering of content appropriate to the user. The XML tags form an agreed-upon lexicon between the traveling public, the electronics that support the VDR, and service providers. It is this type of structured data format and standards that has fueled the information explosion through the internet and will propel geospatially enabled delivery of data into mobile platforms.

Categorization of information and advertising offers the traveling customer control over the messages they wish to receive. The success of this kind of service will be strongly tied to the accuracy and reliability of that categorization. If people looking to purchase a used car happen upon a newspaper, they will likely navigate straight to the classified advertisements, then onto the automobile section. Likewise, if travelers only want to know about lane closures within 10 miles, zoos within 100 miles, and Mexican restaurants within 5 miles, they should have the ability to filter the information they receive to include only those categories. The internet search engine market teaches us that, unless information can be obtained intuitively and the results of filtering the information by category are relevant, the service will not be used.

Driver Display

The driver display concept is given in Figure 2. The primary point of Figure 2 is the simplicity of control. The layout reflects the control concept of existing appliances and radios, rather than some type of data entry device. The information is displayed in a map format. The controls are knobs and push-buttons, not keypad entries or complicated multi-function, reprogrammable buttons. This does not preclude the use of touch screens, heads-up displays, or more complicated controls. It simply emphasizes that the interface of basic electronics must have a low-level entry point in order to be accepted across the broad spectrum of society. The form factor for the basic car radio has survived the digital revolution without replacement of the primary controls of a tuning mechanism, volume control, and preset stations. Although rotary dials have, in some models, been replaced by up and down buttons, the basic minimal control philosophy has survived and appeals to a broad spectrum of users.

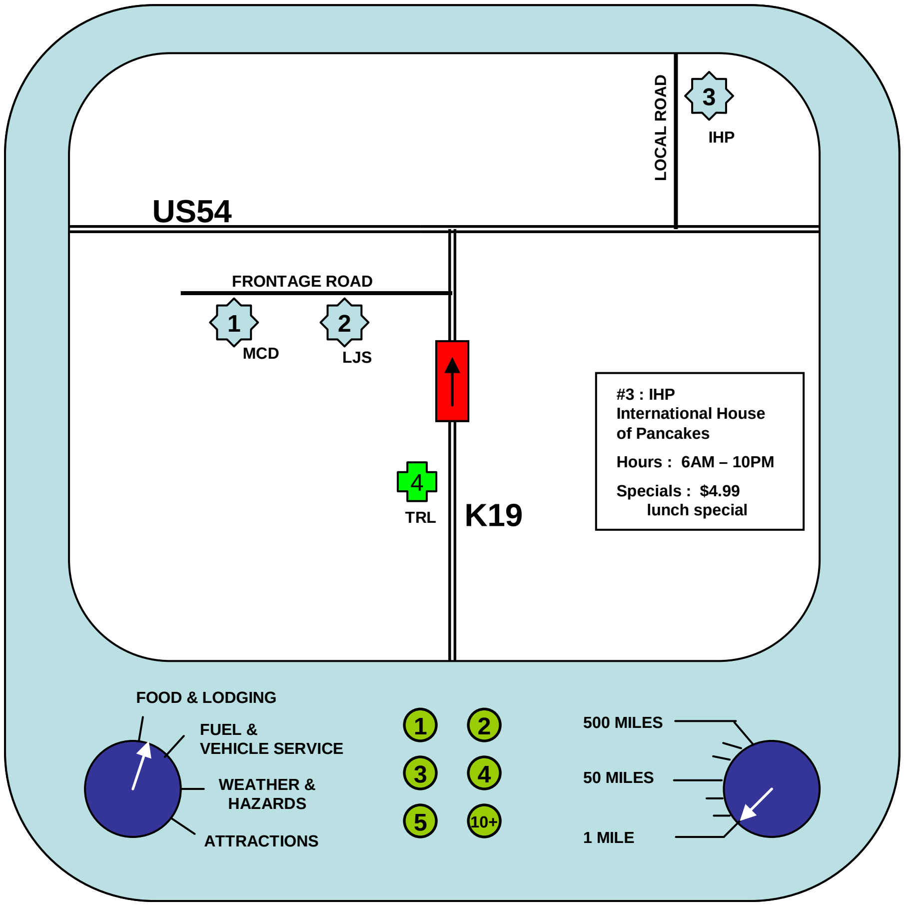

Figure 2. Sample layout of the visual data radio

Internet Backbone and Internet-Based Applications

Similar to GPS, the internet and internet-based applications that have proliferated in the past ten years provide the template for a distributed, maintainable backend data support structure for the VDR. The business model to support the VDR will require extensive input from geographically diverse clientele. Although it is conceivable that all the data pertaining to traveler services could reside in a single, highly fail-proof data center, experience with internet applications indicates that a distributed application architecture is dependable and much more scaleable and expandable. For instance, time-sensitive traveler alerts may emanate from a number of different sources, such as weather services, law enforcement agencies, road departments, or the homeland security department. The time-sensitive data is, in most instances, location-dependent as well. The distributed application infrastructure can more easily distribute load and provide fail-over support in the event of network outages.

The database necessary to drive the VDR and make it useful to travelers is immense. It can be pictured as a compilation of all the phone directories of all municipalities interconnected and searchable. The internet-based application used to input traveler services provides for a highly parallel architecture for data input. The burden of the inputting the data and maintaining its accuracy can be transferred to the business owners seeking recognition as traveler services. Fee collection can be completely automated, eliminating accounts receivable.

Access to data by the traveler, particularly in the 802.11 data communication mode, is streamlined. All updates to servers are conveyed over public infrastructure. Data encryption ensures security and accuracy of information.

Business Model

Although the business model is discussed last, it is probably the most critical link in deploying a VDR concept. Methods to generate revenue to recover the cost of deployment and maintainence of a consumer information system vary. Telephone circuitry is billed by the connection. Radio is essentially free to the consumer, except for the investment of the receiver. Satellite radio (with traditional programming) requires a monthly access fee, much in the same way cable TV and dish TV networks operate. However, apart from the telephone system, the majority of consumer data systems derive the revenue from advertising rather than user fees. This is true even for internet access, as described in the Wi-Fi initiatives for Texas and Iowa. Businesses will pay a premium for exposure to customers.

The primary revenue source envisioned for a VDR is advertising. Similar to the phone directory, the system will provide highly directed marketing to customers. The information will not only be location-specific, but also highly time-sensitive. The data is delivered to potential customers as they pass through the service area. In many instances, customers may have no prior knowledge of the services available, so the opportunity to provide relevant, targeted marketing service is immense. The VDR architecture does not preclude the use of other revenue sources; however, evidence from other markets indicates that advertising will be the primary driver.

CONCLUSION

The architecture presented for the visual data radio is simply an educated guess of the direction that in-vehicle information systems will evolve in the coming years. The confluence of GPS technology and wireless networking will usher in a completely new paradigm of information access for a mobile society. These two technologies, combined with the robust, distributed internet architecture and the electronic data standards, such as XML, made popular through the internet, will form the backbone of spatially enabled information services, not only for roads and highways, but also for any form of mobility. The proliferation of wireless communications offers the ability to distribute information with a minimal investment in physical infrastructure. Similar to broadcast television, wireless data communication offers the promise to reduce the infrastructure cost to a point that advertising revenue could sustain the system. The information device within a vehicle is the only cost born by the user. The robust and scalable nature of digital data provides a path to upgrade service and content (and possibly even functionality) in the future. It will be exciting to see this application develop in the coming months.

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Nondestructive Testing Device for Tie Bar Placement Accuracy

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ABSTRACT

The Kansas Department of Transportation (KDOT) in cooperation with Kansas State University (KSU) and the American Concrete Pavers Association (ACPA) is developing an instrument capable of assessing tie bar placement accuracy suitable for use in a paving environment. The instrument is now in its fourth prototype. The third-generation prototype used a single magnetic sensor and was capable of assessing depth accuracy at a rate of approximately two miles of construction joint per hour on projects during the 2004 construction season. KDOT is developing a new placement specification appropriate for this type of sensing technology. The fourth-generation prototype will possess multi-sensor capability to assess lateral placement as well as depth accuracy. Due to cooperation with the ACPA, this instrument is targeted to be integrated into paving equipment for real-time quality control. This paper reviews the basic sensing principles of magnetic tomography and how it can be successfully integrated with microprocessors and communications into a nondestructive instrument for use in a concrete construction environment.

Key words: magnetic tomography—nondestructive testing—tie bar

PROBLEM STATEMENT

The Kansas Department of Transportation (KDOT), in cooperation with Kansas State University (KSU) and Koss Construction via the American Concrete Pavers Association (ACPA), has been developing an instrument capable of assessing tie bar and dowel bar placement accuracy for use in a paving environment. KDOT demonstrated a single-sensor prototype in 2004 that was capable of assessing tie bar depth at a rate of approximately two miles of construction joint per hour. In 2005, KDOT is moving forward with a revision of the tie bar placement specification appropriate for use with this type of sensing technology. KDOT is also pursuing a multi-sensor prototype in order to assess lateral placement accuracy and depth accuracy. In 2004, Koss Construction committed extra manpower to their paving operation that was dedicated to monitoring tie bar placement depth. The project sponsors intend to integrate the multi-sensor instrument directly into the paving equipment later this year to facilitate real-time quality control.

BACKGROUND

In 2001, KDOT began investigating the feasibility of developing an instrument capable of nondestructively assessing the depth and orientation of steel reinforcement, namely dowel bars and tie bars, in concrete pavements. After surveying possible techniques, magnetic tomography emerged as the most promising technology. Magnetic detection relies on magnetic permeability differences between the target (in this case, steel bars) and its surroundings (such as concrete). It is the same basic principle used in concrete cover meters. Tomography simply refers to the mapping of these magnetic properties to form a curve, contour, or image that represents the target. Although nondestructive electromagnetic techniques such as ground penetrating radar continue to be refined, magnetic tomography has the additional advantage of being unaffected by moisture, which allows it to be used on plastic concrete.

An experimental x-y scanning table was constructed at the KDOT Materials and Research (M and R) in 2001. The scanning table coupled electronic signals from two optical encoders, one for each axis, with the audio feedback from a Kolectric MicroCovermeter and stored the data electronically in a PC. A sample of the data obtained from this device is shown in Figure 1. The picture depicts the placement of dowels in a dowel basket array (note that the vertical units are in error in the diagram).

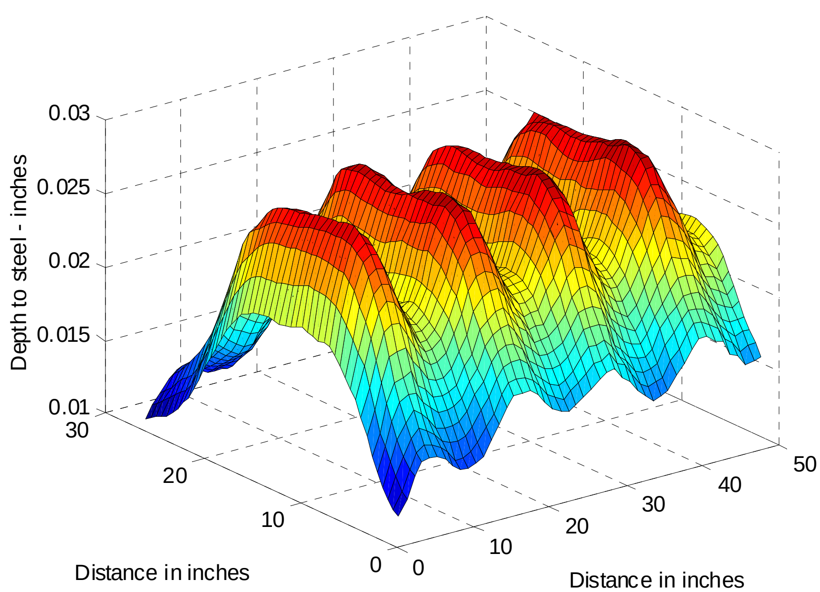

Figure 1. Two-dimensional magnetic scan of a dowel array

Although cover meters are routinely used to investigate suspected misplaced steel and to locate reinforcement steel prior to drilling core samples, the use of a cover meter to systematically assess placement accuracy is far too manpower-intensive to be feasible. The data provided by the experimental 2D scanner indicated that the sensing technology within the cover meter was capable of yielding accurate depth and orientation when coupled with an electronic positioning device. The concept came to be known as a cover meter on wheels.

In the fall of 2001, KSU Electrical and Computer Engineering proposed to KDOT the development of a cart-type device that incorporated this principle and could be used during construction behind a concrete paver. The proposal was accepted and work commenced in the summer of 2002. The resulting prototype is shown in Figure 2. Working with Kolectric, a custom cover meter was produced that directly outputs signal strength in real-time via an RS-232 serial connection. Distance was logged through use of an optical encoder wheel. The device demonstrated that depth and orientation of tie bars and dowel bars could be obtained with multiple passes over the joint. Rectangular slots cut in the transparent tray that carried the detection coil provided known distance left and right of the joint, which allowed the generation of multiple traces and the development of a 3D perspective of the steel's orientation.

Figure 2. Prototype instrument developed at KSU in 2003

The device demonstrated its potential in the laboratory, but practical concerns of field operation prohibited multiple passes and prevented rapid data collection. The basic operational principles were valid, but the design of the cart went through two more iterations at the M and R laboratory in 2003 and 2004 before a functional prototype was produced. The current prototype, shown in Figure 3, was successfully demonstrated in field tests during the 2004 construction season.

Figure 3. M and R prototype built and field tested in 2004

The current three-wheeled device works in a similar fashion to the KSU prototype, but alterations in the chassis allow for rapid data collection. The acrylic sled for the sensor coil maintains an offset of approximately 1/4 of an inch from the concrete. The sled is fastened by two nylon bolts toward the front of the prototype. This allows the sled to rise whenever debris or excessive crown interferes with available clearance. These and other innovations allowed the prototype to demonstrate rapid data collection in 2004 in varying construction environments, including rain, mud, and heavy dust. Over four miles worth of data were collected in less than two hours on the last project. Table 1 lists the projects in 2004 where the current prototype was used to collect data.

Table 1. Data collection activities in 2004

Date	Description
April 2004	Data samples on I-35 and I-70 following the MIT-SCAN
July 2004	Wichita, sections of I135 North of Wichita ~3,000 feet of centerline longitudinal joint during rain
August 2004	Kansas City, sections of 635 ~1,200 feet of longitudinal joint, extremely dusty environment
August 2004	Wichita, ~6,000 feet of I-135 to confirm correction of tie bar placement process
August 2005	Wellington Airport, ~1,000 feet of shoulder longitudinal joint to confirm suspected tie bar problem
October 2004	I-35 near BETO junction, ~4 miles of data along center and shoulder longitudinal joint confirmed correct placement of tie bars

DATA ANALYSIS

Presentation and analysis of data was performed using a customized program that displayed the one-dimensional trace and estimated the depth to the tie bar. Examples of this are shown in Figures 4 and 5. Figure 4 depicts the location of tie bars along a shoulder joint on I-135 North of Wichita, Kansas. An exposed shoulder joint provides a convenient source for validating the instrument. The exposed steel tie bars allow for visual inspection and direct comparison with the results from the prototype instrument. Notice the irregularity at ~17 feet in Figure 4. This phenomenon is caused by the proximity of the steel dowel baskets at a transverse joint.

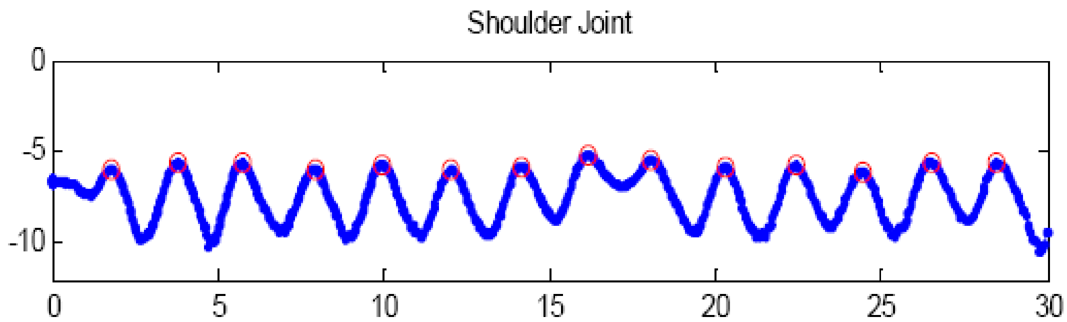

Figure 4. Tie bar depth (in inches) along a 30-foot section

Figure 5 depicts the depth of tie bars along the centerline joint on the same section of roadway. Note that in the first 30 feet, the tie bars are discernable, with one tie bar near the surface at 15 feet, probably at a transverse joint location. From 30 to 60 feet, most of the bars are near the bottom of the pavement. The data collected on this project confirmed a suspected tie bar inserter problem.

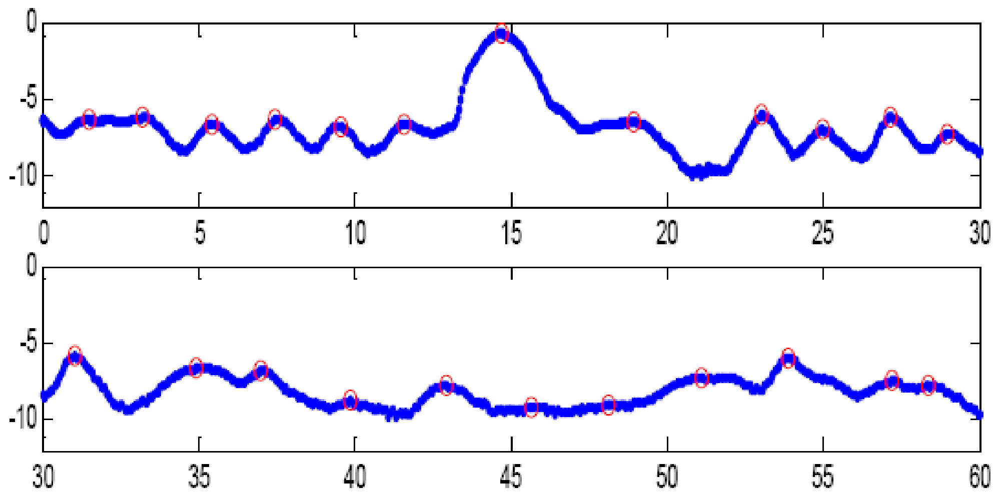

Figure 5. Tie bar depth (in inches) along a 60-foot section

The current prototype has been ruggedized for field work, but provides only depth to steel at the construction joint. Theoretically, multiple passes of the instrument at known lateral offsets from a construction joint could be combined to obtain orientation as well as depth, as was demonstrated in the laboratory at KSU. However, multiple passes are not practical in the field. Even if manpower were not an issue, debris on the road causes slight deviations in measured distance. These errors in turn cause inaccurate orientation calculations. An instrument that could take two or more depth measurements simultaneously would be more practical. This is the goal of the next prototype.

COMPLIMENTARY WORK

In 2003, a German company began marketing a product in the United States called MIT-SCAN. MIT-SCAN is similar in concept to the current prototype, but it specifically targets the automated dowel bar inserter market. The MIT-SCAN instrument requires that a track be placed on the pavement prior to collecting data. MIT-SCAN contains five sensors and can assess the orientation of a dowel array in a single pass. However, several factors inhibit it from being used in the field on an ongoing basis. The dependence on a customized track limits its data collection speed. In April 2004, the M and R prototype was tested alongside the MIT-SCAN. The M and R prototype collected data at approximately twice the speed of the MIT-SCAN device, and this was prior to improvements in the prototype software that allowed for more rapid data collection. This combined with its high cost and other factors led KDOT to continue with its independent development.

During the 2004 construction season, Koss Construction hired an additional worker on its concrete paving jobs to monitor tie steel placement. The sensing coil of a cover meter was placed on the burlap drag directly behind the paver. The worker was responsible for monitoring the cover meter, continuously checking for appropriate depth of tie steel. Normal paving consists of placement of two lanes of traffic and a shoulder in one pass. The meter was rotated between and centerline and shoulder joint throughout the day. Of the data collection activities in 2004, the I-35 project near BETO junction, constructed by Koss, was the only job in which the prototype confirmed correct placement of tie steel rather than confirming misplaced steel.

FUTURE DEVELOPMENT

Moving forward, KDOT has committed to revisit the specification that governs the accuracy of tie bar placement, assuming that an instrument is available, similar to the current prototype, suitable for field operation and capable of assessing both depth and lateral position with respect to the construction joint. The development of an appropriate specification is being led by Andrew Gisi from the KDOT Geotechnical Unit.

Assessing lateral placement accuracy requires multi-sensor capability. The periodic send signal creates an exponentially decaying magnetic field which generates eddy currents in the tie bar. The eddy currents in turn induce magnetic fields about the tie bar. A receive coil detects the disturbance in the exponentially decaying magnetic field as a result of the presence of the tie steel. Synchronization of the send signals from multiple sensors is necessary if two or more cover meters operate within several meters of one another. If the meters are not synchronized, the timing circuitry of the receive coil will be foiled by the presence of an out-of-phase decaying magnetic field emanating from a different cover meter. The interference due to non-synchronized meters is analogous to the operation of multiple radars in close proximity. Send signals from one radar can interfere with receive signals in a second radar, causing erroneous readings.

The prototype currently under development will be capable of simultaneously collecting three traces. Figure 6 depicts the control circuitry provided by Kolectric that allows for three sensors to operate synchronously. The cart to house the three-sensor version is being designed similarly to the cart shown in Figure 3. At the time of this writing, the multi-sensor prototype is under construction. Although synchronized operation of the three magnetic sensors has been demonstrated in the lab, the effects of proximity on detection range have yet to be assessed.

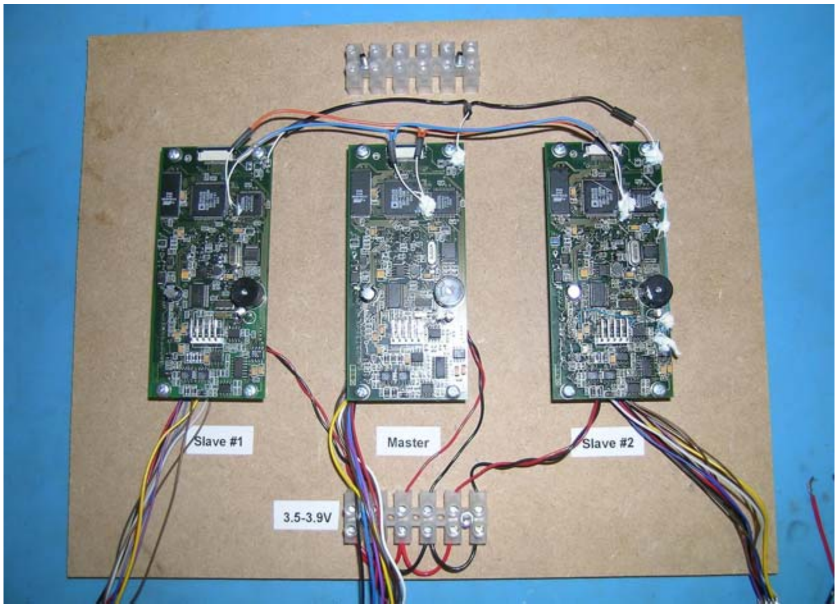

Figure 6. Control circuitry of three synchronized cover meters provided by Kolectric

Figure 7 plots the sensor data from the three synchronized meters during a preliminary laboratory test. In the test, the sensor coils are positioned parallel to one another approximately six inches apart in the following order: Slave1, Master, Slave2. A steel dowel was passed over the array of sensor coils at a constant height (by hand). In the results graphed in Figure 7, Slave1 detects the bar first, then the Master, and then Slave2. On the return trip, the order is reversed, with Slave2 detecting the bar first, then the Master sensor in the middle, and finally Slave1 detects it again. The results in Figure 7 simply demonstrate that multiple sensors can operate in close proximity as long as the control circuitry is synchronized. It is unknown whether the detection range of each individual sensor is diminished due to the proximity of the other sensor coils.

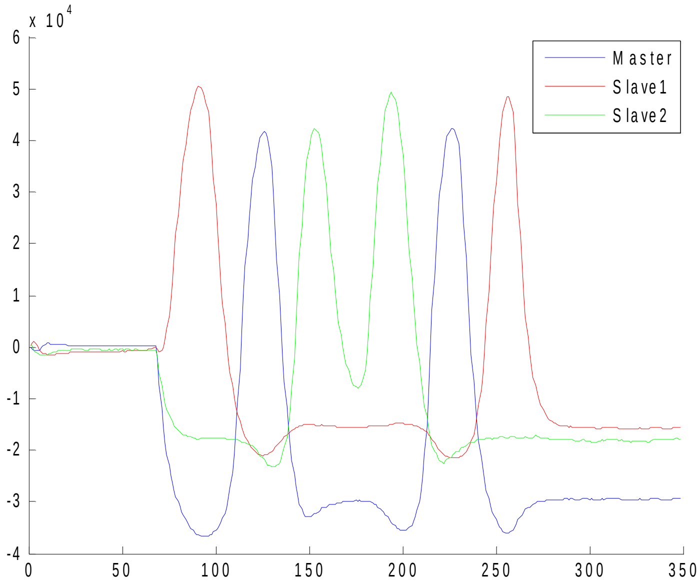

Figure 7. Results of preliminary laboratory test with synchronized meters

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Effects of Materials and Mixing Procedures on Air Void Characteristics of Fresh Concrete

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ABSTRACT

The air void analyzer (AVA) was used to study the effects of concrete materials and mixing procedures on air void characteristics of fresh concrete. Twenty-seven batches of concrete were prepared with three mixes (with and without fly ash or water reducing agent), five mixing procedures (one-step mixing for one, two, or four minutes, two-step mixing for four minutes, and ASTM standard lab mixing procedures), and two sizes of pan mixers (0.5- and 1.5-cf capacity). The air content, size distribution, specific surface, and spacing factor of all the batch mixtures were examined. The results indicated that incorporating 15% fly ash replacement or the recommended dosage of water reducing agent into concrete generally reduced the spacing factor of air voids. The two-step mixing method (mixing mortar first, then adding coarse aggregate) produced a lower air void spacing factor than the one-step (four-minute) mixing method (mixing all concrete materials together at once) and the ASTM mixing method. For concrete mixed with the one-step mixing method, the air void spacing factor reduced with mixing time. For a given concrete mix and mixing procedure, use of different sizes of pan mixers provided the mixtures with different air contents and spacing factors.

Key words: air voids—fly ash—mixing—water reducing agent

INTRODUCTION

Mixing is important to achieve desirable concrete performance and homogeneity. Studying mixing is still difficult because to date there has been no consensus on evaluation criteria for concrete mixing quality, which depends primarily on mixing energy, time, and mixing sequence. The material and mixing systems usually mutually interact. Supplementary cementitous materials (SCMs), such as silica fume, fly ash, slag, natural Pozzolan, etc., have been used in concrete for many years because of the benefit to the environment and durability of the concrete. Some mixtures containing SCMs tend to be sticky and may need additional mixing time to reach uniformity. The requirements for mixing concrete with material-effect consideration, especially SCMs, have not been clearly established in the United States. Powers (1968) pointed out that the air void system is critical for concrete structures, such as pavement and bridge decks subjected to frost/thaw action and deicing salts. The air void system in concrete is generally formed during the mixing process and is significantly affected by both the material and the mixing method. For example, fly ash often contains carbon, which has a large surface area and absorbs the air-entraining agent (AEA); thus, it may impair the air void system. The mixing procedure (time) for normal concrete may not be appropriate for concrete mixing with SCMs.

The air void system in concrete is regarded to be the most significant factor in freeze-thaw resistant concrete (Powers 1968; Schlorholtz 1998). The pressure developed by water as it expands during freezing depends upon the distance the water must travel to the nearest air void. Smaller, closely spaced voids provide better protection by relieving the pressure than larger, more distant void spacing. Common test methods are only capable of measuring the volume of air voids, not the size or spacing of the voids. In the late 1980s, the air void analyzer (AVA) was developed to characterize the air void structure of fresh concrete (Magura 1996; AASHTO TIG 2003). The clear advantage of the AVA is its ability to obtain air void structure information on fresh concrete in less than 30 minutes. With this information, adjustments can be made in the production process to rectify any problems with the air void system during concrete placement.

RESEARCH SIGNIFICANCE

In this research, the AVA was first used as a tool to study the effect of mixing and material on the air void system of portland cement concrete in the laboratory. The correlation between Rapid Air 457 testing and AVA testing are presented to find a better way of measuring mixing quality.

MATERIALS, MIX PROPORTION, AND MIXING METHODS

The chemical properties of the cementitious materials used in this study are summarized in Table 1. Portland cement is an ASTM Type I cement and meets the requirements of ASTM C 150. Class C fly ashes meet the appropriate requirements of ASTM C 618. Concrete sand with a fine modulus of 2.92 and limestone with a nominal maximum size of one inch are used for all concrete mixes. An (AEA), Daravair 1000, is employed in all mixes to gain approximately 6% air content for the concrete. The water reducer (WR) from the same company is applied to selected concrete mixes.

As shown in Table 2, the C-4 mix proportion specified by Iowa DOT is used as the basis, and a total of three mix proportions are used. To determine the effect of the mixture, replacement with 15% of class C fly ash (C-3-C) and the addition of water reducer (C3-WR) are used for two other mix proportions as modifications of the standard C-4 mix.

Table 1. Chemical composition of raw materials

	Type I cement	Class C fly ash
CaO	64.77	24.95
SiO_2	20.97	34.96
Al_2O_3	5.59	19.86
Fe_2O_3	2.27	5.4
K_2O	0.51	0.53
Na_2O	0.19	3.2
(Na_2O)eq.	0.53	-
SO_3	2.99	-

Table 2. Nominal concrete mix proportions

Constituent	Absolute volume	Specific gravity	Pounds per yd^3
Cement	0.118	3.14	624
Water	0.159	1	268
Air	0.06	0	--
Fine aggregate	0.331	2.63	1478
Coarse aggregate	0.332	2.7	1482

Three mixing methods are used to prepare samples for AVA tests:

1. *Standard ASTM C192 lab mixing procedure*. The standard ASTM C192 lab mixing procedure is used as the reference. Coarse aggregate and a portion of the water with AEA are premixed for about 30 seconds. Then sand, cement, and the remainder of the water are added, after which the mixer runs for three minutes. The mixer is stopped for three minutes, after which the mixer runs for another two minutes.
2. *One-step mixing*. All materials, including coarse aggregate, sand, cement, and water, are loaded before mixing. One-, two-, and four-minute mixing times are applied to investigate the effect of mixing time on the air void system of fresh concrete.
3. *Two-step mixing*. Cementitous materials, sand, and half of the water are first mixed for two minutes. Coarse aggregate and the remainder of the water are then added and mixed together for another two minutes.

TESTING METHODS

The air content of fresh concrete is measured by the pressure method per ASTM C231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method." The instrument is shown in Figure 1.

Figure 1. Pressure type B meter

The AVA test (see Figure 2) is performed on a sample of mortar from a cylinder specimen. The sample of mortar is extracted using a 20-ml syringe and vibrated into the fresh concrete with a percussion drill. This sample is injected into the bottom of the AVA testing device, a temperature-conditioned riser column assembly that contains a layer of analysis liquid under a column of water. The analysis liquid has specific properties that ensure that the air void system in the fresh mortar is released into it without affecting the quantity or sizes of the air voids.

Figure 2. Air void analyzer (AVA)

RESULTS AND DISCUSSION

The air void system of concrete can be characterized by the total air content, the specific surface, and the spacing factor. Of these three parameters, the spacing factor is considered to be the most significant indicator of the durability of the cement paste matrix to the freezing and thawing exposure of the concrete. In this study, the AVA test method is used for air void characterization of fresh concrete. The air void distribution, based on paste air content rather concrete content, is used for the following reasons:

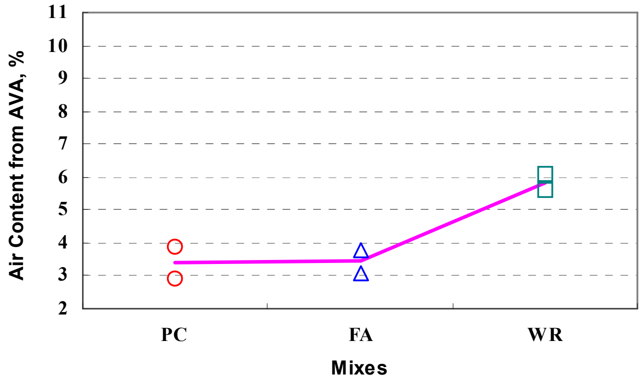
1. Paste content in concrete generally varies with the nominal maximum size of coarse aggregate. The paste needs to be protected by the air voids. Therefore, paste air content is more appropriate for use as an indicator of freeze-thaw resistance.
2. Only mortar samples extracted from the concrete are used in the AVA tests. The concrete air content is calculated from the mix proportion based on the mortar or paste air content. Additionally, the estimated air content is used for calculating the spacing factor, so using the paste air content would probably be more reasonable.

Effect of Concrete Materials on the Air Void System

Three parameters measured from the AVA tests and air void distribution curves of three different mixes using the same standard ASTM lab mixing procedure are plotted in Figures 3 and 4, respectively. As observed from Figure 3, when compared with the portland cement mix, 15% fly ash replacement for portland cement does not change the total air content of concrete. It reduces the spacing factor and increases the specific surface. Addition of a water reducer dramatically increases the total air content, reduces the spacing factor, and increases the specific surface of the air voids.

Figure 4 indicates that, compared with the portland cement concrete mix, adding water reducer generated much smaller air voids (50–300 mm), desirable for freeze-thaw resistance while reducing the larger air voids (1,000–2,000 mm) in the paste system. Fly ash replacement produces a slightly smaller air void (125–500 mm) while significantly reducing the larger air voids (2,000 mm) in the paste. In terms of the total air content of the paste (< 2,000 mm), the water reducer mix has a much higher air content than the portland cement concrete and fly ash mixes, which show about the same total air content in this study. In Figure 4 (b), the accumulative distribution curve of water reducer moved upward and leftward, while the fly ash curve moved leftward from the portland cement concrete mix curve. This also indicates that the water reducer produces finer and a greater number of air voids, and fly ash produces finer air voids.

One-step two-minute and four-minute mixing methods show similar trends with the standard ASTM lab mixing, when comparing the effects of the mixture on the air void system.



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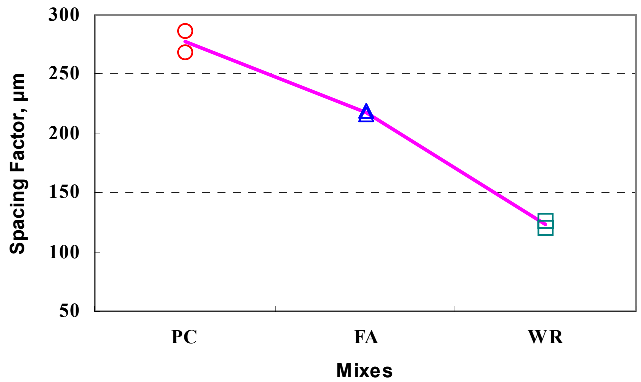

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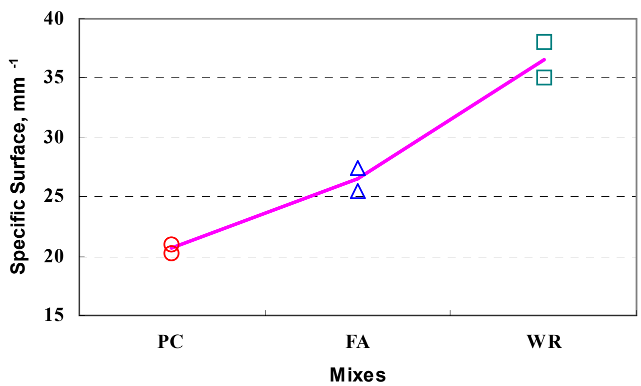
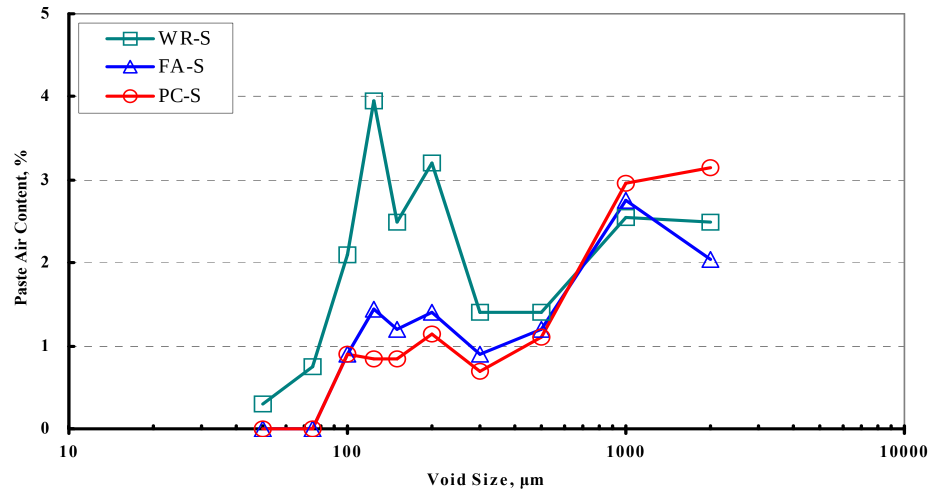


(c)

Figure 3 Effect of mixes on the AVA parameters (3-3-2 mixing)



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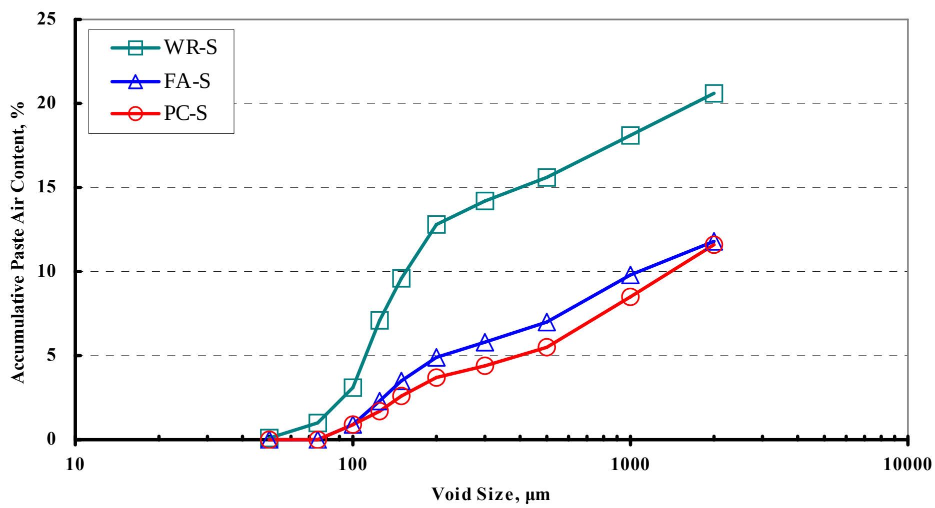


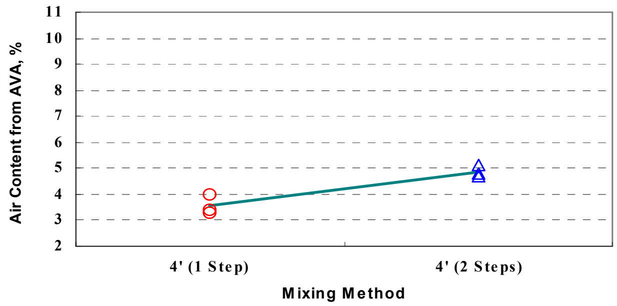
(b)

Figure 4 Effect of mixes on paste air content distribution (3-3-2 mixing)

Effect of Mixing Procedure on Air Void Distribution

Figures 5 and 6 illustrate the AVA test results of concrete mixtures mixed with one-step and two-step methods for four minutes. Compared with the one-step mixing procedure, the two-step mixing procedure produces a slightly higher total concrete air content. The spacing factor is reduced and the specific surface is increased (see Figure 5).

Compared with the one-step mixing procedure, the two-step mixing procedure generates many more and smaller air voids (100–500 mm) while reducing the larger air voids (2,000 mm) in the paste, as shown in Figure 6. In terms of the total air content of the paste (< 2,000 mm), the two-step mixing procedure generates a higher air content than the one-step mixing procedure. The accumulative distribution curve of two-step mixing moves up and leftward from the one-step mixing curve. This also indicates that two-step mixing produces finer and a greater number of air voids than one-step mixing.



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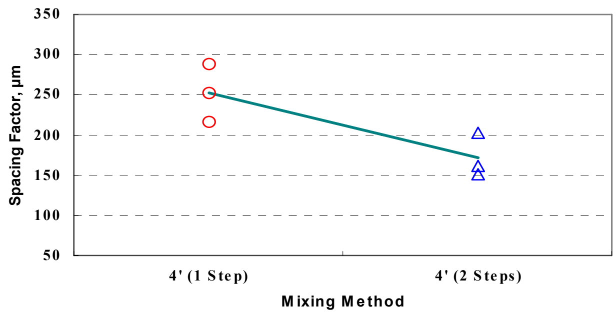

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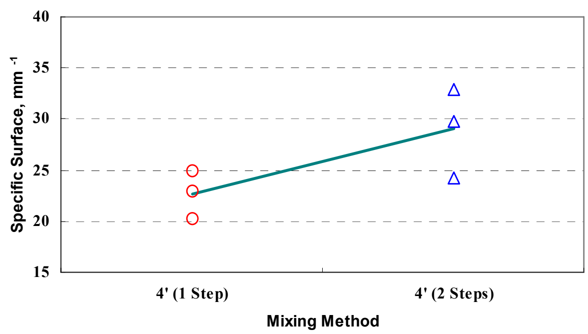
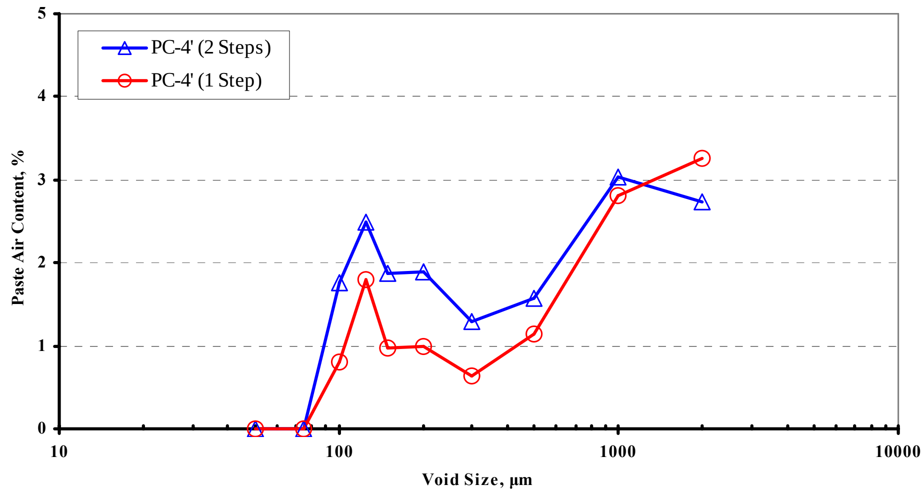


(c)

Figure 5. Effect of mixing sequence on AVA parameters (portland cement, four min. mixing)



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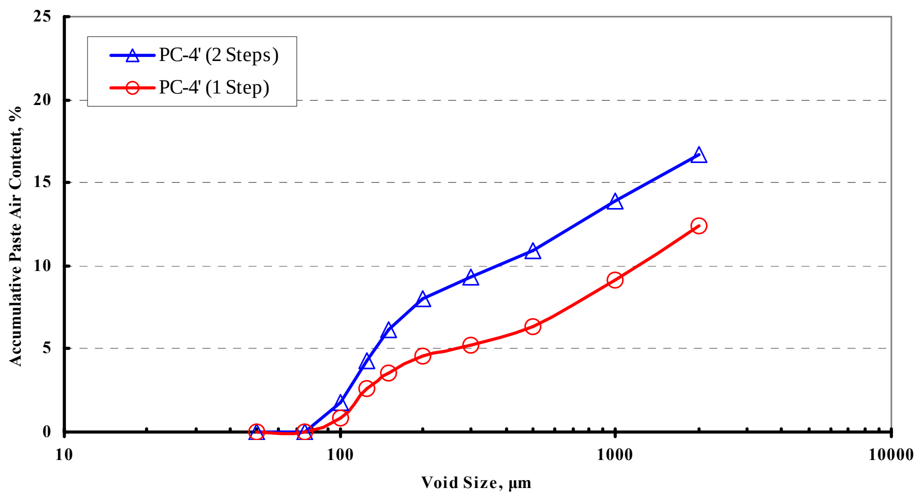


(b)

Figure 6. Effect of mixing sequence on air distribution (portland cement, four min. mixing)

Effect of Mixing Time on Air Void Distribution

Figures 7 and 8 illustrate the effect of mixing time on AVA test results. Three mixing times (one minute, two minutes, and four minutes) are used for one-step mixing of a portland cement concrete mix. Compared to one-minute mixing, two-minute mixing increases the total concrete air content slightly, but has little influence on the specific surface. The spacing factor decreases a little when the mixing time is changed from one minute to two minutes. A four-minute mixing time does not increase the total concrete air content at all from the two-minute mixing time, but it does reduce the spacing factor and increases the specific surface greatly from the one-minute or two-minute mixing times (see Figure 7).

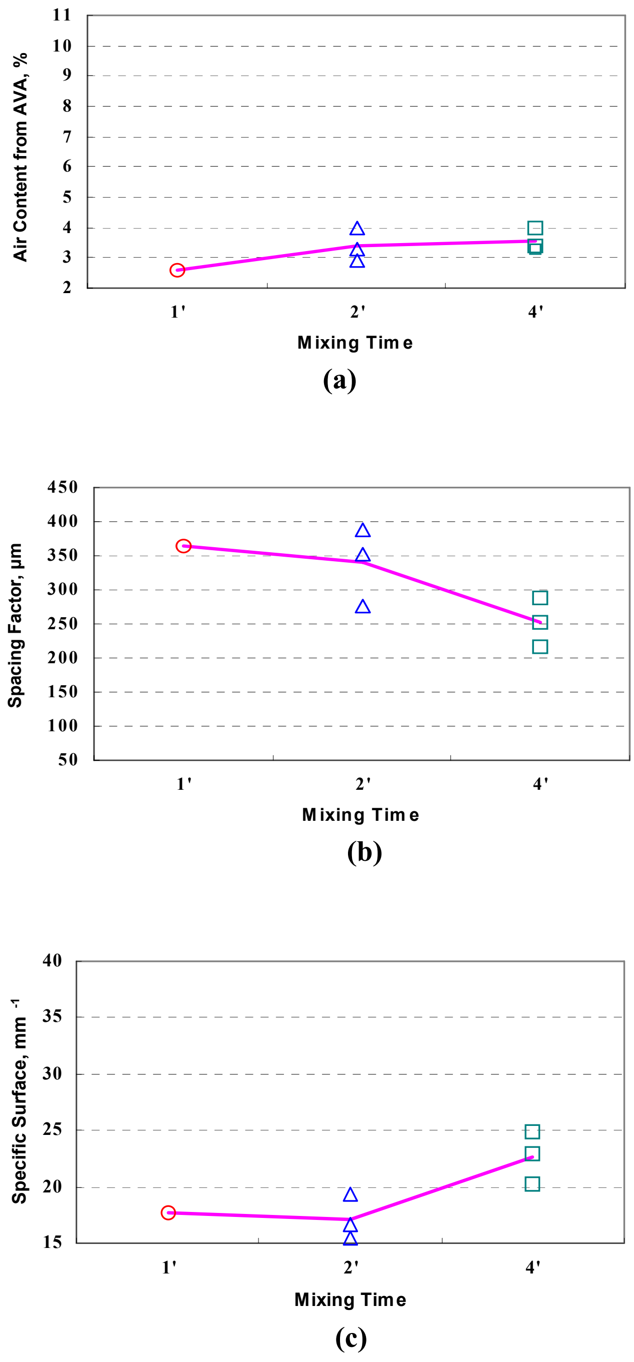
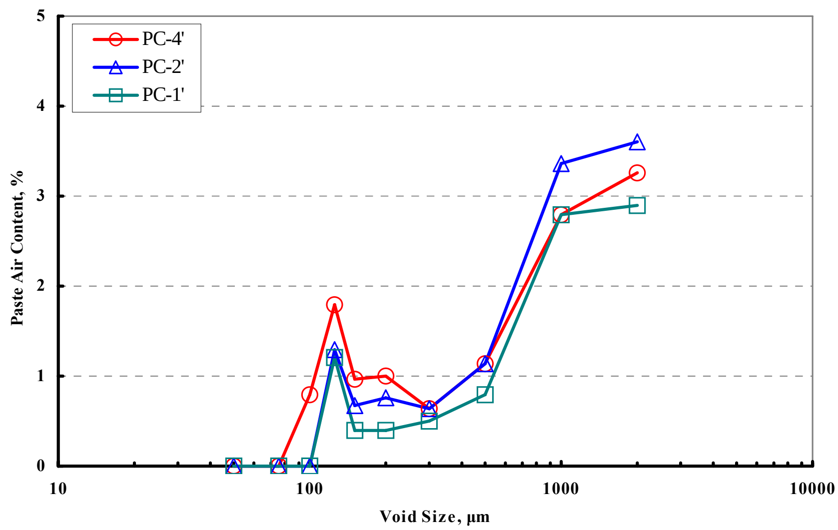


Figure 7. Effect of mixing time on the AVA results (portland cement one-step mixing)

Figure 8 shows the air void distribution curves for three different mixing times. It can be seen that one-minute mixing generates the lowest air content within all size ranges. A two-minute mixing time produces a few smaller air voids (150–500 μm) and the highest number of larger air voids (1,000–2,000 μm). Four-minute mixing increases the number of small air voids (100–200 μm) and produces fewer large air voids (1,000–2,000 μm). Based on Figure 8(b), one-minute mixing has lower total air content in the paste (<2,000 μm) than the two-minute and four-minute mixing. The four-minute mixing generates nearly the same total air content compared with two-minute mixing, but four-minute mixing produces a more desirable air void distribution. The two-minute mixing curve moves leftward and upward from the one-minute mixing curve, and the four-minute curve continues to move leftward from the two-minute mixing curve. This indicates that as mixing time increases, the amount of small air voids in the concrete increase.



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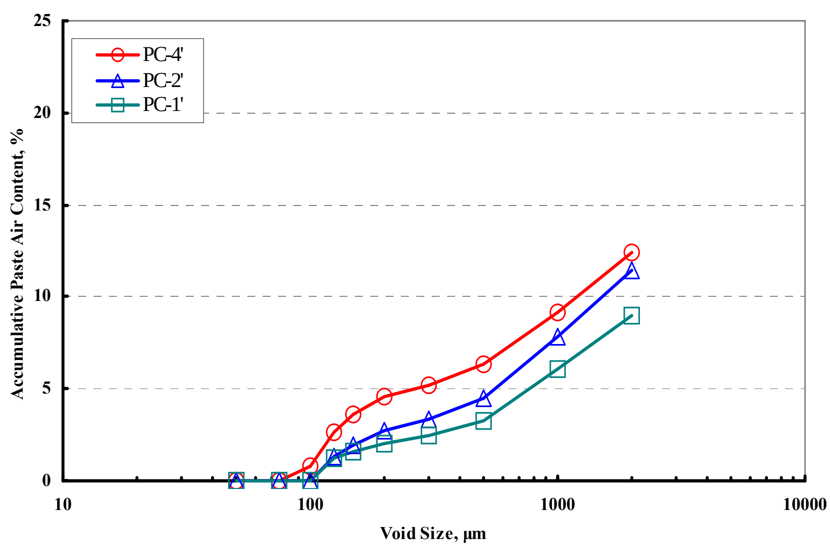

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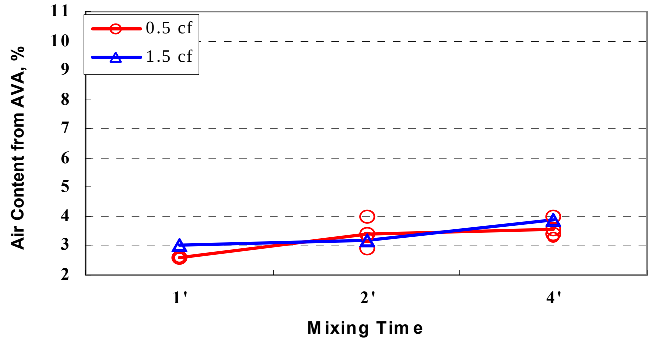
Figure 8. Effect of mixing time on air void curve (portland cement one-step mixing)

Effect of Mixer on Air Void Distribution

Two lab mixers with capacities of 0.5 and 1.5 ft^3 are used in this study. Three numerical parameters measured from the AVA and the average air void distribution curves of two mixers using portland cement mix are plotted in Figures 9 and 10.



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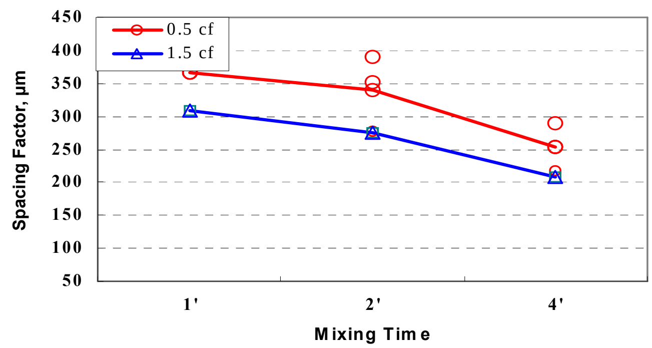

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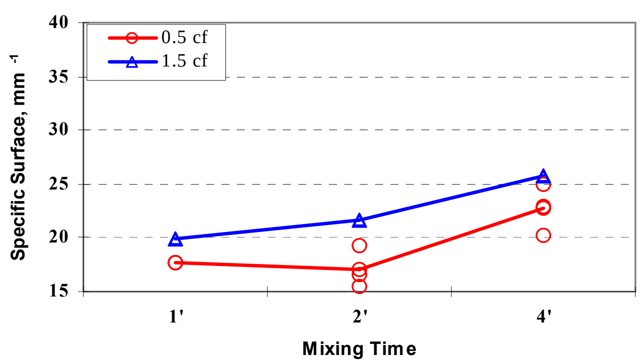
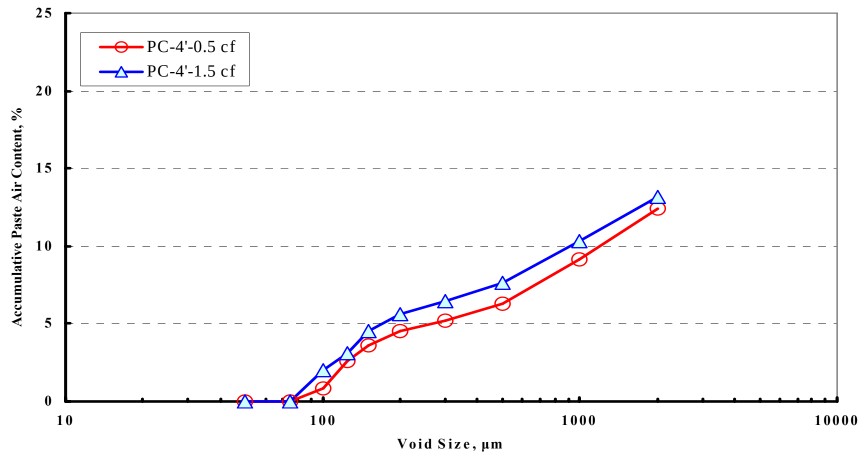


(c)

Figure 9. Effect of mixer on the AVA results (portland cement one-step four-min. mixing)

Based on Figure 9, the big mixer produces nearly the same total air content as the small mixer for the concrete at every mixing time, but it reduces the spacing factor and increases the specific surface of the concrete air void at every mixing time.

The air void distributions of the mixture mixed with two mixers are plotted in Figure 10. According to Figure 10(a), a larger mixer, with more mixing energy, made the distribution curve at smaller air void levels shift leftward. That is, there are more small air voids and fewer large air voids in the mixture. Figure 10(b) indicates that the total air contents of the mixture produced by the two mixers are close. The accumulative air void curve of the mixture made with the larger mixer moves leftward, which indicates more fine air voids in the mixture.



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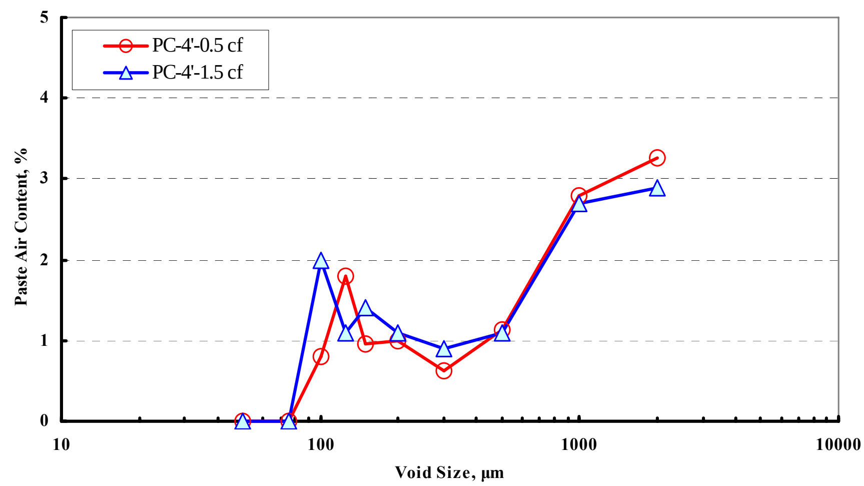

(b)

Figure 10. Mixer effect on air void distribution (portland cement one-step four-min. mixing)

RESEARCH FINDINGS

The major findings from this study are summarized in the following:

1. AVA tests indicate that 15 % fly ash replacement for portland cement reduces the spacing factor of the concrete. Addition of a water reducer also significantly reduces the spacing factor and increases the specific surface of concrete.
2. The two-step mixing method produces a higher spacing factor and increases the amount of small-size air voids while reducing the large air voids, when compared to the one-step mixing method.
3. When a small-pan lab mixer (0.5 cf) is used, mixing time less than four minutes seems insufficient to produce a desirable spacing factor. Two-minute mixing produces more air voids than one-minute mixing, but not as good as four-minute mixing, which produces a desirable air void distribution. Use of a large mixer appears to facilitate obtaining a normal air void system.

CONCLUSION

AVA testing can provide valuable information on the concrete air void system. This testing method is recommended for evaluating mixing and concrete quality control. Not only mixing methods, but also materials used in the concrete mixture, significantly influence concrete air void characteristics.

ACKNOWLEDGMENTS

The present research is part of Shihai Zhang's Master's thesis. The support from the Iowa Highway Research Board and the Center for Portland Cement Concrete Pavement at Iowa State University for this study are greatly appreciated.

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Reducing School Bus Emissions in Texas

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ABSTRACT

School bus emissions are of particular concern, not only for their contribution to overall air quality problems, but also for their impact on children, who are highly vulnerable to this health risk. The purpose of this study was threefold: to develop a methodology to estimate the emissions from school buses in nonattainment (NA) and early action compact (EAC) areas in Texas, perform an order of magnitude estimate of school bus emissions, and develop recommendations for reducing such emissions.

Information about school bus fleets was obtained from 65 out of 228 school districts in the NA and EAC areas in Texas. By combining this data with an array of demographic data, such as school district location, population, minority population, and median household income, a model was developed to estimate the number of buses per category per school district. The study results indicate that school buses in the NA and EAC areas in Texas produce more than 2,400 tons of nitrogen oxides (NOx) emissions and almost 150 tons of particulate matter (PM 2.5) emissions per year. Although these emissions represent only 0.8% and 3.1% of the total NOx and PM 2.5 mobile source emissions, respectively, reducing these emissions is important because it can help NA areas reach attainment and reduce health risks to children. Furthermore, school bus emissions can be reduced by using strategies such as engine retrofitting or replacement, replacement of buses with new clean-burning units, and using cleaner fuels.

Key words: emissions—retrofits—school bus

Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.

INTRODUCTION

Approximately 450,000 school buses operate every day in the United States. These buses transport 24 million children to and from school, which equates to almost 10 billion student rides annually (School Transportation News 2004). Of these buses, approximately 90% are diesel fuel-powered, which has been shown to cause or exacerbate a host of health problems, including respiratory ailments such as asthma and even cancer. Children are particularly vulnerable to the harmful impact of air pollution because they breathe at a faster rate than adults and their lungs are still developing (children breathe 50% more air per pound than adults) (US EPA 2002). In addition, children spend much more time outdoors than adults.

School buses expose children to soot (particulate matter) and smog-forming pollutants (NOx) and hydrocarbons (HC). They also add to the problem of global warming by emitting the greenhouse gas carbon dioxide (CO_2). It is estimated that the nation's school buses release 3,000 tons of soot, 95,000 tons of smog-forming pollutants, and 11 million tons of greenhouse gas emissions annually. Older school buses are exempt from today's stronger emissions standards, but buses built before 1991, which constitute around a 1/3 of buses now in operation in the United States, have six times higher particulate matter (PM 2.5) emission rates and three times higher NOx emissions rates than buses built in 2004 (Monahan 2002).

The purpose of this study was threefold: to develop a methodology to estimate the emissions from school buses in the nonattainment (NA) and early action compact (EAC) areas in Texas, perform an order of magnitude estimate of school bus emissions in NA and EAC areas in Texas, and develop recommendations for reducing such emissions.

TEXAS CASE STUDY

Methodology

The methodology in this study can be summarized in the following steps:

- Focus on the NA and EAC areas as defined by the EPA in the new eight-hour ozone standards.
- Identify the school districts within each of the NA and EAC counties.
- Use U.S. Census Bureau data and other sources to obtain demographic information, such as total area of the school district, population, total enrollment, percentage minority population, and median household income, for each school district.
- Develop criteria for selecting samples of school districts within each of the NA and EAC areas.
- Select the sample school districts based on the pre-determined criteria.
- Use personal interviews and mail-out questionnaires to obtain specific information regarding the school bus fleets and usage rates.
- Develop statistical relationships between the demographic information of the sample school districts and the number of buses per category.
- Develop a procedure to predict the average miles driven per bus category.
- Develop the appropriate emissions rates for the categories and model years of the school buses.
- Calculate emissions estimates for NOx and PM 2.5 for school districts in the NA and EAC areas.
- Aggregate the emissions estimates to NA and EAC counties.
- Identify the most productive emissions control strategies and estimate possible emissions reductions.
- Develop conclusions and recommendations.

Data Collection

Demographic Data for School Districts

Data from the U.S. Census Bureau was used to determine the values for relevant independent variables, such as the total area of the school district, population, minority population, and median household income (U.S. Census 2004). The population for each school district was calculated as a proportion based on surrounding population density and the coverage area of the school district. The number of schools and total enrollment in the various school districts were obtained from a study performed by the Galveston-Houston Association for Smog Prevention (GHASP) to investigate the reduction of air pollution from Houston-area school buses (GHASP 2004). This study examined retrofit and fuel technology options and focused on school districts within five of the eight NA counties. Table 1 shows the summarized demographic data for the various NA and EAC areas.

Table 1. 2000 demographic data for the NA and EAC areas

NA or EAC area	Population	% minority population	Average household income	Total enrollment	School districts	Schools
Austin	812,279	31.8	$64,817	187,672	14	276
Beaumont	385,090	31.7	$38,776	70,988	17	126
Dallas/Fort Worth	4,910,758	31.6	$53,706	966,077	98	1403
Houston	4,715,639	37.3	$44,202	948,881	69	1230
Tyler/Longview	111,379	27.3	$36,825	22,030	6	45
San Antonio	1,392,931	31.1	$42,030	298,344	20	438
Total/Average	**12,328,076**	**31.8**	**$46,726**	**2,493,992**	**224**	**3,518**

Emissions Rates

MOBILE6 was used to develop emissions rates for the various school bus categories. Emissions rates were developed for NOx and PM 2.5. In the MOBILE6 model, only one category is specifically dedicated to the larger type C and D school buses: Heavy-Duty Diesel School Buses (HDDBS). In terms of heavy-duty gasoline buses, the closest relevant category is Heavy-Duty Gasoline Buses (HDGB), which includes school, transit, and urban gasoline-fueled buses. There are no specific categories for the smaller (type A and B) school buses. However, these buses are closely related to the two class 2b heavy-duty vehicles. Specifically, in the case of diesel-fueled type A and B school buses, the selected category is Class 2b Heavy-Duty Diesel Vehicles with 8,500–10,000 gross vehicle weight rating (GVWR) (HDDV2b). In the case of gasoline-fueled type A school buses, the selected category is Class 2b Heavy-Duty Gasoline Vehicles with 8,500–10,000 GVWR (HDGV2b).

MOBILE 6 runs were performed to produce the respective emissions rates. An important assumption in emissions estimation with MOBILE 6 is the average speed, because emissions rates are developed based on different drive cycles, and these drive cycles are defined according to average speeds. Questionnaires and interviews were used to determine the average speeds. The average speeds mentioned by the individual school districts varied between 18 mph and 24 mph, with 20 mph being a reasonable average. Therefore, it was decided to use 20 mph as the average speed for emissions modeling purposes.

Table 2 shows the final emissions rates in grams per mile for the 20-mph scenario. MOBILE6 produces emissions rates for individual years over a 25-year period. The model years were grouped into six groups, as shown in Table 2. In addition, the school bus types are grouped into two categories: those that carry less than 20 passengers (types A and B) and those that carry more than 20 passengers (types C and D). As expected, the rates for larger buses are higher than for smaller vehicles, and are higher for diesel-fueled vehicles than for gasoline-fueled vehicles.

Table 2. Emissions rates for school buses

Model years	Diesel				Gasoline			
	< 20 passengers		> 20 passengers		< 20 passengers		> 20 passengers	
	NOx	PM 2.5	NOx	PM 2.5	NOx	PM 2.5	NOx	PM 2.5
1978-1984	7.47	0.60	12.77	2.94	5.40	0.17	7.46	0.17
1985-1989	7.29	0.60	12.81	2.94	7.01	0.07	10.36	0.08
1990-1993	4.87	0.27	12.51	1.97	4.54	0.06	6.34	0.05
1994-1998	4.46	0.11	13.80	0.18	4.07	0.06	6.31	0.05
1999-2000	3.56	0.11	11.04	0.18	3.37	0.06	4.95	0.05
2001-2004	3.56	0.11	10.40	0.18	3.15	0.06	4.75	0.06

Sample Data

A sample set of school districts was selected within each of the NA and EAC areas for detailed analysis to determine information regarding the bus fleets and usage rates. These school districts were selected based on criteria such as geographic location, median household income, and total enrollment.

The objective was to obtain reasonable sample sizes in terms of the number of school districts within the NA and EAC areas and to have a good spread of the above-mentioned criteria. The respective data elements for the various school districts were organized in quartiles to assist in selecting a good distribution for the sample.

In the case of the Houston/Galveston area, the previous GHASP study generated most of the needed information (GHASP 2004). To supplement the GHASP data, personal interviews and written questionnaires emailed and faxed to the selected school districts were used to obtain information regarding the number of buses by size and model year categories, particularly for districts not covered in the GHASP study. In addition, information was obtained regarding the total miles driven per year, retrofit and alternative fuels used, idling policies, and plans for purchasing additional buses. The response rate was good: 46% of the selected school districts provided the required information. Specifically, for Houston/Galveston, 7 out of 22 responded (32%); for Beaumont/Port Arthur, 3 out of 5 responded (60%); for Dallas/Fort Worth, 10 out of 22 responded (45%); for San Antonio, 4 out of 5 responded (80%); for Austin, 3 out of 5 responded (60%); and for Tyler/Longview, 2 out of 3 responded (66%).

A total of 16 of the 28 respondents indicated that they had some type of idling policy or practice, 9 of the 25 respondents reported using ultra-low sulfur diesel (ULSD) fuel, and 2 school districts indicated that they use their older buses more frequently than the newer buses and reserve the new buses for field trips. In most cases, however, the older buses, especially 1978–1984 model years, are used as back-up buses or used in very limited cases.

Rural versus Urban

It was expected that school districts in rural areas would have different bus fleets and mileage accumulation characteristics than those in urban areas. Urban school districts were defined as those located in cities or towns with a population higher than 30,000. The rural school districts were defined as districts located in towns with a population less than 30,000.

Average Speed

An important assumption in emissions estimation is the average speed, because emissions rates are developed based on different drive cycles, and these drive cycles are defined according to average speeds. It was initially assumed that the average speeds of school buses in rural school districts are higher than those in urban school districts. This trend could not be confirmed based on the feedback from the individual school districts. As mentioned above, an average speed of approximately 20 mph was found.

Number of Buses

A sample of the number of buses per category was obtained from the questionnaires. A regression equation was then developed to obtain an estimate of the total number of buses for the school districts that did not provide information. A broad range of independent variables were tested to determine which ones are statistically significant for estimating the number of buses per school district. The significant variables found were number of minorities in the school district area, total enrollment, and number of schools in the district. An equation represented by the negative binomial distribution produced the best fit to the data (Equation 1).

$$\log N = \left[4.0896 - 0.010\,M\ /1000 + 0.0940\,E\ /1000 - 0.0336\,S\right] - RU \qquad (1)$$

Where

N	=	Number of school buses
M	=	Total minority population in the school district area
E	=	Total enrollment of all the schools in the district
S	=	Total number of schools in the district
RU	=	If in a rural area then RU = 0.4629; otherwise RU = 0

The assessment for the goodness of fit test for a negative binomial regression is the Pearson Chi-Square test, where a result close to 1.00 indicates the model fits the distribution (Social Sciences Teaching and Research Statistics 2004). The Pearson Chi-Square test for this model produced 0.9673, which indicates a good fit for the model. The model was subsequently applied to the remaining school districts in the NA and EAC areas to estimate the number of school buses per district. The total number of buses could then be distributed according to the various categories: age, size, and fuel type (diesel or gasoline). The distributions were based on the information obtained from the questionnaires. Figure 1 shows the age distribution of school buses in the NA and EAC areas in Texas. The figure shows a fairly even distribution of the buses between the different age groups, with the 1994–1998 model year buses representing the largest group.

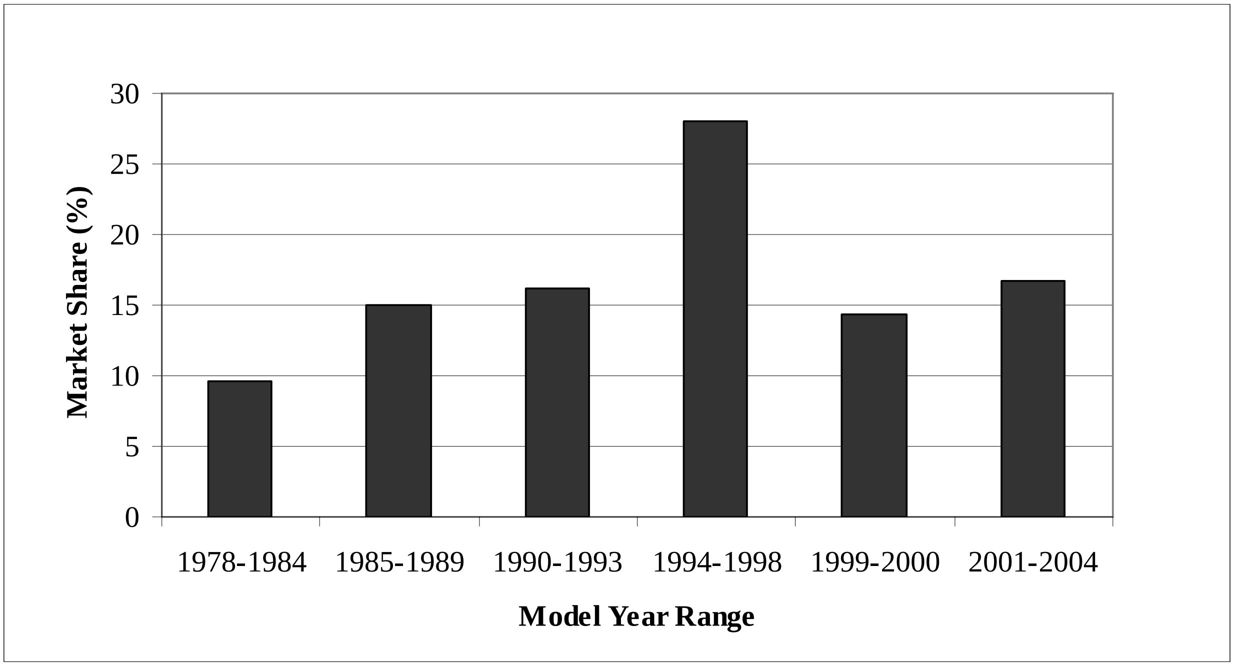

Figure 1. Age distribution of school buses in the NA and EAC areas in Texas

Vehicle Miles of Travel

Another important factor in emissions estimation is the vehicle miles of travel (VMT). Previous research and findings from this study indicate that school districts generally prefer to drive newer buses longer distances than older buses (GHASP 2004). The GHASP study researched the relationship between the model year of school buses and annual miles driven. The study produced a linear relationship between these two parameters. Based on the additional information gathered from the surveys performed for this study, the linear relationship developed through the GHASP study was modified by a factor to define the relationship between the total miles driven per bus per age group.

The total number of buses per school district was estimated using Equation 1. These buses were then distributed according to fractions of the model years, as determined from the sample data. The mid-point of each range was then selected to estimate the annual mileage for that specific range.

Data Analysis

The NOx and PM 2.5 emissions were calculated by multiplying the relevant emission rate for the age and size category with the average annual miles accumulated for that bus category. Emissions were then determined for each school district per model year, school bus category, and rural or urban classification. These emissions were then aggregated to the school district level and then to the NA and EAC county level. Figure 2 illustrates the process followed in this study to estimate the school bus emissions for the various school districts.

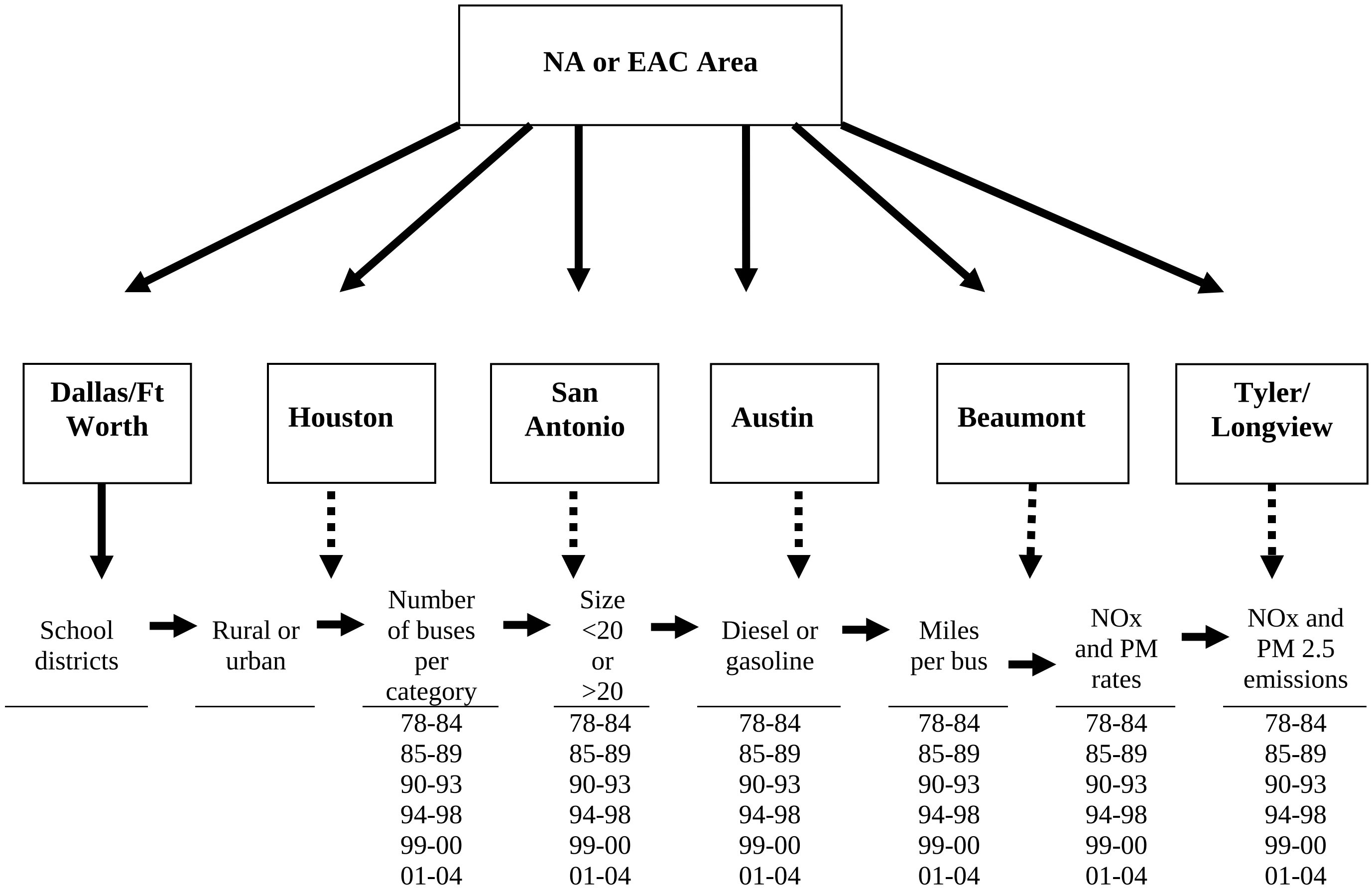

Figure 2. Procedure to estimate school bus emissions

Table 3 shows the results from the analysis by NA and EAC areas. The table shows that school buses in the NA and EAC areas in Texas produce more than 2,400 tons of NOx emissions and almost 150 tons of PM 2.5 emissions per year.

Table 3. School bus emissions for NA or EAC areas

Nonattainment or EAC area	Enrollment	Number of schools	Number of gasoline-fueled buses		Number of diesel-fueled buses		Annual NOx emissions (tons)	Annual PM 2.5 emissions (tons)
			<20	>20	<20	>20		
Austin	187,672	276	9.2	64.1	68.2	791.6	131	9
Beaumont	70,988	126	9.2	64.0	76.1	586.3	84	5
Dallas/Fort Worth	966,077	1403	94.4	660.2	856.2	5510.2	844	52
Houston	948,881	1230	293.4	812.4	645.0	6311.0	1146	64
San Antonio	298,344	438	13.7	95.8	161.5	1243.5	204	16
Tyler/Longview	22,030	45	10.2	22.5	24.6	249.2	36	2
Total	**2,493,992**	**3,518**	**430**	**1,719**	**1,832**	**14,692**	**2,445**	**147**

Table 4 shows the proportion of school bus emissions versus the total on-road mobile source emissions inventory (Texas Transportation Institute 2003). The table shows that the total NOx emissions and PM

2.5 emissions produced by school buses in the NA and EAC areas represent 0.8% and 3.1% of the total on-road emissions, respectively. Although these emissions represent very low percentages of the overall on-road emissions, they are important because reducing these emissions can help NA areas reach conformity and reduce the health risk of a vulnerable sector of the population (school children).

Table 4. Annual school bus emissions vs. total on-road mobile source emissions

NA or EAC area	Total on-road mobile source emissions		Total school bus emissions		% produced by school buses	
	NOx	PM 2.5	NOx	PM 2.5	NOx	PM 2.5
Austin	33,160	501	131	9	0.4	1.8
Beaumont	16,141	203	84	5	0.5	2.5
Dallas/Fort Worth	104,000	1456	844	52	0.8	3.6
Houston	118,070	1810	1146	64	1.0	3.5
San Antonio	45,050	727	270	16	0.6	2.2
Tyler/Longview	6,990	96	36	2	0.5	2.1
Total/Average	**323,411**	**4,793**	**2,445**	**147**	**0.8**	**3.1**

RECOMMENDED EMISSIONS REDUCTION STRATEGIES

Technologies

Table 5 shows a summary of emissions reduction technologies for heavy-duty diesel vehicles such as school buses. Note that some technologies are only in the development phase, whereas others are already commercially available. Compatibility refers to the extent that the specific technology can be used with commercially available school bus engines.

Table 5 shows that there is a wide range of potential NOx and PM 2.5 emissions reductions strategies as a result of the various fuel and engine technologies. Depending on the specific strategies selected by the various school districts, NOx and PM 2.5 emissions can be reduced from 0% to almost 100%. By only considering the EPA-verified retrofit technologies, NOx emissions can on average be reduced by 10% and PM 2.5 emissions by 35%. By applying these averages to the school bus emissions calculated for the school districts in the NA and EAC areas in Texas, up to 244 tons of NOx emissions and 52 tons of PM 2.5 emissions can be reduced per year.

Table 5. Summary of possible emissions reduction technologies for school buses

Technology	% reduction NOx	% reduction PM 2.5	Status	Compatible engines	Cost over 10 years*
Low sulfur diesel	6	11	Available	Most	Low
Diesel emulsions	14	30	Available	Most	Low
Fuel-borne catalyst	15	40	Available	Most	Low
Diesel oxidation catalyst	0	20	Verified	Most	Low
Particulate traps	0	85	Verified	1994 & newer	Medium
Lean NOx catalyst	25	85	Verified	Select	High
NOx trap	90	0	In development	TBD	TBD
Selective catalyst reduction	85	25	Available	Most	High
Exhaust gas recirculation	30	0	Demonstrations	Most	Medium
Fuel line devices	TBD**	TBD	Available	Most	Low
Natural gas	60	90	Ltd. availability	Most	High
Propane	20	90	Available	Different engine	Low
Ethanol	20	75	Available	Different engine	High
Biodiesel	10	20	Available	Most	Low
Methanol	50	TBD	Available	Different engine	Low
Hybrids	50	TBD	In development	Different engine	High
Fuel cells	100	TBD	In development	Different engine	High
Electric vehicles	90	TBD	In development	Different engine	High

*Low: less than $5,000; Medium: from $5,000 to $10,000; High: more than $10,000
**TBD: to be determined

CONCLUSIONS

Overall Findings

- Information regarding school bus fleets was obtained from 65 of 228 school districts in the NA and EAC areas in Texas. The study showed that in Texas NA and EAC areas school buses emit approximately 2,500 tons of NOx emissions and approximately 150 tons of PM 2.5 emissions per year. Although these emissions represent only 0.8% and 3.1% of the total NOx and PM 2.5 mobile source emissions, respectively, reducing these emissions can help NA areas reach attainment and reduce the health risk to a vulnerable sector of the population (school children).
- Several technological approaches can be used to reduce NOx and PM 2.5 emitted by school buses. By only considering the U.S. EPA-verified retrofit technologies, up to 244 tons of NOx emissions and 52 tons of PM 2.5 emissions can be reduced per year in Texas NA and EAC areas.
- In Texas, 35 school districts use propane to power about 1,600 buses. In addition, several school districts are involved in demonstration projects that use Texas Low-Emissions Diesel (TxLED; ultra low-sulfur diesel) fuel and installing certified PM 2.5 traps. Others do not have the resources to purchase the slightly higher priced propane buses or the necessary fueling facilities.
- Programs used by other school districts across the nation include converting to natural gas, biodiesel, and propane. In addition, limited retrofits are installed that typically involve PM filters.
- A total of 16 of the 28 responding school districts indicated that they had some type of idling policy or practice, 9 of the 25 respondents reported using ULSD fuel, and 2 districts indicated that they use their older buses more frequently than the newer buses and reserve the new buses for field trips. In most cases, however, the older buses, especially 1978–1984 model years, are used as back-up buses and/or used in very limited cases.

Strategies to Reduce Emissions

- School bus fleet emissions can be reduced by using the following broad strategies: (1) retrofitting existing buses; (2) replacing the engines of existing buses; (3) replacing existing buses with new clean burning units; and (4) using cleaner fuels, some of which require engine conversions.
- Retrofits are available for newer buses at a cost of less than $5,000 per unit. However, the retrofits will not work effectively without low-sulfur fuel because the retrofits clog up very quickly. In addition, many buses in the current fleet cannot be retrofitted because of age and model-year configurations.
- There are concerns with the reliability of retrofits as well as the ease with which they can be repaired. The preferred strategy, therefore, would be to replace older buses rather than retrofit them. This alternative becomes even more attractive, considering that 42% of all the buses in the Texas fleet are estimated to be over 10 years old. If bus replacement costs are considered to be prohibitive, engine replacements may be considered (approximately $30,000 for a new engine versus more than $60,000 for a new bus).
- In terms of new buses, clean diesel units are available for approximately $7,000. It is important to note that these buses do not operate effectively without the correct fuel type (mostly ULSD fuel).
- Some school districts make a concerted effort to ensure that their older and higher emitting buses drive the shortest routes, whereas other districts do it more randomly. It would be beneficial from an emissions perspective if the newest buses are used on the longest routes.
- Idling could be a large source of school bus emissions, and various strategies are listed in this report for reducing idling. Approximately 60% of the respondents indicated they already have some type of idling policy or practice. Those with policies indicated they limit idling to less than five minutes.

Future Research

- VMT is an important component of school bus emissions. There is a need to develop more accurate estimates of miles driven by the various school bus fleets, particularly per age group.
- There is a need to determine the actual emissions rates of the various categories of school buses relative to age, size, and fuel type.
- There is a need to develop courses for drivers, maintenance personnel, and transportation managers to train them on improved operational aspects and available clean-burning equipment.
- There is a need to investigate the possibilities and implications of having school buses subjected to an inspection and maintenance program.

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